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APPLICATION OF THE PARAMETRIC PUSHOVER CURVE MODEL FOR SEISMIC LOSS ESTIMATION OF A BUILDING STOCK

Neja Fazarinc (1), Anže Babič (2), Matjaž Dolšek (3)

(1) *Young researcher, Faculty of Civil and Geodetic Engineering University of Ljubljana, neja.fazarinc@fgg.uni-lj.si*
(2) *Assistant Professor, Faculty of Civil and Geodetic Engineering University of Ljubljana, anze.babic@fgg.uni-lj.si*
(3) *Professor, Faculty of Civil and Geodetic Engineering University of Ljubljana, matjaz.dolsek@fgg.uni-lj.si*

Abstract

An effective seismic risk assessment of building stock is crucial for informing stakeholders about potential issues in the case of strong earthquakes and enhancing community resilience. Recently, a parametric pushover curve model was introduced, enabling the estimation of building-specific pushover curves and fragility functions even with a low level of knowledge about the building. The model estimates a trilinear pushover curve using twelve parameters assessed for reinforced concrete and masonry buildings in Slovenia, considering limited knowledge about the structures, and can also be used to simulate the pushover curves of an equivalent new building stock. This paper provides an overview of the parametric pushover curve model and its application to the preliminary seismic stress test of the University of Ljubljana building stock. The findings indicate that the seismic capacity of the existing stock, in terms of limit-state peak ground acceleration, is only half of what is expected for new buildings, highlighting significant seismic risk and the urgent need for systematic improvements in seismic safety. Such an underestimation of structural capacity is reflected in the losses. We have shown that the EAL of the existing building stock exceeds the EAL of the equivalent building stock by a factor of 9 and the EAF by a factor of 52. Based on the preliminary seismic stress test results, the University of Ljubljana decided to conduct a second-level seismic stress test, which requires more detailed building data. This ongoing project aims to provide a comprehensive understanding of the seismic resilience of the University of Ljubljana and serves as a foundation for developing a strategic plan to enhance seismic safety.

Keywords: Parametric pushover curve model · Building stock · Seismic performance assessment · Reinforced concrete buildings · Masonry building · Loss estimation

1. Introduction

Seismic risk assessment of a building stock is a complex process, characterized by uncertainties at every stage of analysis (e.g., [1-3]). It begins with establishing both the exposure model and the seismic hazard model. Next, a framework for defining building damage states must be developed, followed by an assessment of the seismic demand model, which can be used to assess the seismic capacity in terms of seismic intensity measures that represent a basis for the building-specific fragility functions. By integrating seismic fragility functions with location-specific seismic hazard data and an appropriate damage-to-loss model, loss estimation can be conducted while considering various seismic risk metrics.

The exposure model includes essential building data used in the analysis, where the level of detail and associated uncertainties depend on the target knowledge level about the buildings and available resources. When assessing larger building stocks, much of the required data can be sourced from publicly available databases or obtained directly from building stock owners or managers. If such data is insufficient, it can be supplemented using various methods, including building inspections, surveys, building stock inventories, or expert judgment. Consequently, buildings with similar characteristics are typically categorized into classes (e.g. [4-6]), allowing for the application of predefined fragility curves ([1, 7-9]), which significantly simplifies the analysis.

In the present study, an alternative approach was implemented using a recently developed parametric pushover curve model [10], which is briefly introduced in Section 2. This model makes it possible to assess building-specific pushover curves based on their fundamental characteristics. The model's

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practical application is demonstrated through the seismic loss estimation of the University of Ljubljana's building stock (Section 3), considering both the existing structures and an equivalent new building stock.

2. Overview of the parametric pushover curve model and its application for Slovenian buildings

The parametric pushover curve (PPC) model [10] allows for the modeling of buildings on a building-by-building basis with minimal knowledge of their characteristics, relying on a set of twelve basic structural parameters. These parameters can be estimated or measured directly, derived from building's design documentation, or determined using additional empirical and theoretical models that account for the construction standards in effect at the time of building. When only limited data is available—a common scenario when utilizing existing databases (e.g., [13])—simplified models can be introduced to estimate the necessary parameters for defining the PPC of a building. The description in the following is directed towards the application to Slovenian buildings and taking into account only a low level of knowledge about the buildings, with particular parameters and models drawn from Slovenian construction practices, highlighting the evolution of building standards and regulations that have been in force in the past. A more general description of the proposed model is available elsewhere [10].

