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Robustness and fragility of the RC building designed according to yu-81 and European Standards

Miloš Čokić[1], Boris Folić[2], Radomir Folić[3]

[1] *PhD student*, University of Novi Sad, Faculty of Technical Sciences, Novi Sad, Serbia, *cokicmilos@gmail.com*
[2] *Dr. Ing.,* University of Belgrade, Faculty of Mechanical Engineering, Innovation Centre, Belgrade, Serbia, *boris.folic@gmail.com*
[3] *Professor Emeritus*, University of Novi Sad, Faculty of Technical Sciences, Department of Civil Engineering and Geodesy, Novi Sad, Serbia, *folic@uns.ac.rs*

Abstract

In the countries of the former Yugoslavia, the first regulations for the design of seismically resistant structures date back to 1964, and the reason for their adoption was the Skopje earthquake in 1963. The next, much more advanced structural code dates from 1981 with a few additions. Many RC multi-storey buildings were designed and built according to the mentioned code for high-rise buildings from 1981, especially in larger cities. Some of the reasons were that they are mainly calculated according to their bearing capacity and global displacements values, and not according to the interstory lateral deformations. However, there was no possibility to correlate their reliability to resist earthquake actions with modern International documents (starting from ATC 40 (1996) - until the latest one from 2017, EC 8-part 3, FEMA 273, etc.), nor was it checked later. Recent earthquakes, including the Montenegrin (1979) earthquake, shortly before the 1981 Regulations, showed the vulnerability and even progressive collapse of reinforced concrete (RC) structures. In many countries, for more significant structures, an assessment of their condition from the aspect of vulnerability and/or robustness is performed. It is significant that many seismically resistant structures have shown less sensitivity to progressive collapse because their integrity is, to some extent, a barrier against the occurrence of progressive collapse. Such analyses precede the reinforcement detailing of existing structures to the calculated seismic action characterized, above all, by the intensity and return period. In this paper, a brief application of nonlinear modern methods of analysis and reliability assessment of these structures, with risk analysis according to the performance limit states of the building is performed. The robustness and seismic fragility of two models of an RC building with the same geometric characteristics were analysed, with one building designed according to European building design standards and the other according to Yugoslav regulations from 1981. The goal was to establish the difference in the performance of the two structures, because there is a large number of a reinforced concrete buildings designed by applying old codes in this region. The methodology of calculation and design of RC buildings, necessary for the mentioned analyses, is described in the paper. Numerical analyses were conducted on RC building designed according to Yugoslav standards from 1981 and according to the mentioned modern norms (Eurocode) with a comparison of the important indicators in the obtained results.

Key words: RC building structure, nonlinear analysis, seismic analysis, robustness, fragility, comparative analysis

1 Introduction

In the domain of seismic engineering, fragility function can be used to calculate the possibility for different states of damage to occur in certain construction, at observed value of intensity measure (IM). This methodology can as well be applied to robustness analysis. According to [1], one can define a fragility function as a mathematical function that expresses the probability that some undesirable event occurs as a function of some measure of environmental excitation. Fragility function represents the cumulative distribution function of the capacity of an asset to resist an undesirable limit state.

Most frequently, progressive collapse of buildings structures is initiated when one or more vertical bearing element is removed under extreme events (terrorist attacks, vehicle impacts, explosion, etc.). The chain reaction which occurs after a local failure is transferred to adjacent elements and leads to a progressive collapse of most part of the structure, or entire structure. The most extensive review of numerical and experimental research and codes devoted to progressive collapse, with comparative analyses is presented in [2, 3].

In this paper, fragility curves are calculated for two types of 5 story reinforced concrete (RC) building that exhibits the properties of the frame structural system [4], which is the most common type in Balkan region. The first structural system was designed according to set of structural Eurocodes [4-7] and the other according to Yugoslav regulations from 1981 (JUS) [8]. The goal was to establish the difference in the performance of the two structures, because there is a large number of a reinforced concrete buildings designed by applying old codes in this region. Seismic fragility curves for different types of damage are calculated using the damage state (DS) values according to [9]. Seismic structural response is analysed through nonlinear static analysis (NSA) method. Fragility curves were constructed using the methodology described in [9]. Robustness fragility curves are calculated for different scenarios in which the vertical column was completely removed by incidental actions were analysed, as the most critical cases. Structural response of the structure is obtained using NSA and NDA. NDA results were used to observe the occurrence of damage limit states (LS) in each model and calculate the fragility curves of the system. Based on the analysis, obtained results are compared and final remarks and conclusions were formulated.

