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ORIGINAL ARTICLE

Parametric pushover curve model for seismic performance assessment of building stock

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Abstract
An effective assessment of the seismic risk of building stock is important for informing stakeholders and developing strategies to improve community seismic resilience. To address this need, a parametric pushover curve model for seismic performance assessment of building stock is introduced. The model employs twelve parameters for estimating the trilinear pushover curve. The models of these parameters are developed specifically for reinforced concrete and masonry buildings in Slovenia by focusing on the low level of knowledge about the building. Consequently, it can be used for preliminary studies of seismic performance assessment of building stock to raise awareness about seismic risk. To evaluate its efficacy, the model was applied to assess the seismic performance of the University of Ljubljana's building stock. Because of uncertainties arising from the lack of knowledge about the buildings and the associated assumptions, the results were interpreted relative to those obtained for a new building stock scenario. The case study revealed that the seismic capacity in terms of the limit state peak ground acceleration of the building stock is only half of what would be expected in the new building stock scenario, highlighting a significant seismic risk. These findings emphasise the need for systematic efforts to enhance the seismic safety of the building stock under consideration. Further research is recommended to explore the model's accuracy concerning the level of knowledge about the building, which will contribute to a better understanding of its applicability.

Keywords Pushover curve · Parametric model · Building stock · Seismic performance assessment · Building knowledge · Reinforced concrete buildings · Masonry buildings

1 Introduction

The pushover-based seismic performance assessment of buildings is an intuitive approach used for designing earthquake-resistant buildings and evaluating the seismic performance of existing buildings. As such, it has been implemented in the new draft of Eurocode 8 (CEN 2021). However, pushover curves are also used for assessing the seismic performance

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of building stock. In this case, the pushover curves are usually transformed into capacity curves that are used to estimate fragility curves for the specific building types.

HAZUS (FEMA 2022), a software tool utilised for seismic risk analysis of the built environment, uses building capacity curves for various building types and three levels of design codes. These capacity curves are constructed based on estimates of engineering properties that impact the design capacity, yield capacity, and ultimate capacity of each building type. The properties taken into consideration include the design strength coefficient as a fraction of the building's weight, the elastic fundamental-mode period of the building, the fraction of the building weight effective in the pushover mode, the fraction of the building height at the location of the pushover mode displacement, overstrength factors that define the yield-to-design and ultimate-to-yield strength ratios, as well as a ductility factor that relates the ultimate displacement to the yield displacement. In Europe, a probabilistic framework (SP-BELA) has been developed and applied to generate vulnerability curves for reinforced concrete and masonry buildings using the variability in the pushover curves and the variability in the seismic demand prediction (Borzi et al. 2008a, b).

In a recent study by Kohrangi et al. (2021), the equivalent single-degree-of-freedom model was parameterised using nine parameters based on values from the literature rather than being determined from a sample of buildings. The parametric model provides a useful framework for estimating simplified trilinear pushover curves, considering the varying levels of available data for different buildings. A vulnerability modellers toolkit was also developed (Martins et al. 2021). The capacity module of the toolkit defines the capacity of structures using bilinear, trilinear, or quadrilinear pushover curves.

This paper first introduces a parametric model that provides a trilinear pushover curve of a building. The outcome of the model is a set of five independent coordinates used to define the three points of a trilinear pushover curve. Because it is very uncertain to estimate the points on the pushover curves directly, the five coordinates are estimated through twelve parameters classified into the basic structural parameters, essential load-bearing parameters, and essential deformation capacity parameters. The paper proceeds with presenting the parametric pushover curve model implementation for Slovenia's reinforced concrete and masonry buildings by focusing on low-level building stock data. The Slovenia's model was primarily developed to be used in preliminary studies of seismic performance assessment of building stock to raise awareness about seismic risk. In a case study, the parametric pushover curve model is used to estimate pushover curves and the limit-state peak ground accelerations for the 109 buildings of the University of Ljubljana's building stock under consideration. Results are communicated in relative terms, comparing the existing building stock to a new building stock scenario.

