Probabilistic Seismic Loss Assessment of RC High-Rise Buildings in Southern Euro-Mediterranean Zone

Jelena Radenko Pejovic (✉ jelenar@t-com.me)

Faculty of Civil Engineering University of Montenegro <https://orcid.org/0000-0001-6720-7311>

Nina Nikola Serdar

University of Montenegro: Univerzitet Crne Gore

Research Article

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Posted Date: May 27th, 2021

DOI: <https://doi.org/10.21203/rs.3.rs-526102/v1>

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Probabilistic seismic loss assessment of RC high-rise buildings in Southern Euro-Mediterranean zone

Jelena Pejovic⁽¹⁾ and Nina Serdar⁽¹⁾

⁽¹⁾Faculty of Civil Engineering, University of Montenegro, Podgorica, Crna Gora

Corresponding author: Jelena Pejovic, email: jelenapej@ucg.ac.me, ORCID iD: <https://orcid.org/0000-0001-6720-7311>

Abstract

In this paper probabilistic seismic loss assessment of RC high rise-buildings for seismic excitation typical for Southern Euro-Mediterranean zone is presented. The loss assessment methodology developed in paper is based on a comprehensive simulation approach which takes into account ground motion (GM) uncertainty, the random effects in seismic demand as well as in predicting the damage states (DSs). The methodology is implemented on three RC high-rise buildings of 20-story, 30-story and 40-story with core wall structural system designed according to Eurocode 8. The loss functions described by a cumulative lognormal probability distribution are obtained for two intensity level for a large set of simulations (non-linear time-history analyses) based on 60 GM records with wide range of magnitudes, distance to source and different site soil conditions. The losses expressed in percent of building replacement cost for RC high-rise buildings are obtained. In the estimation of losses, both structural (S) and non-structural (NS) damage for four DSs are considered. The effect of different GM characteristics (magnitude, distance to source and site soil condition) on the obtained losses are investigated. It is checked if the estimated performance of the RC high-rise buildings fulfill limit state requirements according to Eurocode 8.

Keywords: RC high-rise buildings, probabilistic seismic loss assessment, loss functions, non-linear time-history analysis, cumulative lognormal probability distribution, random effects.

1 Introduction

The process of urbanization and rapid growth of cities, concentration of material resources in urban areas, intensive migration of people, high land prices, development of construction technology, rapid economic development and prestige of countries are some of the reasons why construction of high-rise buildings has become an everyday design trend in the world. A similar trend is also present in the southern Euro-Mediterranean zone. Given that the high-rise buildings, are usually concentrated economic and human assets, they represent significant elements of seismic risk. For this reason, it is necessary to conduct seismic risk assessment of this population of buildings and to develop strategies for predicting and mitigating their losses in seismically active zones.

By the analysis of large damages and losses that occurred during the earthquakes, it is concluded that traditional force-based design concepts are less suitable for building owners and investors as well as the community in general and do not provide a reliable assessment of seismic building performance. Probabilistic performance-based seismic design (PEER 2010, 2017) is an alternative to traditional design procedures, which in the design process includes predefined desired building performance, defined in probabilistic manner. Desired building performance is described by possible, acceptable losses, i.e. possible levels of structural and non-structural building damage, number of fatalities, economic losses, time during which the facility will be out of order, repair costs and others (Cornell and Krawinkler 2000). Using this method, the economic losses can be analyzed, as well as other types of losses caused by damage. This allows designers, building owners or investors to better understand the losses that may occur as a result of future earthquakes. Building damages as well as losses can be kept within desired limits, that depends on the type of building and its purpose, as well as on the requirements of the investor or owner.

Probabilistic performance-based seismic analysis of a building can be organized into four steps (PEER 2010, 2017; Moehle and Deierlein 2004; Porter 2003; Deierlein 2004; Miranda et al. 2004): 1) Probabilistic Seismic Hazard Analysis, 2) Probabilistic Seismic Demand Analysis, 3) Probabilistic Seismic Damage Analysis and 4) Probabilistic Seismic Loss Analysis. Previously defined steps can be summarized and presented in the form of an equation based on the total probability theorem:

$$P[DV > dv^{LS}/IM] = \int_{DM,EDP,IM} P[DV/DM]/dP[DM/EDP]/dP[EDP/IM]/dIM \quad (1)$$

where:

- $P[DV > dv^{LS}/IM]$ – probability of exceeding the limit state dv^{LS} of a certain parameter of the decision DV for a seismic intensity measure IM
- $P[DV/DM]$ – probability of exceeding a certain parameter of the decision DV for a damage measure DM
- $P[DM/EDP]$ – probability of exceeding the damage measure DM for a seismic demand measure EDP
- $P[EDP/IM]$ – probability of exceeding the seismic demand measure EDP for a seismic intensity measure IM

Within the probabilistic seismic loss analysis, the probabilities of exceeding certain losses (economic, human, time of non-functioning of the facility, etc.) are calculated for a certain level of structural damage, that can be used by the owner / investor to make decisions. As a final result, the probability of exceeding the limit state dv^{LS} of a certain parameter of the decision DV for a seismic intensity measure IM is calculated and loss functions are obtained. Loss functions provide a graphical interpretation of the distribution of losses, i.e. represent the functions of the probability of exceeding a certain loss for a certain intensity level. In seismic risk analysis for loss estimation, the direct economic losses are commonly used as the type of losses. Direct economic losses include the cost of repair and replacement of damaged systems and components.