2 Methodology of the analysis and structural modelling

2.1 Geometric and material properties of the structure

The subject of the analysis is office-residential building (Fig. 1) with 5 levels (ground floor+4 stories). The structural system exhibits the properties of a frame structural system [4]. The plan view and the 3D model of the structure are shown in Fig. 1. The length of one span in both directions is 4.8 m which makes the total length of the building 19.2

m in both directions. The height of the first story is 3.6 m and the height of the other stories is 3.2 m which makes the total height of the building 16.4 m. In order to simplify the modelling and calculation process, all vertical elements are fixed at the bottom level of the structure, i.e. soil-structure interaction is not included in the calculation and design.

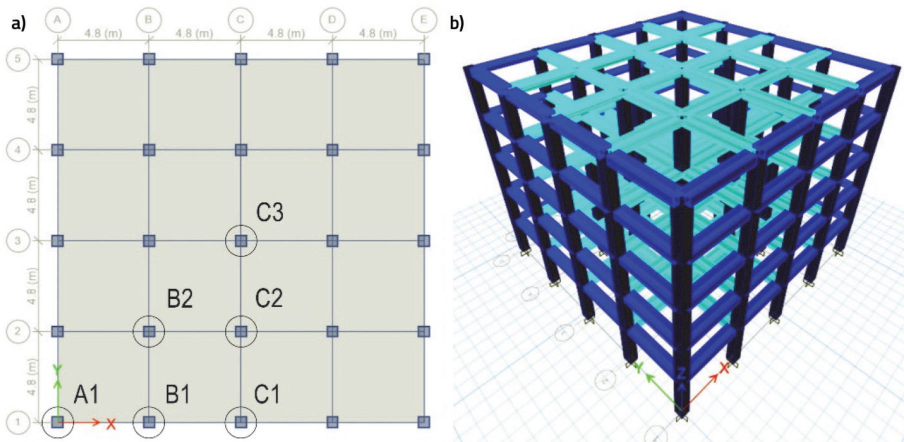

Figure 1. a) Building plan; b) Numerical model [10]

The design of the structural model M1 is done according to the recommendations given in the set of structural Eurocodes [4, 5, 6, 7] and the design of the structural model M2 is done according to [8], using linear-elastic analysis methods. Model M1 is designed with concrete C30/37 (f_{ck} = 30 MPa, E = 33 GPa) [5] and model M2 with the concrete MB30 (f_{ck} = 30 MPa, E = 31.5 GPa) [8] and rebar that corresponds to steel class C [5] and RA400/500 [8] (f_{yk} = 400 MPa, E_s = 200 GPa) with the ultimate strain limit equal to ε_{su} = 7.5 % [4].

The structural design is done according to the European building design standards, for the structure that has ductility class high (DCH) behaviour [4]. The calculations are performed using [10]. Geometric and reinforcement characteristics of the cross-section properties of the beams and columns are shown in Fig. 2, where $b_{eff,S}$ represents the effective plate width for the seismic [4] and $b_{eff,R}$ represents the effective plate width for the robustness [5] analysis.

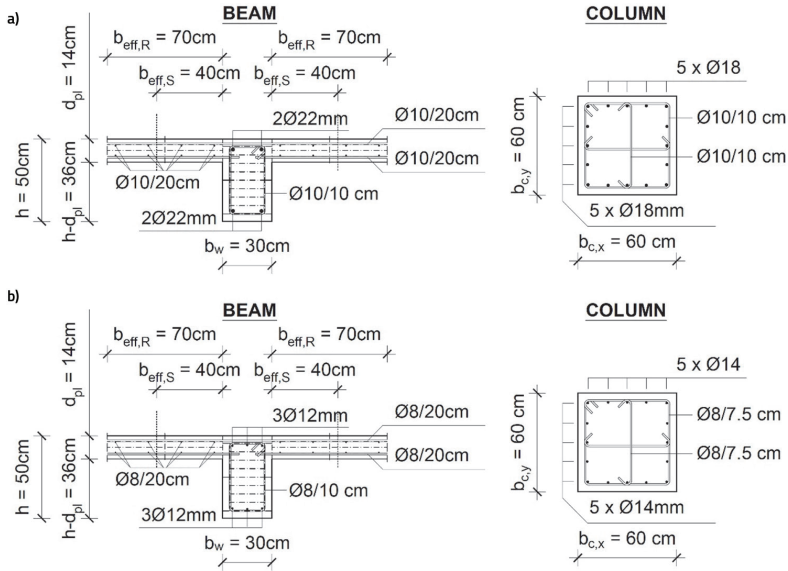

Figure 2. Geometric and reinforcement characteristics of the cross-section properties a) for M1 and b) for M2