2 Parametric model of a building pushover curve

2.1 Pushover curve parametrisation

The proposed pushover curve parametrisation is based on an ideal elastoplastic pushover curve with softening (Fig. 1), which represents a simplified non-linear base shear–roof displacement relationship of a building. Such a trilinear pushover curve can be defined using only three points, i.e., the yield point Y, the capping point M and the near-collapse point U, with five independent coordinates (Fig. 1).

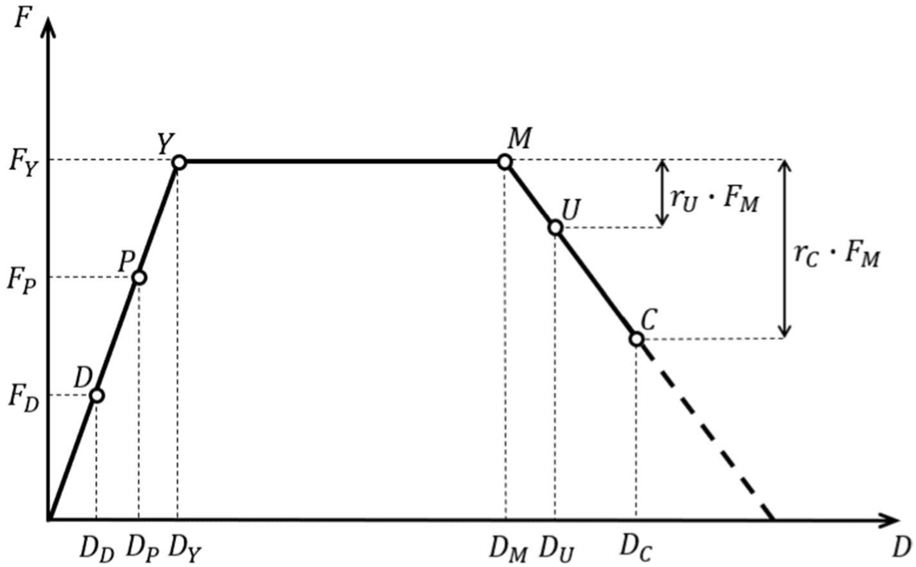

Fig. 1 A trilinear pushover curve of a building structure showing six points (D, P, Y, M, U, and C) with the corresponding base shears and roof displacements

Because the models for the direct estimation of yield, capping and near-collapse points can be uncertain, the pushover curve parametrisation is introduced using six points with interdependent coordinates (points D, P, Y, M, U, and C in Fig. 1). Such a parametrisation makes it possible to determine the five independent coordinates of the trilinear pushover curve indirectly based on structurally dependent parameters that can be measured, estimated by engineering judgement, or calculated depending on how much is known about the building structure. In the following section, the models showing six points for the pushover curve are introduced and related to the five coordinates that define the assumed trilinear pushover curve.

The base shear–roof displacement relationship is assumed to be linear until the yield point Y. If the building was designed for earthquake resistance, the model assumes that the maximum base shear of the building F_Y can be estimated gradually, starting by estimating the design base shear F_D:

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

where BS_C is the design base shear coefficient, m_i is the mass of the i-th storey of the building from the seismic design situation, and g is the acceleration of gravity. The sum in Eq. (1) is calculated for all storeys.

The base shear at the first plastification of the load-bearing structure can then be determined as:

$$F_P = q_S \cdot F_D, \tag{2}$$

where q_s is the overstrength factor (CEN 2021), which accounts for all sources of the overstrength (e.g., minimum requirements, minimal safety, etc.) except for structural redundancy. The redistribution of the seismic action effects in redundant structures can then be considered with the second component of the overstrength factor q_R, and the maximum base shear F_Y can be estimated as a product of q_R and F_P.

In a more general case, the model for F_Y should account for the possibility that the building was not designed for earthquake resistance. Therefore, the minimum base shear at the yield point $F_{Y,min}$ is introduced, and the model for F_Y is expanded as follows:

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

The models for $F_{Y,min}$ depend on many parameters, including the structural system type, the predominant material of the structure, and the geographical location. Simplified models for $F_{Y,min}$ for reinforced concrete and masonry buildings in Slovenia are introduced in Sect. 3.

The base shear in the plastic branch of the pushover curve is assumed to be constant until the softening of the building. Therefore, the base shear at the capping and yield point are the same:

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

The base shear at the near-collapse (NC) limit state is defined as a proportion of F_Y:

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

where r_U is the base shear reduction factor at the NC limit state. It can be assumed constant, for example, equal to 0.2 (CEN 2004), or defined as structurally dependent.