Different researches are done in order to improve probabilistic seismic loss assessment methodology for different type of buildings. The PEER performance assessment methodology (PEER 2010, 2017) was further developed by Yang et al. 2009. Yang et al. (2009) developed methodology for implementation of a probabilistic performance-based seismic analysis with aim to express performance of a facility in terms of the risk of repair cost, down time and casualties. PEER methodology is also adopted by the ATC-58 (2011), and uses a Monte Carlo simulation procedure to quantify the performance of different structural facilities. Shome et al. (2015) performed loss estimation of tall buildings designed for the PEER Tall Building Initiative Project (PEER TBI, 2017). Based on the dollar results, the performance of the dual-system building is compared with the other buildings systems considered in TBI study. Kappos and Panagopoulos (2010) have tackled a number of issues relating to vulnerability and loss assessment of RC buildings, with particular emphasis on the situation in Southern Europe. In their analysis, nine-story RC buildings were selected as representative of high-rise buildings. Cardone et al. (2020) developed a displacement-based framework for the simplified seismic loss assessment of pre-70 RC buildings up to 8-story located in Italy. Saadat et al. 2016 proposed an optimized probabilistic seismic loss analysis methodology for steel structure and compared it for the structure located in two geographic locations (in central and western United States). Tian et al. (2016) compared seismic losses of tall buildings designed according to Chinese and US codes. Gentile and Galasso (2021) proposed optimal retrofit

selection framework for RC buildings based on simplified seismic loss assessment. Koduru and Haukaas (2010) derived probabilistic seismic loss curves of a 14-story RC high-rise building situated in Vancouver. They used the structural damage model developed by Mehanny and Deierlein (2000) to assess monetary loss based on the structural damage index. Shoraka et al. (2013) developed the loss simulation procedure for estimation of the losses of existing non-ductile concrete buildings considering their vulnerability to collapse. Bradley and Lee (2010) investigate correlations between multiple components in structure-specific seismic loss estimation and their results are presented for a typical New Zealand 10 story office RC building.

Currently, there is a very little information in the literature on seismic loss assessment for RC high-rise buildings designed according to EN1998-1 (2004) and located in the southern Euro-Mediterranean seismic zone. Since EN1998-1 (2004) does not define specific rules for tall buildings, it is very important and useful also to check the seismic performance of high-rise buildings designed in accordance with EN1998-1 (2004).

This paper presents the probabilistic seismic loss assessment for RC high-rise buildings for seismic excitation specific for Southern Euro-Mediterranean zone. For analysis three representative RC high-rise buildings, 20-story, 30-story and 40-story, with core wall structural system designed according to EN1998-1 (2004) are selected. The GM selection is done using the earthquake records from Southern Euro-Mediterranean zone. The developed methodology with characteristic steps is shown in Fig.1. The methodology is based on a comprehensive simulation approach which takes into account GM uncertainty and random effects in seismic demand as well as in predicting the DSs.

Fig.1 Probabilistic seismic loss assessment framework

2 Description of selected RC high-rise buildings

In this paper, three representative RC high-rise buildings, 20-story, 30-story and 40-story, with core wall structural system, are selected for analysis. The base plan of selected buildings is shown in Fig.2.

Fig.2 Base plan of the buildings

The buildings are in size 36x32 m, with spans in the X and Y direction equal to 10x8x8x10 m and 10x6x6x10 m, respectively. The RC core, centrally located in the base plan of the buildings, is 16x12 m. The core consists of three complex cross-section walls (U and I cross-section shape) connected in the X direction by coupling beams. RC frames are formed in the exterior axis. Floor slabs are solid RC slab. Cross section sizes of structural elements differ depending on the building height and they are determined in accordance with the requirements defined in the European provisions EN1992-1-1 (2004) and EN1998-1 (2004). The main characteristics of each individual building are shown in Table 1.

Table 1 The main characteristics of the RC high-rise buildings

Feature	20-story	30- story	40- story
Total height (m)	60	90	120
Story height (m)	3	3	3
RC slab thickness	20cm	20cm	20cm
RC beams	40x65cm	40x65cm	40x65cm
RC columns	80x80cm	80x80cm	90x90cm
RC core wall thickness	1-5 story: 30cm	1-5 story: 40cm	1-10 story: 55cm
	6-20 story: 20cm	6-30 story: 30cm	11-40 story: 45cm
Coupling beams	20x80cm; 30x80cm	30x80cm; 40x80cm	45x80cm; 55x80cm
Concrete f_{ck} (f_{cm}) (MPa)	35(43)	45(53)	55(63)
Reinforcement f_{yk} (f_{ym}) (MPa)	500(575)	500(575)	500(575)
Concrete modulus of elasticity E_{cm} (MPa)	34000	36000	38000

The buildings are designed according to European provisions EN1992-1-1 (2004) and EN1998-1 (2004). The seismic action is applied through an elastic response spectrum, type 1 ($M_s > 5.5$) for soil class A. The buildings are designed for medium ductility class (DCM). The structural system of the buildings is a ductile wall system in both of the horizontal direction according to the EN1998-1 classification, i.e. uncoupled wall system in the X direction and coupled wall system in the Y direction. ETABS2013 (2013) was used for linear seismic analysis. 3D spatial models of structures are developed. Effective stiffness equal to 50% of stiffness of uncracked cross section is adopted for structural elements in accordance to EN1998-1. The floor slabs are modelled as rigid diaphragms in a horizontal plane. Fully fixed supports are assumed at the ground level. The

total base shear forces in two main directions are calculated using response spectrum analysis. Total base shear forces, vibration periods and modal participation factors of individual modes are shown in Table 2.