2.2 Loads and actions

The loads acting on the structure are as follows: permanent loads (G_i) – self-weight of structural elements and an additional permanent load; the variable-live load (Q_i) and the seismic load (S_i). The adopted value of the permanent constant load is g_{pl} = 3.0 kN/m^2 and the load intensity of the variable-live load amounts q = 2.0 kN/m^2 on all floors except the roof, where it amounts q_R = 1.0 kN/m^2. The self-weight load of façade elements, which is imposed on all façade elements except on the roof is equal to g_f = 10.0 kN/m. The value of the reduction factor of the live loads is $\psi_{2,i}$ = 0.3 [6]. To calculate the earthquake impact on the structure, an elastic response spectrum, type 1 was used, for ground type C, with the PGA $a_g = 0.2 \cdot g$. Behaviour factor q of the design response spectrum for a frame structural system is equal to 5.85 [4]. Load combinations and design values of actions for calculations are used according [6] for model M1 and according to [8] for model M2. Load combinations, for nonlinear seismic analysis, is used according to [6] for both models. For the nonlinear robustness analysis, load combinations are [11, 12]:

$$W = 1.2 \cdot G_i + 0.5 \cdot Q_i \tag{1}$$

$$Q_R = \Omega_R \cdot \left(1.2 \cdot G_i + 0.5 \cdot Q_i\right) = \Omega_R \cdot W \tag{2}$$

where represents the gravity loads combination and represents additional gravity loads parameter or dynamic increase factor (DIF) of the additional gravity load for the analysis of the non-linear behaviour of the structural system. DIF is incrementally increased for the robustness analysis until the collapse, demanded state or non-convergence of the model is reached. In both NSA and NDA procedures, loading of the structure and the column removal scenario in the NDA are done according to the [11, 12] provisions. This procedure [11, 12] is described in detail in [13].

2.3 Modal analysis

Rayleigh viscous (mass – tangent stiffness) proportional damping was used in THA. An overview of modern seismic analyses with different damping models is explained in [14]. The appropriate values of periods of vibration in both directions for T_1 correspond to the first translation period in *X* or *Y* direction (the structure is orthogonal in the plane and its behaviour will be the same in the both main directions). The value of T_2 corresponds to the period of vibration in which the structure reaches at least 90 % of the sum of the effective modal masses [4].

Calculation parameters of interest for the robustness analysis are the first and the last period of vibrations $T_{1,i}$ and $T_{2,i}$. T_1 corresponds to the first vertical translation period in *Z* direction, related to the removed column node's vertical displacement. The value of T_2 corresponds to the vertical translation period in *Z* direction, related to the removed column node's vertical displacement in which the structure reaches at least 90 % of the sum of the effective modal masses in the *Z* direction. Values of the used periods in seismic and robustness analysis are shown in Table 1.

Table 1. Vibration periods for seismic and robustness analysis

Model (Design Code)	**M1 (EN)**		**M2 (JUS)**	
Vibration periods [s]	**T_1 (ΣM_{eff})**	**T_2 (ΣM_{eff})**	**T_1 (ΣM_{eff})**	**T_2 (ΣM_{eff})**
Seismic analysis	0.742 (80.53 % M)	0.215 (92.75 % M)	0.880 (78.85 % M)	0.245 (92.12 % M)
Robustness analysis	T_1	T_2	T_1	T_2
A1	0.220	0.023	0.326	0.024
B1	0.195	0.023	0.321	0.024
C1	0.193	0.022	0.321	0.023
B2	0.210	0.023	0.333	0.024
C2	0.205	0.024	0.322	0.026
C3	0.199	0.023	0.322	0.029

3 Structural model

3.1 Model for linear-elastic analysis

For calculation and design of the structure in [10], a spatial (3D) model was used. The following parameters, assumptions and simplifications were adopted:

- The calculation includes the effects of second order logic (P-Δ);
- Occurrence of cracks in structural elements was included in the calculation with the stiffness reduction of the elements according to [4].
- The elastic bending stiffness and shear stiffness of columns and beams was reduced to 50 %;
- Torsion stiffness of columns and beams was reduced to 10 % of their elastic stiffness;
- The elastic stiffness of the RC plate was reduced to 50 %.