To establish the trilinear pushover curve, it is necessary to determine at least three displacements from the six points in addition to F_Y and r_U, with at least one displacement associated with the points on the elastic branch and two displacements associated with the points on the softening branch of the pushover curve. Thus, it was decided to determine displacement corresponding to F_Y, F_M and F_U: the displacement at the yield point D_Y, at the beginning of the softening $D_{M,}$ and the NC limit state D_U.

The yield displacement is still related to the linear elastic response of the structure, but it depends on several structural parameters. An equivalent single-degree-of-freedom (SDOF) model can be utilised to introduce the most important structural parameters into the formula for D_Y. Assuming the shape of the pushover curve of the structure and the shape of the equivalent SDOF model are the same, the fundamental period of the structure, T, and the period of the equivalent SDOF model, T^*, are equal. Consequently, the following correlation between the spectral displacement S_d and the spectral acceleration applies:

$$S_d = \frac{T^2}{4\pi^2} \cdot S_a. \tag{6}$$

where T is the fundamental period of the building with the trilinear pushover curve. The spectral acceleration from Eq. (6) can be determined as the ratio between the force and mass of the equivalent SDOF model (Fajfar 2000). If the force refers to the design base shear, the corresponding spectral acceleration is defined as follows:

$$S_{aD} = \frac{{F_D}^*}{m^*}. \tag{7}$$

where ${F_D}^*$ is the design base shear transformed to the equivalent SDOF model and m^* is the mass of the equivalent SDOF model that can be estimated as follows (Fajfar 2000):

$$m^* = \sum_{i=1}^{n_e} m_i \cdot \phi_i, \tag{8}$$

where ϕ_i is the component of the deformation shape vector corresponding to the i-th storey of the building and n_e is the number of storeys of the building above the ground level. The design base shear for the equivalent SDOF model is calculated according to Fajfar (2000):

$$F_D{}^* = \frac{F_D}{\Gamma}, \tag{9}$$

where Γ is the transformation factor that defines the relationship between the building structure model and the equivalent SDOF model:

$$\Gamma = \frac{m^*}{\sum m_i \cdot \phi_i{}^2}. \tag{10}$$

The displacement of the SDOF model that corresponds to the design base shear is then derived by considering Eqs. (6–7) as follows:

$$D_D{}^* \equiv S_{dD} = \frac{T^2}{4\pi^2} \cdot \frac{F_D{}^*}{m^*}. \tag{11}$$

Analogously, the yield displacement at the equivalent SDOF model can then be determined as:

$$D_Y{}^* = \frac{T^2}{4\pi^2} \cdot \frac{F_Y{}^*}{m^*}. \tag{12}$$

Finally, the displacements of the equivalent SDOF model should be transformed into the roof displacements D_D and D_Y (Fajfar 2000):

$$D_D = \Gamma D_D{}^* = \frac{T^2}{4\pi^2} \cdot \frac{F_D}{m^*}, \tag{13}$$

$$D_Y = \Gamma D_Y{}^* = \frac{T^2}{4\pi^2} \cdot \frac{F_Y}{m^*}. \tag{14}$$

The calculation of the displacement at the NC limit state D_U cannot be determined by Eq. (6) because this equation is solely applicable to linear-elastic SDOF models. The following parametric model is thus introduced for the estimation of D_U:

$$D_U = C_U \cdot \theta_U \cdot \sum_{i=1}^{n_e} h_{e,i}, \tag{15}$$

where C_U is the storey-drift-angle uniformity coefficient, θ_U is the storey drift angle at the NC limit state of the most deformed storey, n_e is the number of storeys of the building above the ground level and $h_{e,i}$ is the i-th storey height. The θ_U can be determined as the deformation capacity of the most important structural elements as defined in building codes (CEN 2021), while C_U accounts for the deformation shape of the building in the case of the NC limit state. If the storey drift angle at the occurrence of the NC limit state is the same in all storeys, then $C_U = 1$. Conversely, when the extreme soft storey effect is present and storey height is constant, $C_U = 1/n_e$. For all other cases, C_U is between the bounding values, but it can be, to some extent, more precisely estimated with reference to the structural system of the building and the earthquake-resistant design approach.