Table 2 Total base shear forces, vibration periods and modal participation factors for buildings

Buildings			20-story	30-story	40-story
Total base shear force (kN) Y direction			18503	24331	32454
Total base shear force (kN) X direction			16348	23266	32340
Periods Y direction (sec)	Mode	1	1.652	2.880	4.097
		2	0.389	0.623	0.858
		3	0.181	0.270	0.355
Periods X direction (sec)	Mode	1	1.641	2.597	3.511
		2	0.480	0.702	0.880
		3	0.250	0.347	0.423
Mass participation factors (MPFs) in Y direction (%)	Mode	1	64.26	63.53	63.24
		2	20.32	19.43	18.94
		3	7.04	7.05	7.05
Sum of MPFs in Y direction (%)			94.85	93.58	92.88
Mass participation factors (MPFs) in X direction (%)	Ton	1	69.36	67.70	66.08
		2	15.96	17.40	18.78
		3	5.49	5.23	5.68
Sum of MPFs in X direction (%)			93.64	93.11	93.18

Based on the results of the linear analysis, the total seismic force is dominantly accepted by the RC core walls. Therefore, only RC core was considered for detailed design according to the provisions of EN1998-1, and for further conducted non-linear time-history analysis. All EN1998-1 provisions related to the design and detailing of ductile walls have been complied. More on linear seismic design of buildings is given in (Pejovic et al. 2019, 2020).

3 Probabilistic Seismic Hazard Analysis

The first step in probabilistic seismic performance-based analysis is to select GMs for different seismic hazard levels that should be used in the evaluation of the performance of the building for various performance objectives. To account for the complex characteristics of RC high-rise buildings, it is necessary to include a wide range of frequency and seismic energy levels in the selection of GMs because their response frequency range is much wider than for shorter buildings (Ji J et al., 2007)). In this study, natural strong motion records were selected based on magnitude of events (M), distance to source (R), and site soil condition (S). As a location of interest Southern Euro-Mediterranean seismic zone is selected.

The selected natural records are categorized into four combinations: 1) Close to source and large magnitude; 2) Distant to source and large magnitude; 3) Close to source and small magnitude and 4) Distant to source and small magnitude. The details for these categories are presented in Table 3. For each category, 14-16 natural strong motion records are selected from the European earthquake records database (Ambraseys et al. 2002) and Seismological Institute of Montenegro database. In total 60 GMs were selected out of which, 25 were recorded on the rock (EN 1998-1 soil type A) and 35 recorded on stiff soil (EN1998-1 soil type B). The magnitude of selected GMs varies in the range between 5.1 and 7.0 while the distances to source are in the range from 5 to 70km. It is selected that the large magnitudes are in range $M=[6.3-7.0]$, while small magnitudes are in range $M=[5.1-6.2]$. Close to source corresponds to $R=[1-20\text{km}]$, while distant to source correspond to $R=[20-70\text{km}]$. By using a number of records that represent the various magnitudes, distances, and site conditions, this study account GM uncertainty.

Table 3 Selected GM Records

Category	Station data source	M	R(km)	soil	PGA(m/s ²)
I. Close and Large (n=15)	Ulcinj Hotel Albatros, E-W	7	9	rock	2.252
	Ulcinj Hotel Albatros, N-S	7	9	rock	1.715
	Auletta, E-W	6.6	10	rock	0.588
	Auletta, N-S	6.6	10	rock	0.588
	Bagnoli-Irpino, E-W	6.6	6	rock	1.776
	Bagnoli-Irpino, N-S	6.6	6	rock	1.364
	Sturno, E-W	6.6	14	rock	3.168
	Sturno, N-S	6.6	14	rock	2.122
	Ulcinj Hotel Olimpik, E-W	7	9	stiff soil	2.405
	Ulcinj Hotel Olimpik, N-S	7	9	stiff soil	2.598
	Petrovac Hotel Oliva, E-W	7	12	stiff soil	3.019
	Petrovac Hotel Oliva, N-S	7	12	stiff soil	4.491
	Bar Skupština Opštine, E-W	7	12	stiff soil	3.659
	Bar Skupština Opštine, N-S	7	12	stiff soil	3.639
	Calitri, N-S	6.6	14	stiff soil	1.526
II. Distant and Large (n=16)	Titograd Seizmološka stanica, E-W	7	46	rock	0.309
	Titograd Seizmološka stanica, N-S	7	46	rock	0.286
	Herceg Novi O Škola D.Pavičić, E-W	7	29	rock	2.553
	Herceg Novi O Škola D.Pavičić, N-S	7	29	rock	2.126
	Rionero in Vulture, E-W	6.6	30	rock	0.975
	Rionero in Vulture, N-S	6.6	30	rock	0.969
	Tricarico, E-W	6.6	63	rock	0.343
	Tricarico, N-S	6.6	63	rock	0.466
	Torre del Greco, E-W	6.6	65	rock	0.4
	Torre del Greco, N-S	6.6	65	rock	0.593
	Benevento, E-W	6.6	41	stiff soil	0.532
	Benevento, N-S	6.6	41	stiff soil	0.381
	Titograd Geološki zavod, E-W	7	46	stiff soil	0.496
	Titograd Geološki zavod, N-S	7	46	stiff soil	0.364
	Bisaccia, E-W	6.6	22	stiff soil	0.779
	Bisaccia, N-S	6.6	22	stiff soil	0.904
III. Close and Small (n=14)	Ulcinj Hotel Albatros, E-W	5.1	19	rock	0.601
	Kotor Naselje Rakite, E-W	6.1	19	rock	1.5
	Kotor Naselje Rakite, N-S	6.1	19	rock	1.117
	Herceg Novi O Škola D.Pavičić, E-W	6.1	18	rock	0.746
	Herceg Novi O Škola D.Pavičić, N-S	6.1	18	rock	0.653
	Tivat Aerodrom, E-W	6.1	15	stiff soil	1.371
	Kotor Zavod za biologiju mora, E-W	6.1	19	stiff soil	0.552
	Kotor Zavod za biologiju mora, N-S	6.1	19	stiff soil	0.588
	Petrovac Hotel Rivijera, E-W	6.1	7	stiff soil	2.863
	Petrovac Hotel Rivijera, N-S	6.1	7	stiff soil	1.78
	Bar Skupština Opštine, E-W	6.1	12	stiff soil	2.209
	Bar Skupština Opštine, N-S	6.1	12	stiff soil	2.051
	San Rocco, N-S	6.2	12	stiff soil	0.65
	Kalamata-Prefecture, N355	5.5	5	stiff soil	2.911