The PPC model represents an idealized tri-linear pushover curve, characterized by at least three key points (e.g., Y – the yield point, M – the capping point, and U – the near-collapse point) or five independent coordinates, determined indirectly. These parameters sufficiently describe the relationship between the displacement at the top of the building and the seismic force (i.e., the force-displacement (F-D) relationship), as illustrated in Fig. 1.

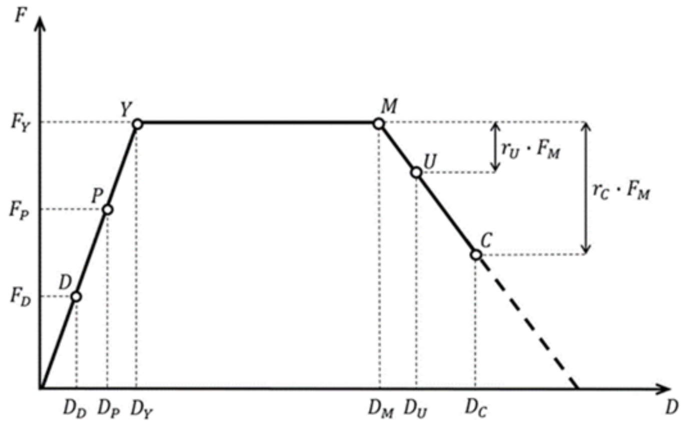

Figure1: The base shear-top displacement relationship of the parametric pushover curve model ([10]).

The F-D relationship is assumed linear until the yield point, Y, where the building reaches its maximum base shear, F_Y, which is defined as the maximum of (i) the value based on the design base shear, F_D, and (ii) the minimum base shear, $F_{Y,min}$, assessed as the capacity of the critical storey of the building, taking into account the possibility that some buildings were not designed for earthquake resistance.

F_D is determined as:

$$F_D = BS_c \cdot \sum m_i \cdot g, \tag{1}$$

where m_i is the mass of the i-th storey of the building, which also represents the i-ih component of the building mass vector $\boldsymbol{m}$. It is calculated as a product of the gross area of the storey A_G and the building typology-specific average mass per unit area m_A, taken as $1.2\,\frac{t}{m^2}$ for RC buildings and $1\,\frac{t}{m^2}$ for masonry buildings [1,14]. In Eq. (1), g is the gravitational acceleration and BS_C is the design base shear coefficient, modelled by considering the development of the regulations for earthquake-resistant design of buildings from past building codes [15, 16], the building location and the construction year CY:

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$$BS_C = \begin{cases} 0{,}01;\ CY \leq 1948 \\ 0{,}011 - 0{,}02;\ 1948 < CY \leq 1963 \\ \begin{cases} 0{,}02 - 0{,}08 \cdot \gamma_{I,1};\ A \\ 0{,}025 - 0{,}1 \cdot \gamma_{I,1};\ B, C \\ 0{,}03 - 0{,}12 \cdot \gamma_{I,1};\ D, E \end{cases};\quad 1963 < CY \leq 1981 \\ K = K_0 \cdot K_s \cdot K_d \cdot K_p;\ 1981 < CY \leq 2008 \\ S_{ad}(PGA_{475}, S, T, T_B, T_C, T_D, q) \cdot \lambda \cdot \gamma_I;\ 2008 < CY. \end{cases} \tag{2}$$

Most of the parameters from Eq. (2) adhere to the standards that were in force during specific periods in the past. Between 1948 and 1981, the values of BS_C are given as intervals because the value for a specific building depends on its location relative to the seismic zones or seismic hazard maps from the relevant construction periods. The letters $A - E$ for the construction period from 1963 to 1981 refer to the soil type at the location of the building, while $\gamma_{I,1}$ and γ_I are the building's importance factors. The parameter K represents the seismic factor, consisting of four components, S_{ad} is the design acceleration at the fundamental period of the building as defined in Eurocode 8 [17], and λ is the proportion of the building's mass considered in the lateral force analysis. The values of these parameters are detailed elsewhere [10, 18].

Considering the low level of knowledge about the buildings, the fundamental period T is estimated using simplified models, taking into account the number of storeys n_e and the predominant structural material. For masonry buildings, the POTROG model [19-21] is adopted. This model is based on measured values of the buildings' fundamental period, so the calculated values are further multiplied by $\sqrt{2}$ to account for the cracking of the elements during the seismic action, while for RC buildings, the HAZUS model [9] for the assumed dual system is adopted.