Finally, the roof displacement at the beginning of the softening D_M and at the collapse D_C can be calculated by assuming the linear post-capping branch of the trilinear pushover curve:

$$D_M = \frac{D_U}{\mu_{0M} - (1 - r_U) \cdot \mu_{0M} + (1 - r_U)}, \tag{16}$$

$$D_C = D_M \cdot (1 + \mu_{0M} r_C - r_C), \tag{17}$$

where μ_{0M} is the post-capping ductility, defined as the ratio between the displacement at the theoretical zero base shear (i.e., where the dashed line in Fig. 1 touches the horizontal axis) and the displacement at the beginning of softening D_M, and r_C is the base shear reduction factor at collapse. Ductility μ_{0M} is considered to be structurally dependent so it is treated as a model input parameter, while r_C is assumed constant. According to Cattari et al. (2018), it could be assumed equal to 0.5. However, the proposed model can be further generalised by considering r_C as structurally dependent.

Based on the above assumptions and derivation, the proposed parametric model of the pushover curve is defined by twelve structurally dependent parameters ($BS_C, \boldsymbol{m}, q_S, q_R, F_{Y,min}, T, \boldsymbol{\phi}, \mu_{0M}, C_U, \theta_U, n_e$ and $\boldsymbol{h_e}$) where $\boldsymbol{m}$, $\boldsymbol{h_e}$ and $\boldsymbol{\phi}$ are the storey mass vector, storey height vector and deformation shape vector, respectively. Parameters $BS_C, \boldsymbol{m}, q_S, q_R$ and $F_{Y,min}$ primarily affect the load-bearing capacity of the building, while $\mu_{0M}, C_U, \theta_U, n_e$ and $\boldsymbol{h_e}$ are associated with the building deformation capacity. Finally, both the load-bearing and the deformation capacity of buildings are affected by T and $\boldsymbol{\phi}$.

Estimation of some parameters is straightforward, while others can be introduced by models, depending on the knowledge level about the building under consideration. For an easier understanding of the entire parametric model, the twelve parameters are classified into three groups: basic structural parameters, essential load-bearing parameters, and essential deformation capacity parameters.

2.2 Basic structural parameters

The basic structural parameters as per the proposed methodology are: m_i, T, ϕ_i, n_e and $\boldsymbol{h_e}$.

The number of storeys above the ground level n_e and the storey height vector $\boldsymbol{h_e}$ can be assessed during the building inspection, adopted from building design documentation, or even from building stock databases if they are available for the building stock being studied.

There are several approaches to estimating the building mass. The mass of the i-th storey of the building can be obtained from the design documentation, estimated using the requirements of the building code that was in force at the time of its design or based on the average mass per unit floor area m_A typical for the building's structural system. Alternatively, the mass per unit floor area can be defined by standard recommendations, e.g., (JBDPA 2001).

Many models are available for the estimation of the fundamental vibration period T (Crowley and Pinho 2004; Chopra and Goel 2000; Hong and Hwang 2000). On the other hand, T can also be determined with ambient or forced vibration tests (Snoj et al. 2013; Ditommaso et al. 2013). However, in such cases, the period should be increased because the initial stiffness of the trilinear pushover curve refers to the secant stiffness of the building. In this study, the HAZUS (FEMA 2022) and POTROG (Lutman et al. 2013; 2016; 2018) models were used to estimate T of reinforced concrete and masonry buildings, respectively. The POTROG model had been developed based on the measurements of the fundamental vibration periods of undamaged existing structures. To account for the secant stiffness of the idealised pushover curve, the fundamental vibration periods obtained from the POTROG model were multiplied by a factor of $\sqrt{2}$, which approximates the effect of a 50% reduction of the cross-section's shear area and moment of inertia.

The deformation shape vector $\boldsymbol{\phi}$ depends on the building type and regularity. It can be assumed equal to the fundamental mode shape of the building if such information is available from the design documentation. Otherwise, $\boldsymbol{\phi}$ can be calculated based on guidelines available in building codes and past studies (CEN 2004; Fajfar 2000).