IV. Distant and Small (n=15)	Kotor Naselje Rakite, E-W	5.1	20	rock	0.566
	Kotor Naselje Rakite, N-S	5.1	20	rock	0.846
	Bevagna, EW	5.6	25	stiff soil	0.528
	Ulcinj Hotel Olimpik, E-W	6.1	30	stiff soil	0.584
	Ulcinj Hotel Olimpik, N-S	6.1	30	stiff soil	0.734
	Bevagna, NS	5.6	25	stiff soil	0.335
	Mascioni, E-W	5.5	37	stiff soil	0.367
	Priština Zavod za urbanizam, E-W	5.9	90	stiff soil	0.293
	Priština Zavod za urbanizam, N-S	5.9	90	stiff soil	0.275
	Mascioni, N-S	5.5	37	stiff soil	0.351
	Niš Osnovna škola D.Jovanović, N-S	5.9	83	stiff soil	0.367
	Codroipo, E-W	6.2	35	stiff soil	0.197
	Senigallia, E-W	5.8	71	stiff soil	0.362
	Gonen-Meteoroloji , NS	5.8	45	stiff soil	0.5
	Izmir-Bayindirlik TRAN	5.5	30	stiff soil	0.384

Seismic hazard of the considered location is defined through the design response spectrum corresponding to the seismic intensity level for reference return period of 475 years (probability of exceedance, 10%, in 50 years). As a target design spectrum, elastic EC8 spectrum for reference return period of 475 years with design ground acceleration of 0.37g was selected. The mean squared error method (MSE) was chosen for scaling of GMs (PEER GM 2010). By this method GMs are scaled in a way where the mean squared error is minimized over the whole range of periods. The mean square error represents the difference between the spectral acceleration of GM records and target spectrum and it is calculated by Eq.2.

$$MSE = \frac{\sum_{i=1}^n [s_a^{target}(T_i) - f \cdot s_a^{record}(T_i)]^2}{n} \quad (2)$$

Parameter f in Eq.2 is a linear scale factor. The mean squared error method (MSE) is effective in the selection of GMs since it allows to choose, from the large number of available records, GMs whose response spectra match best the target spectrum. In this way, it is possible to discard the GMs for which dispersion of seismic response parameters will be large.

Seismic hazard level which has reference return period of 2475 years (probability of exceedance 2% in 50 years) is also considered for loss estimation. According to results of the project *Seismic hazard harmonization in Europe-SHARE* (Giardini et al. 2013), seismic intensity which corresponds to return period of 2475 years for the Southern Euro-Mediterranean territory is two times higher than seismic intensity which corresponds to return period of 475 years. Fig.3 and Fig.4 show the response spectra of selected GMs for the intensity level of 10%/50 and the mean spectra for 10%/50 and 2%/50 as well as plus-minus one standard deviation from the mean (16th percentile and 84th percentile spectra) for 10%/50.

Fig.3 Response spectra for soil type A

Fig.4 Response spectra for soil type B

3.1 Random effects on seismic demand

For further understanding of the effects of GM uncertainty on the demand, it is necessary to evaluate the influences of basic features of GM records such as magnitude, distance to source and site soil condition. Fig.5 shows the Matsumura mean spectrum intensities (Matsumura, 1992) of selected GMs for four selected categories and for each building. The Matsumura mean spectrum intensity (the area below the elastic spectrum of velocity between the periods T_y and $2T_y$, where T_y is the yield period of the structure) is selected to evaluate the influence of the frequency contents from different GMs. Dashed horizontal line for every category represents the mean value of intensity measure for that category of GMs. The variations in SIs are relatively large for individual GMs. But if the mean values of SIs for each category are compared, it can be seen from Fig.5 that there is no significant difference between each category.

Fig.5 Mean spectrum intensity for buildings

Further, Fig.6 shows the variations in seismic demands (the maximum inter-story drift over the height of the building (IDR_{max})) of buildings caused by different GM categories in Y direction.

Fig.6 The variation in IDR_{max} of buildings for different categories

As expected, the variations in IDR_{max} values approximately follow the trend of mean spectrum intensities which capture well wide frequency range of high-rise buildings. It is evident that large magnitude earthquakes cause large seismic demands but the differences are not so pronounced compared to small magnitude earthquakes. Even small magnitude and more distant earthquakes cause large seismic demands. Also there is no significant difference in the level of seismic demands for different categories of distance to source. For small magnitude earthquakes (III and IV category) larger seismic demands are obtained for more distant earthquakes.

Fig.7 shows the variations in seismic demands (IDR_{max}) of buildings caused by different site soil conditions. As expected, the seismic demands are larger for stiff soil than for the rock. This happens because GMs at stiff soil sites induce larger responses from longer period modes, as it is for RC high-rise buildings.

Fig.7 The variation in IDR_{max} of buildings for different soil conditions

Fig.8 shows the random effects on the IDR_{max} values from the selected GMs records and different intensities (depending on PGA). Continuous lines present mean values while dashed lines are values of individual GMs. It can be clearly noted that variations in IDR_{max} are large even within the same GMs category. This indicates the importance of using wide range of different GM characteristics for evaluation the seismic performance of high-rise buildings.