The minimum base shear $F_{Y,min}$ is modelled as construction period dependent, taking into account the predominant structural material and the assumed critical storey. The load-bearing capacity of RC buildings is determined using a simplified model, grounded in the Japanese standard for seismic evaluation of existing buildings [22] and considering the findings of Sinkovič et al. [23]. The model is based on the horizontal cross-sectional area of columns and walls in the critical storey, their shear strength, τ_j, and the total weight of the building, W. For masonry buildings, the shear failure mechanism is assumed, with the occurrence of the characteristic inclined cracking of the walls, which occurs when the primary tensile stresses exceed the tensile strength of the masonry, f_t [24, 25]. In addition to f_t, the essential parameters for this model are the cross-sectional area of a typical wall oriented in the direction of the earthquake ground motion, A_W, the factor accounting for the uniformity of shear stress distribution across the section, b, and the compressive stress in the walls, σ_0, determined by the weight of the building.

The resulting maximum base shear of the building is:

$$F_Y = \max\{q_R \cdot q_S \cdot F_D; F_{Y,min}\}, \tag{3}$$

where q_R is the component of the overstrength factor, accounting for the structural redundancy and the consequent possibility of the redistribution of the seismic action effects, and q_S is the component of the overstrength factor accounting for all the remaining sources of overstrength (e.g., minimum requirements, material safety factors, etc.). No additional hardening of the building is assumed post-yielding, leading to a constant plastic branch of the PPC. Thus, the base shear at the capping point M, is:

$$F_M = F_Y. \tag{4}$$

The force associated with the third essential characteristic point in the F-D relationship definition corresponds to the near-collapse limit state (point U in Fig. 1) and is defined as a ratio of the maximum base shear. A reduction in load-bearing capacity of 20 % is assumed, modelled by setting the value of r_U to 0.2 (Fig. 1), which results in:

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$$F_U = (1 - r_U) \cdot F_Y = 0.8F_Y. \tag{5}$$

The displacements, corresponding to the three characteristic points, Y, M and U, are defined gradually. To determine the yield displacement, D_Y, an equivalent SDOF system is introduced, assuming it has the same fundamental period as the entire building and a linear F-D relationship until the yield point. Given these assumptions and the conventional relationship between the fundamental period, mass and stiffness of an SDOF system, the displacements corresponding to the design base shear and the yield point, i.e. D_D in D_Y, were determined. The displacement D_U, which is associated with the softening branch of the PPC, determined taking into account the assumed deformation shape of the building at the near collapse limit state, which is reflected in the storey-drift-angle uniformity coefficient C_U and the corresponding storey drift angle of the most deformed storey θ_U:

$$D_U = C_U \cdot \theta_U \cdot \sum_{i=1}^{n_e} h_{e,i}. \tag{6}$$

In Eq. (6), $h_{e,i}$ represents the height of the i-th storey. The storey heights, comprising the storey-height vector $\boldsymbol{h_e}$, are summed up to account for the entire height of the building. Due to the low level of knowledge about the buildings, the term $\sum_{i=1}^{n_e} h_{e,i}$ is replaced by $n_e \cdot h_e$, because all of the storeys are assumed to be of the same height. Parameter C_U can take a value between $\frac{1}{n_e}$, which defines the extreme soft storey effect, and 1 in the case of uniform inter-storey drifts. For this study, it is taken as $\frac{1.2}{n_e}$ for masonry buildings and 1 for RC buildings. Parameter θ_U is modelled considering the predominant structural material and the construction period of the building. The remaining two characteristic displacements, corresponding to the capping point, M, and the collapse point, C, were determined by assuming the linear post-capping branch of the PPC and adopting constant values of $r_U = 0.2$ and $r_C = 0.5$ (Fig. 1) regardless of the building type using Eqs. (7) and (8):

$$D_M = \frac{D_U}{\mu_{0M} - (1 - r_U) \cdot \mu_{0M} + (1 - r_U)} \tag{7}$$

$$D_C = D_M \cdot (1 + \mu_{0M} r_C - r_C). \tag{8}$$

In Eqs. (7) and (8), μ_{0M} represents the post-capping ductility, which defines the ratio between the displacement at the theoretical zero base shear, where the dashed line in Fig. 1 would cross the horizontal axis, and the displacement at the capping point, D_M. This ratio is assumed 1.2 and 3.5 for masonry and RC buildings, respectively.