3.2 Model for nonlinear analysis

In models for post-elastic analysis of structural response to the removal of individual vertical elements, the following assumptions and simplifications were used:

- The calculation includes the effects of second order logic (P-Δ);
- To describe the nonlinear behaviour of the material, the nonlinear properties of the material were used to describe the behaviour of concrete (Fig. 3) and reinforcement steel [5, 15, 16];
- Parameters describing the appearance of cracks as a result of elastic bending stiffness in structural elements from the linear-elastic model were not included in the nonlinear model, because plastic hinges are modelled as fiber elements, whereas the properties of fibers are described by stress-strain relations in concrete and reinforcement steel (Fig. 3);
- Columns and beams were modelled as confined RC elements with a protective layer of concrete [15,16];
- The beams are modelled as "L" and "T" cross sections, with the effective width of the RC plate. $b_{eff,S}$ [4] was used for seismic and $b_{eff,R}$ [5] for robustness analysis (Fig.2).
- RC plates are modelled as rigid diaphragms in seismic analysis.
- In robustness analysis, RC plates are included in the calculation models through corresponding effective widths within beams, i.e., the plates are not treated as surface elements. The consequence of this simplification is that the results may indicate lower system robustness than the actual one, but the calculation favours safety.

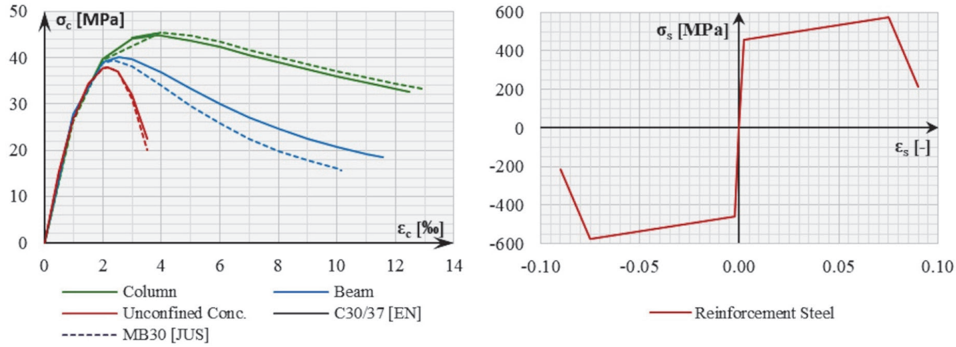

Figure 3. Material properties of concrete (left) and rebar (right)

3.3 Properties of plastic hinges

Plastic hinges are modelled as fiber cross sections. They are modelled by automatic selection of fiber division in the cross section of elements [10] for seismic analysis. For robustness analysis, column hinges are modelled using automatic discretization option, while beam plastic hinges are modelled "manually" (fiber discretization is shown in Fig. 2, left). Among many expressions for the calculation of plastic hinge length [17] and because of the inconsistency among the values obtained by different expressions, the equations suggested by [18] and [19] are the most practical for the modelling and the analysis. It is estimated that the lengths of plastic hinges, calculated according to [18] and [19] correspond approximately to the relative lengths of columns and beams of 0.1*L*, where *L* is the length of the element. Therefore, the locations of the hinges are assigned as 0.05*L* and 0.95*L* to columns and beams in [10].

4 Non-linear analysis results and calculation of fragility curves

4.1 Seismic analysis

The results of NSA for mass-proportional (PROP) and modal (MOD) load distributions are shown in Fig. 5. Modal pushover curve was chosen as a referent curve for the calculation of fragility curves, according to [9].

To calculate damage state (DS) threshold values, it was necessary to do a bilinear approximation of NSA pushover curve, using Equivalent Energy Elastic-Plastic (EEEP) method and determine yielding (Sd_Y,Sa_Y) and ultimate capacity (Sd_U,Sa_U) points on capacity (spectral displacement – spectral acceleration) curve of SDOF system for both M1 and M2 (Fig. 5).