Fig.8 IDR_{max} variation, mean and COV values for selected GMs categories

4 Probabilistic Seismic Demand Analysis

The second step in probabilistic seismic performance-based analysis is demand analysis to determine the appropriate demand parameters (EDPs) to best describe its response (Bachman 2004, Saadat et al. 2019). In nonlinear time-history analyses of the buildings for selected GMs, the EDPs are derived. The EDPs that are used in seismic loss assessment are the maximum inter-story drift over the height of the building (IDR_{max}) and the maximum peak floor acceleration over the height of the building (PFA). Nonlinear time-history analysis using the PERFORM-3D computer program (CSI 2006) is used to obtain the EDPs (IDR_{max} s and PFAs).

The nonlinear models are designed as a 3D models and they consist of RC core walls. The PERFORM-3D models of the considered buildings are shown in Fig.9. The core walls are modeled using non-linear vertical fiber elements representing the expected behavior of the concrete and reinforcing steel (Powell 2007). The area and location of reinforcement within the cross-section and the properties of the concrete are defined using individual fibers. The shear behavior is modeled as elastic. The behavior for out-of-plane bending and behavior in horizontal transverse plane are assumed to be elastic. The hinge lengths at the base of the wall are adopted according to EN1998-1 (2004). The coupling beams are defined as elastic beam elements with a nonlinear displacement shear hinge at the mid-span of the beam. These are connected to the shear walls using embedded elements as suggested by Powell (2007). The shear hinge behavior is based on test results by Wallace (2012). Structural element properties are based on mean values of the properties of the materials. Stress-strain relationship for unconfined concrete, confined concrete and reinforcement steel are adopted in accordance with the recommendations of EN1998-1 (2004). The floor slabs are modeled as rigid diaphragms.

Fig.9 PERFORM-3D models of considered buildings

The distributions of IDR_{max} and PFA in Y direction of 30-story building over the height of building for the 60 selected GMs and both intensity levels are shown in Fig.10 and Fig.11. Similar results are obtained for other considered buildings. Besides distributions over the height for each record, their mean values as well as plus-minus one standard deviation from the mean (16th percentile and 84th percentile spectra) are shown. Analyzing the distributions of IDR_{max} and PFA, it is concluded that they follow the lognormal distribution. Histograms of IDR_{max} and PFA and corresponding lognormal fits for 30-story buildings are shown on Fig.12. The Fig.12 indicates a good lognormal fit to the data (confirmed by the Kolmogorov test).

Fig.10 Distribution of a) IDR_{max} and b) PFA of 30-story building under the 60 GMs for intensity level 10%/50

Fig.11 Distribution of a) IDR_{max} and b) PFA of 30-story building under the 60 GMs for intensity level 2%/50

Fig.12 Histograms and corresponding lognormal fits of a) IDR_{max} and b) PFA for 30-story buildings

The Fig.10 and Fig.11 indicate that IDR_{max} increases across the height but does not change much at higher stories. The PFA is relatively constant across the building height except the top few stories where PFA values increases rapidly reaching its maximum at the top. The lognormal standard deviations of demands (approximately equal to coefficient of variation COV) over the height are shown in Fig.13.

Fig.13 The lognormal standard deviations of demands (coefficient of variation COV) over the building heights: a) IDR_{max} and b) PFA

In addition, the histograms of IDR_{max} and PFA and the corresponding lognormal fits at two illustrative stories (15 and 30) are shown in Fig.14. The differences in these distributions for different stories refer to possibility of accounting variation of IDR_{max} and PFA over the height of the buildings for high-rise buildings for future studies. In this study loss estimation is based on maximum values of inter-story drifts and peak floor accelerations over the height of the building.

Fig.14 Histograms and the corresponding lognormal fits of a) IDR_{max} and b) PFA at two illustrative stories (15 and 30)

5 Probabilistic Seismic Damage Analysis

Realistic and comprehensive DSs determination is one of the most important step in the probabilistic seismic performance-based analysis because of their direct impact on derivation of fragility curves (Erberik and Elnashai 2004). For the considered RC high-rise buildings, the detailed probabilistic seismic damage analysis is done in order to quantify the DSs through the inter-story drifts. As the result of this analysis, inter-story drifts at threshold of DS were defined as random variables with the range of possible values (Table 4). The results of this study are shown in (Pejovic and Jankovic 2016).

Table 4 Inter-story drifts at threshold of DSs for the RC high-rise buildings (Pejovic and Jankovic 2016)

DSs	Inter-story drift at threshold of DS IDR _{max} (%)			Lower and upper endpoint of the 84% confidence interval		Relative width of confidence interval
	Median (50 th percentile)	16 th percentile	84 th percentile			
				L ₁	L ₂	i (%)
Slight	0.250	0.190	0.330	0.247	0.253	2.40
Moderate	0.528	0.398	0.702	0.522	0.534	2.27
Extensive	0.945	0.710	1.260	0.935	0.955	2.12
Complete	1.640	1.230	2.190	1.622	1.658	2.19

For the purpose of seismic loss assessment of the buildings, the fragility curves that indicate the probability of various type of damage due to a given DS as a function of the EDP should be developed. Fragility curves are defined by lognormal distribution of the conditional probability of damage exceeding certain DM given EDP (HAZUS 2003, Saadat et al. 2019) (Eq.3).

$$P[DM \geq DM_i/EDP] = \Phi \left[\frac{1}{\sigma_{DM}} \ln \frac{EDP}{EDP_{DM}} \right] \quad (3)$$

Where EDP_{DM} is the median value of the considered EDP and σ_{DM} is the lognormal standard deviation of the EDP for the DM considered (slight, moderate, extensive and complete).