The presented PPC model is thus defined by twelve structurally dependent parameters, i.e. BS_C, $\boldsymbol{m}$, q_S, q_R, $F_{Y,min}$, $\boldsymbol{\phi}$, μ_{0M}, C_U, θ_U, n_e, $\boldsymbol{h_e}$ and T. The majority of them have been referenced in the preceding part of this section, with the exception of $\boldsymbol{\phi}$, which is the mode shape vector for determining the transformation factor for the introduction of the equivalent SDOF model and taken as an inverted triangle. It should also be noted that some of the models and the values for this set of parameters are region- and time-specific, taking into account the time variation in the local construction practice or based on expert judgement, which should be considered when applying PPC for building stocks elsewhere. More details about the essential models and the values of the corresponding parameters can be found elsewhere (e.g. [10]), as well as the impact of the uncertainties arising from the low level of knowledge about the buildings on the resulting loss estimation ([11-12]).

3. Loss estimation of an existing building stock relative to a new building stock

3.1. Description of the existing building stock and a new building stock scenario

The presented PPC model was applied in the preliminary seismic stress test of a University of Ljubljana's (UL) building stock, consisting of 109 buildings (of which 58 are masonry and 51 RC), with a cumulative net area of over 300,000 m^2, which represents more than 95 % of the entire building stock of the UL. Most of those buildings are located in the capital city of Slovenia, Ljubljana.

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The analysis of the building stock revealed that it is quite outdated, with more than half of the buildings being constructed before 1964, prior to the introduction of Slovenia's first earthquake-resistant building code. Out of 55 buildings constructed in that era, 44 are masonry (referred to as MSN), while 11 are reinforced concrete (RC). Only 6 of the studied buildings, all of them RC, were constructed after 2008, when using Eurocode 8 became obligatory in Slovenia.

The 58 masonry buildings represent about 30% of the building stock area, with the majority built before 1964 and ranging from one to six storeys. The remaining 70% consists of RC buildings, mainly constructed between 1964 and 1981 and ranging from two to 14 storeys. The exposure model, i.e. the building stock, categorized by the predominant structural material, number of storeys, and construction period is presented in Tables 1 and 2, where n represents the number of buildings in each category, N is the nominal number of people in these buildings, and A is the net building area of the category expressed as a percentage of the total building stock area. A more detailed description of the development of the exposure model and the studied building stock can be found elsewhere [10- 12].

Table 1: Number of MSN buildings, the nominal occupation of those buildings, and their net area as % of the total area of the building stock categorized by number of storeys and construction period.

MSN	**≤ 1963**			**1964-1981**			**1982-2007**			**∑**		
n_e	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**
1 - 3	31	3865	11.8	7	198	0.8	7	400	1.2	45	4463	13.8
4 - 6	13	5271	15.7	/	/	/	/	/	/	13	5271	15.7
∑	**44**	**9136**	**27.5**	**7**	**198**	**0.8**	**7**	**400**	**1.2**	**58**	**9734**	**29.5**

Table 2: Number of RC buildings, the nominal occupation of those buildings, and their net area as % of the total area of the building stock categorized by number of storeys and construction period.

RC	**≤ 1963**			**1964-1981**			**1982-2007**			**≥ 2008**			**∑**		
n_e	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**	**n**	**N**	**A [%]**
1 - 3	4	1131	4.1	6	4070	5.3	12	4063	8.8	3	277	1.4	25	9541	19.4
4 - 6	5	5506	8.3	6	7979	10.9	3	3039	6.5	3	917	11.2	17	19730	36.9
≥7	2	503	2.5	6	3647	11.4	1	156	0.4	/	/	/	9	4306	14.3
∑	**11**	**7140**	**14.9**	**18**	**15696**	**27.6**	**16**	**7258**	**15.7**	**6**	**1194**	**12.6**	**51**	**31288**	**70.8**

To facilitate the comparison of seismic performance and the relative evaluation of estimated losses, an equivalent new building stock was also modeled using the PPC. In this hypothetical scenario, the entire building stock would be replaced with equivalent new buildings while maintaining the same geometry and location. However, the new buildings would be designed in compliance with Eurocode 8, with masonry buildings of three or more stories replaced by reinforced concrete (RC) buildings, while those with one or two stories would remain masonry.