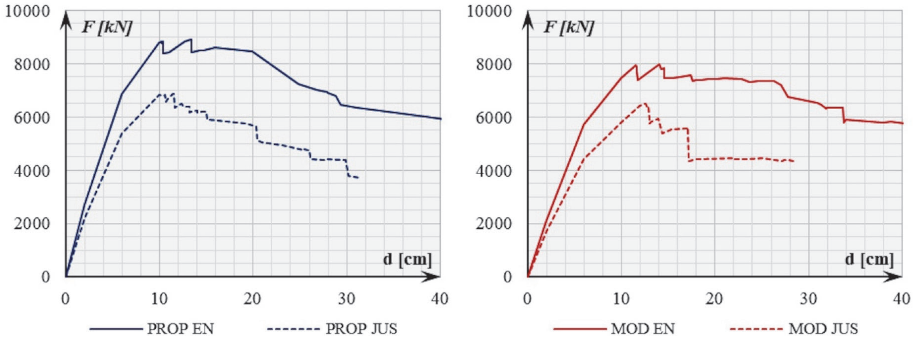

Figure 4. Pushover curves for mass-proportional (left) and modal (right) load distribution

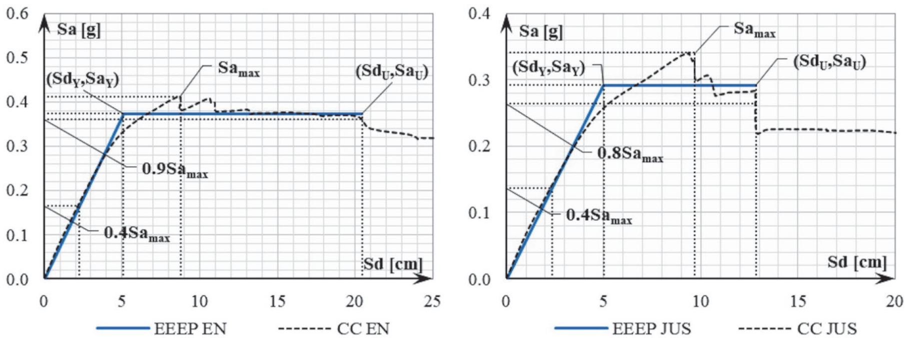

Figure 5. Capacity curves and their bilinear approximation for M1 (left) and M2 (right)

4.2 Robustness analysis

The results of nonlinear static and dynamic pushdown analyses are shown in Fig. 6. The main difference between the pushdown curves obtained using NSA and NDA is the lack of the visual insight into the stages of the beam, transient and catenary phase in NDA. Robustness analysis results still have similar form and their main difference is in the transient (plastic) phase, between flexural action in elastic and early plastic zone (beam phase) and catenary phase.

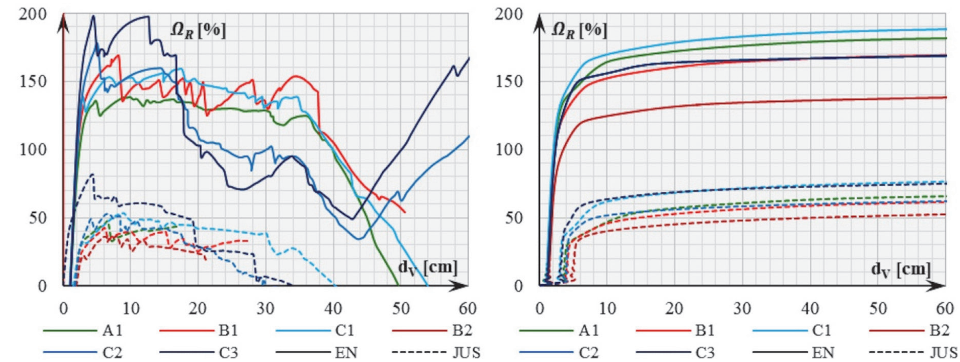

Figure 6. Pushdown curves obtained using NSA (left) and NDA (right) methods

4.3 Damage state performance points

Damage of a structural system may be quantified through threshold performance points (small damage – SD; moderate damage – MD; extensive damage – ED; complete damage – CD), which are determined according to [9] for seismic fragility analysis, where $\mu_{DS} = Sd^{DS}$ and $\mu_U = Sd_U/Sd_Y$ (Table 2) and according to [20] for robustness analysis (Fig. 7).