In this study for the loss estimation both structural damage (S) and non-structural damage (NS) are analyzed. NS is divided into (according to HAZUS (2003)): 1) acceleration-sensitive damage (NSA) (damage to ceilings, equipment that is an integral part of the facility such as mechanical and electrical equipment, piping and

elevators) and 2) drift-sensitive damage (NSD) (partitions, exterior walls, ornamentation and glass). IDR_{max} is used to predict the DSs of the S and the NSD damage, while PFA is used to predict the NSA damage. The fragility curves for S damage are obtained for quantitatively defined DSs (through derived mean inter-story drifts at threshold of DS (Table 4)) for considered buildings. In Fig.15a and 15b are shown the structural fragility curves for total sample of three buildings for four DSs (slight, moderate, extensive and complete) and both considered intensity level. The random effects in predicting the DSs is also captured in this study analyzing the structural fragilities obtained for 16th and 84th percentile of inter-story drifts at threshold of DS (Table 4) (Fig.16 and Fig.17).

Fig.15 Fragility functions for S damage for a) 10%/50 and b) 2%/50 intensity level

Fig.16 Fragility functions for S damage for 10%/50 intensity level obtained for a)16th percentile and b)84th percentile inter-story drifts

Fig.17 Fragility functions for S damage for 2%/50 intensity level obtained for a)16th percentile and b)84th percentile inter-story drifts

Fragility functions for NSD damage are developed based on the information in HAZUS (2003). Using the parameters given for fragility functions in HAZUS (2003) for RC high-rise building type C2H (concrete shear walls), the functions for NSD damage are developed. Fig.18 shows the fragility functions for NSD damage

for total sample of three buildings for four DSs (slight, moderate, extensive and complete) and both considered intensity level.

Fig.18 Fragility functions for NSD damage for a) 10%/50 and b) 2%/50 intensity level

Fragility functions for NSA damage are also developed based on the information in HAZUS (2003). Using the parameters given for fragility functions in HAZUS (2003) for RC high-rise building type C2H (concrete shear walls), the functions for NSA damage are developed. Fig.19 shows the fragility functions for NSA damage for total sample of three buildings for four DSs (slight, moderate, extensive and complete) and both considered intensity level.

Fig.19 Fragility functions for NSA damage for a) 10%/50 and b) 2%/50 intensity level

6 Probabilistic seismic loss analysis

Methodology for loss computation is based on simulation-based Monte Carlo procedure as it is also suggested in ATC-58 performance assessment approach (ATC-58 2011). For applying Monte Carlo procedure a large set (hundreds) of simulations is required per intensity level to generate a loss function (ATC-58 2011). The large sets of simulations are generated directly by a large number of analyses. For each simulation (analysis) per specific intensity level the single value of the performance measure (loss) is calculated. By repeating the simulations for different GM time histories for considered individual buildings, a distribution of loss is constructed for the chosen intensity level and corresponding mean value and dispersion are obtained. Using the obtained mean value and dispersion of the assumed loss distribution, the corresponding loss function is constructed. The calculated losses involve both S and NS damage.

Schematic representation of the developed methodology for loss computation consists of four main steps as it is shown in Fig.20:

- 1) For specific intensity level IM_i , the EDPs (IDR_{max} and PFA) are obtained by conducting nonlinear time-history analyses for all chosen GMs scaled to specific intensity level. The scatter plot of performed simulations EDP_i (nonlinear time-history analysis results in terms of IDR_{max} and PFA) for considered intensity levels are obtained.

- 2) The structural fragility curves defined by lognormal distribution of the conditional probability of damage exceeding certain DM given EDP (Eq.3) $P[DM \geq DM_i / EDP]$ are constructed. The fragility curves for

NS damage: 1) NSA damage $P[DM \geq DM_i/PFA]$ and 2) NSD damage $P[DM \geq DM_i/IDR_{max}]$ are constructed.

3) In third step, the calculation of the losses (e.g. direct economic losses) is done using Eqs.4-7 similar to those given by HAZUS (2003) and Saadat et al. (2019). The expected losses expressed in percent of building replacement cost (%BRC) for each type of damage (S, NSA and NSD) are calculated for a specific intensity level as

$$CS = \sum_{i=1}^{n=4} P[DM \geq DM_i/EDP] \cdot RCS_{DMi} \quad (4)$$

$$CNSD = \sum_{i=1}^{n=4} P[DM \geq DM_i/IDR_{max}] \cdot RCD_{DMi} \quad (5)$$

$$CNSA = \sum_{i=1}^{n=4} P[DM \geq DM_i/PFA] \cdot RCA_{DMi} \quad (6)$$

Where $P[DM \geq DM_i/EDP]$, $P[DM \geq DM_i/IDR_{max}]$ and $P[DM \geq DM_i/PFA]$ are probabilities obtained in step 2 of the procedure, RCS_{DMi} , RCD_{DMi} and RCA_{DMi} are S, NSA and NSD repair cost ratios due to DM_i which varies from slight ($i=1$) to complete ($i=4$) (HAZUS 2003).

Total expected loss CT for a particular building is calculated as the sum of losses for all type of damages as

$$CT = CS + CNSD + CNSA \quad (7)$$

After calculating losses for all simulations, a distribution of losses for a chosen intensity level is constructed and the corresponding mean value and dispersion are obtained.

4) Using the calculated mean value and dispersion of the assumed loss distribution, the corresponding loss function is constructed.

Fig.20 Schematic representation of the methodology for loss computation

7 Methodology applied on selected buildings

7.1 Derivation of loss functions

The considered RC high-rise buildings are exposed to the set of 60 GMs time-histories scaled to two intensity levels (10%/50 and 2%/50) in both directions of the buildings. The total of 360 simulations are performed for both intensity levels. For each performed simulation (analysis) per considered intensity level the single value of the performance measure (loss) based on mean structural fragilities (obtained by 50th percentile inter-story drifts at threshold of DS (Table 4, Fig.15)) is calculated. The repair cost (RC) ratios (%BRC) for S, NSA and NSD damage and different DSs used for calculation of the losses are based on the values given in (HAZUS 2003) for

a residential occupancy class (RES4) (Table 5). By repeating the simulations for different GM time histories for considered individual buildings, a distribution of losses is constructed for the chosen intensity levels (Fig.21).