This approach aligns with current construction practices in Slovenia, as taller masonry buildings are generally not considered suitable for seismically active areas. Consequently, the equivalent new building stock was modeled using mostly the same input parameters as the existing building stock, except for the CY-dependent parameters, which were updated to comply with Eurocode 8. In some cases, additional modifications were made due to the change in the predominant structural material from masonry to RC.

3.2. Parametric pushover curves

First, the parametric pushover curves of existing and the equivalent new buildings were determined and are presented in Fig. 2 in a normalized manner. The maximum base shear was normalized by the total weight of the building, while the top displacement was normalized with respect to the height of the building. Thus, Fig. 2 represents the F_b/W-D/H relationship for each building and its equivalent new counterpart. The first column of the figure corresponds to the masonry buildings that remain masonry,

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the second column to the masonry buildings, for which the predominant structural material changes in the equivalent new building stock scenario, and the third column to RC buildings. Each row represents a different construction period, i.e. before 1963, from 1964 to 1981 and from 1981 to 2007. The PPCs of existing masonry buildings are depicted in red and those of the equivalent new masonry buildings in orange. The PPCs of existing RC buildings are black, while for the equivalent new RC buildings, they are grey. For buildings, constructed after 2008, such a comparison would be irrelevant, as the PPCs of existing buildings and equivalent new buildings would be the same, and relatively similar to the ones of the equivalent new RC buildings (e. g. in the second and third columns of Fig. 2).