Table 2. Seismic structural DS threshold values, according to [9]

DS	SD [EN / JUS]		MD [EN / JUS]		ED [EN / JUS]		CD [EN / JUS]	
μ_{DS}	$0.7 \cdot Sd_Y$		Sd_Y		$Sd_Y + 0.25 \cdot (Sd_U - Sd_Y)$		Sd_U	
$\sigma_{LN,DS}$	$0.25 + 0.07 \cdot \ln(\mu_U)$		$0.2 + 0.18 \cdot \ln(\mu_U)$		$0.1 + 0.4 \cdot \ln(\mu_U)$		$0.15 + 0.5 \cdot \ln(\mu_U)$	
μ_{DS}^{Sd} [cm]	3.56	3.50	5.08	5.00	8.92	6.96	20.43	12.83
$\sigma_{LN,DS}$ [cm]	0.347	0.316	0.450	0.369	0.657	0.477	0.846	0.621
μ_{DS}^{PGA} [g]	0.109	0.098	0.154	0.137	0.264	0.185	0.592	0.319
$\sigma_{LN,DS}$ [g]	0.331	0.298	0.430	0.339	0.634	0.432	0.823	0.558

To quantify and compare the results of the column removal scenarios, from the perspective of the progressive collapse risk, methods proposed by [20] for the determination of damage LS, based on NDA are used. Limit states in this paper are defined in a following way:

- ***LS1*** (minor damage): LS1 occurs either in the first step, when reaching the reinforcement creep limit $\left(\varepsilon_{sy}^{EN} = \varepsilon_{sy}^{JUS} = 0.23\%\right)$ or the stress limit of concrete with maximum strength in the protective layer of concrete $\left(\varepsilon_{c,1}^{EN} = \varepsilon_{c,1}^{JUS} = 2.16‰\right)$.
- ***LS2*** (moderate damage): Occurs whenthe vertical displacement, obtained as the ratio of displacement of the top above the removed column and the length of the beam span, exceeds the determined threshold d_Y = 1.0 %.

- ***LS3*** (significant damage): This level of damage is assumed to occur when reaching the stress limit in the protective layer of concrete $\left(\varepsilon_{c,u}^{EN}=\varepsilon_{c,u}^{JUS}=3.5‰\right)$ or the maximum stress of the confined concrete core $\left(\varepsilon_{cc,1}^{EN}=2.56‰\,;\varepsilon_{cc,1}^{JUS}=2.38‰\right)$.
- ***LS4*** (severe damage): Occurs in the first step, when the ultimate stress is reached in the confined concrete core $\left(\varepsilon_{cc,u}^{EN}=11.56‰\,;\varepsilon_{cc,u}^{JUS}=10.16‰\right)$.
- ***LS5*** (progressive collapse): It is determined as the state at the dilatation value in steel at which tensile fracture in the longitudinal reinforcement bar occurs $\left(\varepsilon_{su}^{EN}=\varepsilon_{su}^{JUS}=7.5\%\right)$.

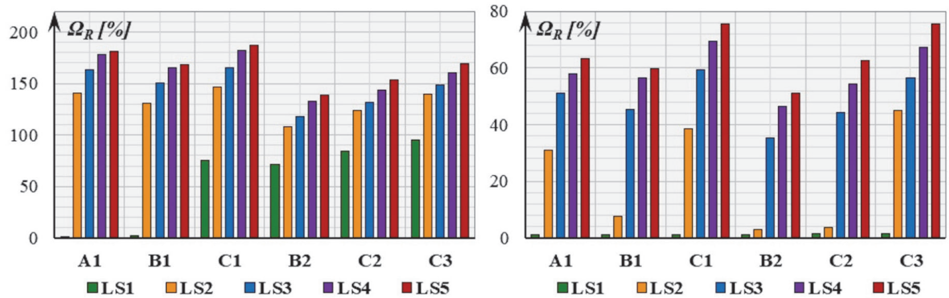

Figure 7. LS values for model M1 (left) and model M2 (right) obtained using NDA

4.3 Statistical analysis of the results

It is generally assumed that fragility curve is a lognormal distribution function, which means that "If a variable is log-normally distributed, its natural logarithm is normally distributed. Which means it must take on a positive real value, and the probability of it being zero or negative is zero." [1] For seismic fragility analysis, lognormal distribution is adopted [9]. For robustness fragility analysis, using *Kolmogorov-Smirnov* and *Anderson-Darling* tests in [21] on the results obtained through NDA method, it is established that for each LS distribution, the values of fit both normal and lognormal distribution, but they have a better fit with normal distribution and it was adopted for the robustness fragility curves calculation.