Table 5 Considered RC ratios (%BRC) for S, NSA and NSD damage and different DSs (HAZUS 2003)

Damage	RC (%BRC)			
	Slight	Moderate	Extensive	Complete
Structural (S)	0.20%	1.40%	6.80%	13.60%
Acceleration sensitive (NSA)	0.90%	4.30%	13%	43.20%
Drift sensitive damage (NSD)	0.90%	4.30%	21.60%	43.20%
Total	2%	10%	41.40%	100%

Fig.21 Distribution of simulated losses of buildings at two considered intensity levels

Losses histogram CTs are obtained and it is shown that the distribution of losses corresponds to the lognormal distribution (Fig.22). The figure indicates a relatively good lognormal fit to the data (confirmed by the Kolmogorov test).

Fig.22 Losses histograms and the corresponding lognormal fits for a)10%/50 and b)2%/50 intensity levels

Based on the obtained distribution results, the loss functions are described by a cumulative lognormal probability distribution, which is fully characterized by a mean value and a dispersion. The parameters of the lognormal loss distribution for two analyzed intensity levels are shown in Table 6. Besides the mean value and standard deviation, the 84th percentile and 16th percentile are shown as well as plus-minus one sigma confidence interval $[L_1, L_2]$ and its relative width. The derived values of confidence interval relative width indicates relatively good level of accuracy of calculated random variables.

Table 6 Log-normal distribution parameters of loss functions

Intensity level	Mean μ	Standard deviation σ	84 th percentile PGA(g)	16 th percentile PGA(g)	Lower and upper endpoint of the 84% confidence interval		Relative width of confidence interval
					L ₁	L ₂	i %
10%/50	10.1	0.65	19.5	5.3	10.5	9.8	6.88
2%/50	30.0	0.68	59.1	15.3	31.1	29.0	7.14

Using the calculated mean value and dispersion of the lognormal loss distribution, the corresponding loss functions are constructed (Fig.23). The expected loss CT is expressed as the total repair cost of the structure in percent of building replacement cost (%BRC).

Fig.23 Distribution of losses for considered intensity levels for total sample of three buildings

Fig.23 and Table 6 identify a mean loss of total repair cost of 10.1(%BRC) for intensity level 10%/50 and 30.0(%BRC) for intensity level 2%/50. The losses that correspond to 84th and 16th percentile are: 1) 19.5(%BRC) and 5.3(%BRC) per 10%/50 and 2) 59.1(%BRC) and 15.3(%BRC) per 2%/50. 84th percentile is a measure of loss that is commonly used by stakeholders or insurers and identifies the loss that has a 84% probability of nonexceedance. This value can be termed as probable maximum loss or upper bound loss for lognormally distributed random variables, while 16th percentile is lower bound loss. The Fig.23 illustrates that there is 84% probability that the loss will not exceed the value of 19.5(%BRC) for intensity 10%/50 and that it will fall with 68% probability between 5.3(%BRC) and 19.5(%BRC). Similarly, the losses will not exceed the value of 59.1(%BRC) with 84% probability for intensity 2%/50, and the loss will fall between 15.3(%BRC) and 59.1(%BRC) with 68% probability.

With the aim to check if the considered high-rise buildings fulfill performance limit states according to Eurocode 8, the obtained results are compared with performance requirements given in EN1998-3 (2005) related to limit states: 1) Near Collapse (NC) with return period of 2.475 years, corresponding to a probability of exceedance of 2% in 50 years and 2) Significant Damage (SD) with return period of 475 years, corresponding to a probability of exceedance of 10% in 50 years. Given that the probable maximum loss is 59.1(%BRC) for intensity level 2%/50 (refer to EN1998-3 NC limit state), it can be noticed that buildings fulfill this performance level because the range of RC ratios between extensive and complete damage is [41.4%-100%]). For this intensity level, even buildings performance is closer to the lower limit i.e. the beginning of extensive damage. The probable maximum loss for intensity level 10%/50 (refer to EN1998-3 SD limit state) is 19.5(%BRC), so the buildings fulfil also this performance level since the RC ratio is within the moderate and extensive damage [10.0%-41.4%]). For both performance level there is a certain safety factor of non-reaching the upper limit of performance (approximately 1.7 for NC and 2.1 for SD). Such results indicate that RC high-rise buildings designed according to Eurocode 8 achieve greater safety than it is required by performance requirements for NC and SD.

As discussed earlier, in estimation of losses, the random effects in predicting the DSs is also captured analyzing the structural fragilities obtained for 16th and 84th percentile of inter-story drifts at threshold of DS (Table 4) (Fig.16 and Fig.17). The variability in the loss results due to the 16th and 84th percentile inter-story

drifts at threshold of DS is shown in Fig.24. The lower and upper ends of the error represent the losses obtained using 84th and 16th percentile inter-story drifts at threshold of DS. It is found that the 10%/50 losses based on 84th and 16th percentiles are 10.3% lower and 17.8% higher respectively than based on the mean inter-story drifts (50th percentile), while the 2%/50 losses are 8.5% lower and 11.9% higher.

Fig.24 Random effects in loss results

Contribution of different type of damage to the maximum probable losses at the two intensity levels is shown in Fig.25. It is known that the relative contribution of S and NS repair costs to the total repair cost varies significantly with the seismic intensity level (Shome et al. 2015). The repair cost at lower intensities is dominated by NS damage, whereas the same for the S damage becomes significant at higher intensities. The Fig.25 shows that a significant part of the loss at lower 10%/50 intensity level is due to the NSD and NSA damage (68% of the total damage). At higher 2%/50 intensity level the S losses increase compared to NS losses.