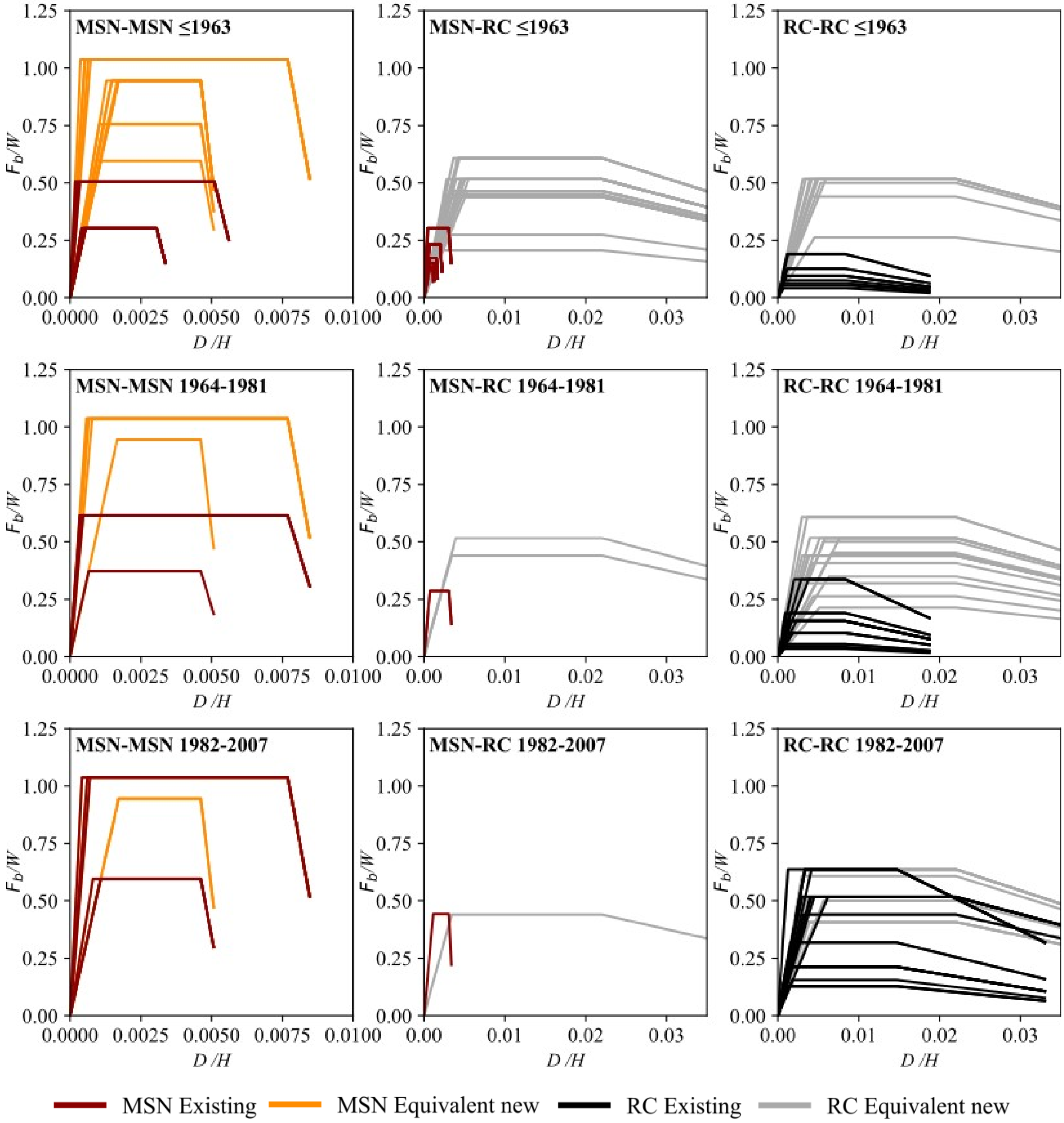

Figure 2: Normalized parametric pushover curves of existing and equivalent new buildings.

Fig. 2 shows that the oldest masonry buildings have the lowest deformation capacity out of all building categories, as well as the lowest normalized load-bearing capacity out of all masonry buildings. For RC buildings, the trend of increasing deformation capacity with respect to construction periods is even more evident, while for the low-bearing capacity, it is less apparent. Nevertheless, the deformation capacity of the RC buildings is significantly larger than that of the masonry buildings regardless of the

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construction period (note also the difference in the limits of the horizontal axis between the first and the remaining two columns of Fig. 2), while the normalized load bearing capacity is mostly larger for masonry buildings. The equivalent new buildings mostly have a larger load bearing as well as deformation capacity than the existing buildings. This is especially significant when existing masonry buildings are replaced with equivalent new RC buildings (second column of Fig. 2).

3.3. Loss estimation

The loss estimation was performed using a conventional seismic risk assessment (e.g. [1]) to estimate two seismic risk measures, i. e. the expected annual loss (EAL) and the expected annual fatalities (EAF). The EAL was determined by taking into account the expected repair costs related to four damage states of the building, DS_1 to DS_4, as:

$$EAL = \sum_{i=1}^{4} c_i C_R \sum_{j=1}^{n} P\left(DS_i \middle| IM = PGA_{j,c}\right) \left(\lambda\left(PGA_{j,a}\right) - \lambda\left(PGA_{j,b}\right)\right) \tag{9}$$

where C_R is the cost of the replacement of the building and c_i defines the relative cost of the repair in the case of the achieved i-th damage state. The latter was modelled in accordance with Dolšek et al. [1] as 0.02, 0.1, 0.4 and 1 for the limit states DS_1 to DS_4. The term $\lambda(PGA)$ represents the mean annual frequency of ground motions with the corresponding intensity of at least PGA, evaluated from the site and soil type-specific seismic hazard curve, while $PGA_{j,a}$, $PGA_{j,b}$ and $PGA_{j,c}$ represent the values of the PGA at the beginning, end and center of each interval, obtained by discretizing the seismic risk integral. The term $P\left(DS_i \middle| IM = PGA_{j,c}\right)$, evaluated using fragility functions, represents the probability that the building reaches damage state DS_i given the ground motion with the maximum peak ground acceleration of $PGA_{j,c}$. The fragility functions were modelled conventionally, using the standardized lognormal distribution, defined by the median ground motion intensity measure, in this case the PGA at a given damage state, i.e. PGA_{DSi}, and the corresponding standard deviation of the logarithmic values of the ground motion intensity, $\beta_{PGA,DSi}$. The values for PGA_{DSi} were determined based on the displacements, corresponding to the four damage states, D_{DSi}, using the N2 method [26]. The displacements D_{DSi} were modelled relative to the characteristic displacements as defined by Fazarinc and Dolšek [12].

The EAF was estimated analogously to the EAL:

$$EAF = E(N)\lambda_{d,C} P(C|DS_4) \sum_{j=1}^{n} P\left(DS_4 \middle| IM = PGA_{j,c}\right)\left(\lambda\left(PGA_{j,a}\right) - \lambda\left(PGA_{j,b}\right)\right), \tag{10}$$

where $E(N)$ represents the equivalent annual number of people in the building, estimated using a model that accounts for fluctuations in building occupancy on a yearly and hourly basis [27], and $\lambda_{d,C}$ represents the proportion of people in the building who would lose their lives if the building collapses, assumed to be 0.1 regardless of the building's structural system [28]. It was also assumed that the building could only collapse upon reaching the limit state DS_4, with the probability of collapse, $P(C|DS_4)$, ranging from 0.05 to 0.15, depending on the predominant structural material and the number of storeys [9].