4.4 Calculation of fragility curves

In case of the calculation of seismic fragility curves, using spectral displacement (*Sd*) as a referent IM value for the DS threshold [9], the fragility functions are calculated as analytical cumulative distribution functions (CDF) for lognormal (LN) distribution:

$$P_{DS_i|IM}\left(IM_j,\mu_{LN|DS_i}^{IM},\sigma_{LN|DS_i}^{IM}\right)=\Phi\left(\frac{\ln IM-\mu_{LN|DS_i}^{IM}}{\sigma_{LN|DS_i}^{IM}}\right) \tag{3}$$

where Φ is the cumulative distribution function of the standard normal distribution, and are the mean and standard deviation of LN distribution values shown in Table 2. However, because it is possible to determine the relation between *Sd* and PGA, it is possible to present the fragility curves with the PGA as the IM, using the Maximum Likelihood Estimation (MLE) method [1, 22] (Fig. 8, left).

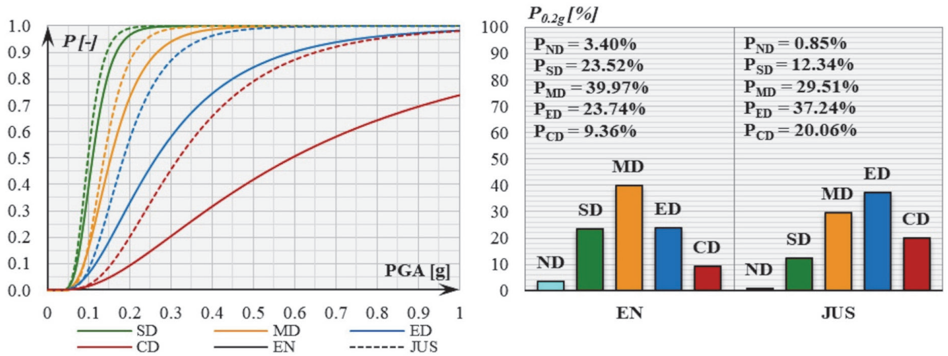

Figure 8. Seismic fragility curves (left) and DS probabilities for design PGA (right)

Probability density functions for the occurrence of different states of damage for the design PGA = 0.2*g* (Fig. 8, right) are calculated using the equations [1, 22]:

$$
\begin{aligned}
P_{DS_0} &= 1 - P_{DS_1}\left[IM_j, \mu_{LN|DS_1}, \sigma_{LN|DS_1}\right] \\
P_{DS_i} &= P_{DS_i}\left[IM_j, \mu_{LN|DS_i}, \sigma_{LN|DS_i}\right] - P_{DS_{i+1}}\left[IM_j, \mu_{LN|DS_{i+1}}, \sigma_{LN|DS_{i+1}}\right] \\
P_{DS_n} &= P_{DS_n}\left[IM_j, \mu_{LN|DS_n}, \sigma_{LN|DS_n}\right]
\end{aligned}
\tag{4}
$$

where is a probability of no damage to occur and and . is an index of a particular DS, and is an index of a particular IM (PGA). is a total number of damage states.
In case of the calculation of robustness fragility curves, using as a referent value for the LS threshold [21], the fragility function is calculated as analytical cumulative distribution function (CDF) for normal distribution:

$$
P_{LS_i|\Omega_{R,i}}\left(\Omega_{R,i}, \mu_{LSi}^{\Omega_R}, \sigma_{LSi}^{\Omega_R}\right) = \Phi\left(\frac{\Omega_{R,i} - \mu_{LSi}^{\Omega_R}}{\sigma_{LSi}^{\Omega_R}}\right) \tag{5}
$$

where Φ is the cumulative distribution function of the standard normal distribution, and are the mean and standard deviation of normal distribution values shown in Fig. 9.

Table 3. Robustness fragility parameters (mean and standard deviation)

Ω_R [%]	$\mu_{LSi}^{\Omega_R}$ **[EN / JUS]**		$\sigma_{LSi,unc}^{\Omega_R}$ **[EN / JUS]**		$\sigma_{LSi,corr}^{\Omega_R}$ **[EN / JUS]**	
LS1	55.033	1.433	41.949	0.151	25.600	12.905
LS2	131.700	21.533	13.876	18.922		
LS3	146.233	48.700	18.409	8.790		
LS4	160.467	58.733	19.181	8.539		
LS5	166.300	64.667	17.903	9.454		

To avoid the overlapping of the fragility functions, their correction is performed by adopting the same standard deviation value for all LS, using the MLE method described in [1, 22]. Another method to avoid the overlapping of the fragility functions is suggested and discussed in [13]. and for uncorrected and corrected fragility curves are shown in Table 3 and robustness fragility curves are displayed in Fig. 9.