Fig.25 Contribution of different type of damage to the maximum probable losses at the two intensity levels

7.2 Loss comparison for different GM categories

The comparison of the obtained losses for considered GM categories are shown in Fig.26. The obtained losses are shown separately for both intensity levels and for two directions (X and Y) of the buildings. Dashed horizontal line for every category represents the mean value of loss for that GM category. As expected, the losses are in line with the seismic demands (IDR_{maxs}). The variations in CTs are relatively large for individual GMs. If the mean values of CTs for each category are compared, it can be seen from Fig.26 that there is no significant difference between each category. From the obtained losses in Y direction, it can be noticed that large magnitude earthquakes cause higher losses but the differences are not so pronounced compared to small magnitude earthquake while also there is no pronounced difference for different distance to source. Even for more distant earthquakes (II category and IV category), the losses are at the same level such as for close earthquakes. These results indicates the possibility of higher seismic risk of RC high-rise buildings even for more distant earthquakes, therefore the importance of seismic risk evaluation of RC high-rise buildings for wide range of different earthquake characteristics is highlighted. Also these conclusions are confirmed with results in X direction in which there is no clear difference between the mean values for each category.

Fig.26 The comparison of CTs for different GM categories: a) 10%/50 in Y direction, b) 10%/50 in X direction, c) 2%/50 in Y direction and d) 2%/50 in X direction

7.3 Loss function comparison for different building heights

In order to analyze the effect of building heights, the loss functions are constructed for each considered RC high-rise building height (20-story, 30-story and 40-story) (Fig.27). From the Fig.27 it can be noticed that there is no significant differences in losses for different building heights. This is expected since three considered buildings are designed according to same Eurocode 8 provisions for fulfillment its performance requirements. This also confirms the validity of using integrated sample of three considered high-rise buildings for constructing the loss functions.

Fig.27 Loss functions for different building heights

7.4 Loss functions comparison for different site soil condition

The Fig.28 presents the comparison of obtained loss functions for two soil types, the rock and stiff soil (soil types A and B according to EN1998-1). From the Fig.28 it can be noticed that there is no significant differences in losses for rock and stiff soil. The slightly higher median losses are obtained for B soil type (~16.5%) i.e. stiff soil condition for both intensity levels. Also, the 10%/50 probable maximum loss is higher (~14%) for B soil type, while 2%/50 maximum loss not differ more than 2%. From these results, it can be concluded that the stiff soil sites are likely to produce higher losses in the high-rise buildings than rock sites.

Fig.28 Loss functions comparison for soil type A (rock) and soil type B (stiff soil)

8 Conclusions

In this study probabilistic seismic loss assessment of RC high rise-buildings for seismic excitation typical for Southern Euro-Mediterranean zone is presented. The loss assessment methodology developed in paper is based on a comprehensive simulation approach which takes into account GM uncertainty, the random effects in seismic demand as well as in predicting the DSs. The random effects in predicting the DSs are captured taking into account, besides the mean structural fragilities, also the structural fragilities obtained for 16th and 84th percentile of inter-story drifts at threshold of particular DS. The loss functions and parameters of log-normal cumulative loss distribution for two intensity levels (10%/50 and 2%/50) are derived by performing 360 simulations for both intensity level based on 60 GMs categorized into four combinations depending on level of magnitude, distance to source and site soil condition. Calculated relative width of confidence interval (lower than 10 %) for the derived log-normal cumulative distribution loss function parameters indicate good level of accuracy of loss functions and their possible implementation for RC high-rise buildings with RC core wall or similar structural systems in Southern Euro-Mediterranean zone. The losses based on mean, 16th and 84th structural fragilities (i.e. obtained by 50th, 16th and 84th percentile inter-story drifts at threshold of DSs) are derived. It is found that the 10%/50 losses based on 84th and 16th percentiles are 10.3% lower and 17.8% higher respectively than based on the mean inter-story drifts (50th percentile), while the 2%/50 losses are 8.5% lower and 11.9% higher. In estimation of losses, both S and NS damage are considered. Analyzing contribution of different type of damage to the maximum probable losses, it is concluded that a significant part of the loss at lower 10%/50 intensity level is due to NSD and NSA damage (~68% of total damage), while at higher 2%/50 intensity level the S loss dominate compared to NS losses.

The effect of different GM categories on the obtained losses are investigated. The large magnitude earthquakes cause higher losses but the differences are not so pronounced compared to small magnitude earthquakes while there is no pronounced difference for different distance to source. Such results indicates the possibility of higher seismic risk of RC high-rise buildings even for more distant earthquakes, so the importance of seismic risk evaluation of RC high-rise buildings for wide range of different earthquake characteristics is highlighted. There is no significant differences in losses for different building heights that is expected since three considered buildings are designed according to same Eurocode 8 provisions for fulfillment its performance requirements. The losses of RC high-rise buildings is slightly higher for stiff soil condition than on the rock.

Finally, it is checked if the estimated performance of the buildings fulfill limit state requirements according to EN1998-3: 1) NC with return period of 2.475 years, corresponding to a probability of exceedance of 2% in 50 years and 2) SD with return period of 475 years, corresponding to a probability of exceedance of 10% in 50 years. For both performance level there is a certain safety factor of non-reaching the upper limit of performance (approximately 1.7 for NC and 2.1 for SD). Such results indicate that RC high-rise buildings designed according to Eurocode 8 achieve greater safety than it is required by performance requirements for NC and SD.

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Declarations**Funding**

No funding was received for conducting this study.

Conflicts of interest/Competing interests

The authors have no conflicts of interest to declare that are relevant to the content of this article.

Availability of data and material

All data and materials support published claims and comply with field standards.

Code availability

All data and materials as well as software application or custom code support published claims and comply with field standards.

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