The losses in terms of EAL and EAF were first estimated for the entire existing building stock. To get a better insight into the contribution of different types of buildings, the results were also analyzed for each predominant structural material and construction period separately and are presented in Table 3. Both EAL in thousands of EUR and EAF are first shown in absolute terms for each category and then in relative terms as a percentage of the losses, estimated for the entire building stock. The analysis revealed that the EAL for the entire existing building stock of Ljubljana is more than 2 million EUR, while EAF is almost 3.9. The masonry buildings contribute more than 60 % of the EAL and more than 90 % to the EAF, although they represent only approximately 30 % of the building stock's net area. Almost all of those losses can be attributed to older masonry buildings, constructed before 1963, which can be almost directly attributed to their low deformation capacity. On the other hand, the EAF for RC buildings is much smaller, contributing only 10 % to the expected risk for the entire building stock. For the EAL, the contribution of RC buildings is almost 40 %, but this percentage decreases if the losses are

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normalized with respect to the net area of buildings. The highest losses for RC buildings are expected for the buildings, constructed between 1964 and 1981 (424,000 EUR), followed by the oldest RC buildings (260,000 EUR). The absolute and relative values for both loss measures for different building categories are presented in more detail in Table 3.

Table 3: Loss measures, i.e. *EAL* and *EAF*, for the existing building stock and different building categories.

		EAL		***EAF***	
MATERIAL	**CY**	**[1000 EUR]**	**[%]**		**[%]**
MSN	**≤ 1963**	1247	60	3.45E-01	89
	1964-1981	11	1	2.02E-03	1
	1982-2007	5	0	1.04E-03	0
	SUM	**1263**	**61**	**3.48E-01**	**90**
RC	**≤ 1963**	260	13	1.38E-02	4
	1964-1981	424	21	2.29E-02	6
	1982-2007	81	4	2.31E-03	1
	≥ 2008	39	2	6.06E-04	0
	SUM	**804**	**39**	**3.96E-02**	**10**
ALL	**SUM**	**2067**	**100**	**3.88E-01**	**100**

To analyze the potential for improvement and risk reduction, the loss measures were then compared on a building-by-building level for the equivalent new and the existing building stock. In addition to EAL and EAF, the third loss measure was added, i.e. the probability of the building reaching or exceeding damage state DS_4 in a given year, denoted as $P(DS_4)$. The values of the loss measures for equivalent new buildings were normalized with respect to the values for the existing buildings and expressed in %, obtaining $\overline{P(DS_4)}$, $\overline{EAL}$ and $\overline{EAF}$, as seen in Table 4. Here, the buildings were first categorised by the construction periods (the three main columns of Table 4) as well as the structural material of existing and the equivalent new buildings (the three main rows of Table 4, i.e. MSN-MSN, MSN-RC and RC-RC) and the number of storeys, n_e. For each loss measure and each such category, the mean value and the corresponding standard deviation were calculated. As expected, older buildings show a higher potential for improvement than newer ones. If older masonry buildings are replaced with masonry buildings, all of the three loss measures for new buildings reduce to about 20-25 % of the values for existing ones, whereas for newer buildings, constructed from 1964 to 1981 and 1982 to 2007 the reduction is less significant, to only 40-60 % and 80-90 %, respectively. For the latter category, i.e. MSN-MSN buildings, constructed from 1982 to 2007 the corresponding standard deviations for all three normalized loss measures are the highest out of all building categories at more than 20 %. As expected, the largest improvement can be observed, if the masonry buildings are replaced with RC buildings (i.e. MSN-RC), where $P(DS_4)$ and EAF reduce to only about 1% of the values for existing buildings, while for EAL, the reduction is less significant. This is the result of the fact that $P(DS_4)$ and EAF are closely related only to DS_4 and thus the deformation capacity, whereas for EAL, all four damage states are taken into account. For RC buildings, constructed before 1963 and from 1964 to 1981, there is a similar potential for improvement regardless of the number of storeys of those buildings. $P(DS_4)$ and the EAF would reduce to only 6-7% of the present value if those buildings were replaced with equivalent new ones, while EAL would reduce to 14-18% of the present values. For the RC buildings constructed from 1982 to 2007, all three loss measures would reduce to approximately 30% of the present value, which means that there is slightly less potential for improvement. Further detail about the relative comparison of the equivalent new and the existing buildings can be observed from Table 4.