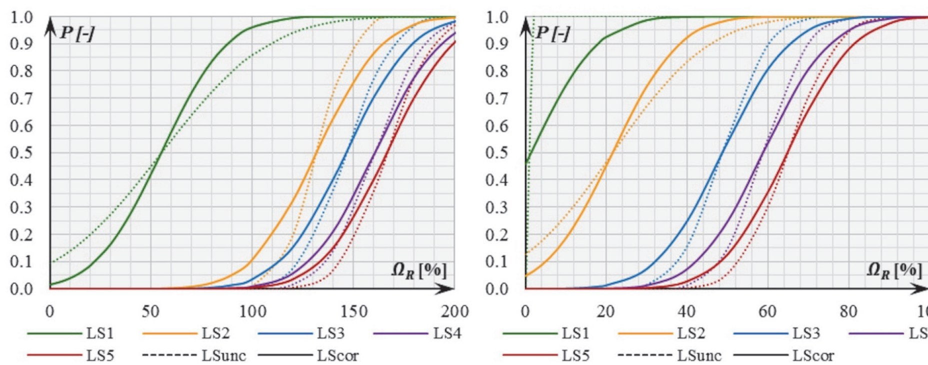

Figure 9. Robustness fragility curves for model M1 (left) and model M2 (right)

5 Discussion of the results

The results show the difference between the structural response of models M1 and M2, but both models have good response for design PGA. Seismic NSA pushover and fragility curves for M1 show a better structural response and greater seismic resilience of the system, than for M2 (Fig. 4 and Fig. 8, left). MD occurrence probability is the highest for M1, and ED occurrence probability is the highest for M2, which means that M1 is a bit more on the safety side, from the aspect of the structural design (Fig. 8, right).

From the aspect of the capacity of building to resist progressive collapse, M1 have much better structural response than M2 (Fig. 6, Fig. 7 and Fig. 9). For the each column removal scenario in M2, the structure will reach LS1 almost at the same time when the column is removed, unlike in M1 where it will happen only in cases A1 and B1 (Fig. 7). The range between the lowest and the highest value of for the ultimate capacity limit state (LS5) is 138.4 % - 187.4 % for M1 and 51.2 % - 75.6 % for M2 (Fig. 7), which means that M1 response to resist progressive collapse is much more on the safety side, than the M2 (Fig. 9).

6 Conclusions

Since buildings cannot be designed for every hazard to which the structural system may be exposed during its lifetime, a general design approach should take into account the action associated with low probability events and huge consequences for the structural system, which is characteristic of progressive collapse [23]. It is very important to provide a sufficient nonlinear capacity of the system for its seismic response, but as well as in order to provide necessary force redistribution in case of the loss of one or more elements.

In this paper, the comparative analysis of two models of RC frame structure, designed using set of Eurocodes [4, 5, 6, 7] and JUS [8] is performed from the aspect of seismic and robustness fragility. The system fragility curves were derived from the results of NSA (seismic analysis) and NDA (robustness analysis), using statistical methods. In seismic analysis, probabilities of the occurrence of each DS are determined and compared, and in robustness analysis, the LS values are compared and discussed.

Based on the results of the analysis, it can be concluded that structural response of the structure designed using the recommendations given in structural Eurocodes [4, 5, 6, 7] (M1) will have better seismic response and better response to resist progressive collapse as well, than the structure designed using the recommendations given in Yugoslav regulations from 1981 (JUS) [8] (M2). Recent implementation of Eurocodes in some of the countries of ex-Yugoslavia (ex-Yu) and future, planned implementation in the ones that still use the old regulations in the design of RC buildings (such as modified versions of JUS [8]), should lead to better performance of the new buildings subjected to the seismic excitation or the incidental situation which can lead to progressive collapse. When it comes to the analysis of the existing structures in ex-Yu Republics, designed using the old codes, several researches were conducted [24, 25, 26, 27] but after recent developments and devastating consequences of the earthquake in Petrinja, Croatia, there is a definite necessity to investigate the risks of the damage, caused by strong earthquakes, on macro and micro levels in both urban and rural areas of the ex-Yu and wider Balkan region.

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