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Table 3: The potential for seismic risk reduction, expressed by normalizing the values of seismic risk measures, $P(DS_4)$, EAL and EAF, of equivalent new buildings to the values of existing buildings.

		≤ 1963						1964-1981						1982-2007					
		$\overline{P(DS_4)}$ [%]		$\overline{EAL}$ [%]		$\overline{EAF}$ [%]		$\overline{P(DS_4)}$ [%]		$\overline{EAL}$ [%]		$\overline{EAF}$ [%]		$\overline{P(DS_4)}$ [%]		$\overline{EAL}$ [%]		$\overline{EAF}$ [%]	
MATERIAL	n_e	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
MSN-MSN	**1-3**	24	5	21	5	23	5	57	8	44	5	57	9	87	20	85	23	87	21
MSN-RC	**1-3**	1	1	8	3	1	1	1	0	10	0	1	0	1	0	14	0	1	0
	4-6	0	0	4	1	0	0	/	/	/	/	/	/	/	/	/	/	/	/
RC-RC	**1-3**	7	1	15	0	6	1	6	1	16	2	6	1	28	5	46	15	27	5
	4-6	7	1	14	1	6	1	7	1	18	3	7	1	28	1	29	0	28	1
	≥7	7	1	14	1	7	1	8	2	15	1	8	1	29	0	31	0	29	0

The losses of the existing building stock and those of the equivalent new building stock were also compared at the level of building categories, defined by the predominant structural material of existing buildings, construction period and number of storeys. As expected, the highest potential for improvement was observed for older masonry buildings, constructed before 1963, especially those with 4-6 storeys, which would be in hypothetical equivalent new building stock replaced with RC buildings. The EAL for those buildings could be 30 times lower and the EAF almost 530 times lower if they were replaced with equivalent RC buildings, designed according to Eurocode 8. Great potential for improvement was also recognized for the RC buildings constructed before 1981, where the EAL could be reduced by a factor of five and the EAF could be reduced by a factor of 12 to 16. It should be noted, that this improvement seems significant, but since the EAF for existing RC buildings is not as high as for masonry buildings (in absolute terms), the replacement of those buildings would not reduce the overall risk of the building stock as significantly as if replacing the masonry buildings. The overall seismic risk could be reduced almost by a factor of 9 in terms of EAL and almost by a factor of 52 in terms of EAF. This shows a significant potential for better protection of human lives in case of seismic events, which is one of the main goals of seismic-resistant design of buildings.

4. Conclusions

A recently introduced tri-linear parametric pushover curve model implemented for seismic risk assessment of buildings in Slovenia is briefly presented. The model is then demonstrated for the prediction of parametric pushover curves of the existing and equivalent new building stock of the University of Ljubljana, considering only limited knowledge about the buildings. This enabled the comparison between the existing buildings and their equivalent new counterparts designed according to Eurocode 8.

First, the normalized parametric pushover curves of existing and new buildings were analyzed, revealing a very low load-bearing and deformation capacity for buildings constructed in older construction periods. This indicates that the building stock of the University of Ljubljana needs to be strengthened regarding earthquake resistance.

The comparison between the existing and the equivalent new buildings revealed a substantial potential for seismic risk reduction. Replacing older masonry buildings with new RC buildings could reduce the EAL on a building level by a factor of 30, while EAF by a factor of 530, while for RC buildings, these reduction factors are 5 and 12 to 16, respectively. Overall, the seismic risk for the entire building stock could be reduced nine-fold in terms of EAL and 52-fold in terms of EAF.

These results highlight the importance of prioritizing the replacement of older, more vulnerable buildings to enhance the seismic resilience of the building stock, thereby improving user safety. The ongoing seismic stress test of the building stock of the University of Ljubljana, incorporating a higher level of knowledge about the buildings, will allow us to refine these assessments further and support

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the development of a strategic plan for improving the seismic safety of the University of Ljubljana. The authors believe the parametric pushover curve model can be used in Croatia with minor adjustments. The demonstrated approach can serve as a model for other institutions and communities aiming to mitigate seismic risk and protect human lives.

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