2.3 Essential load-bearing parameters

The remaining parameters that influence the structural load-bearing capacity in the proposed model are: BS_C, q_S, q_R and $F_{Y,min}$. These parameters are dependent primarily on the type of building structure and its seismic resistance design level, which can be assumed based on the building's location and year of construction. The estimation of the parameters is greatly dependent on how much is known about the building structure.

The base shear coefficient BS_C is a specific parameter because it depends on many factors, including the level of perceived seismic hazard at the location of the building when it was designed, and the region- and period-specific building codes. If available, the BS_C can be obtained from the design documentation. Otherwise, BS_C can be considered to be dependent on the type of structural system, location and design period. A comprehensive overview of the BS_C models for European building stock is presented elsewhere (Crowley et al. 2021).

The components of the overstrength factor (q_R and q_S) can be estimated by performing a pushover analysis of the building structure, but pushover analysis is not performed for each building in the seismic performance assessment of the building stock. Therefore, the values for both components of the overstrength factor may be directly estimated using the working version of the draft of the new Eurocode 8 (CEN 2021). However, for buildings that were not designed according to Eurocodes, the two components of the overstrength factor can generally be estimated through parametric studies. Such studies or building databases may become available in the future. Some guidance can be given to estimate q_R and q_S for older buildings for which one can assume the seismic design action governed the design. Firstly, it is reasonable to assume that, for such buildings, q_R is lower than the corresponding value in Eurocode 8. Earlier standards did not include strict capacity design rules to ensure the global ductility of the structure or the local ductility of critical structural elements. As a result, the potential for redistribution of seismic action in older buildings is less than that in newer buildings. Nevertheless, q_R for older buildings may be considered greater than 1.0, since the $q_R = 1.0$ standard applies to the inverted pendulum system. On the other hand, q_S essentially depends on the building code. Building codes from different periods implement different design factors, which affect the load-bearing capacity. Eurocode 8 (CEN 2004) prescribes it at 1.5. In a more general case, the q_S can be estimated through parametric studies. However, it is possible that the yield strength of a building is not controlled by the seismic design base shear. This can be the case not only for older buildings but also for buildings constructed after the implementation of modern building codes if they are in areas with low seismicity. This is why the proposed parametric model considers $F_{Y,min}$ (Eq. 4), which depends on the material and type of structure. Both parameters vary with the period and the region of the construction.

2.4 Essential deformation capacity parameters

The parameters that directly affect the deformation capacity of a building are C_U, θ_U and μ_{0M} (Eqs. 15 and 16). The bounding values of the storey-drift-angle uniformity coefficient C_U have been discussed in Sect. 2.1. Because C_U accounts for the deformation shape of the building associated with the NC limit state, it depends on the structure type and the building code. Initially, the building codes did not pay much attention to the design of the global structural ductility, which is essential for seismic safety in regions of high seismicity. If detailed information about the building is unavailable, C_U can be estimated by parametric studies or through a review of existing studies that utilise pushover analysis. The same approaches can be used to estimate θ_U and μ_{0M}. If detailed results of pushover analyses are unavailable for a particular building type, then θ_U can be estimated by considering building codes aimed at assessing the seismic performance of existing buildings (CEN 2004). In such cases, it is often assumed that one structural element controls the deformation capacity of the entire building.

3 Implementation of parametric pushover curve model for reinforced concrete and masonry buildings in Slovenia by taking into account low level of knowledge of the buildings

The model introduced in Sect. 2 is implemented for Slovenia's reinforced concrete and masonry buildings. In the model implementation, it is assumed that not much is known about the buildings under consideration. Consequently, the building data were primarily obtained from a publicly accessible real estate register (GRS 2022) and included building location, year of construction, total net area, number of storeys, number of storeys above ground level, predominant material of the load-bearing structure, elevation of the terrain, and the elevation of the roof of the building. Only a few parameters of the parametric model of the pushover curve can be assessed directly from this publicly available data. Additional models are therefore needed, but some input parameters are based on expert judgement as well. The detailed implementation of the model is presented in the following sections.

3.1 Basic structural parameters

The available data included n_e, while $\boldsymbol{h_e}$ was calculated based on n_e, the elevation of the terrain, and the elevation of the roof of the building, assuming a constant storey height $h_{e,i}$.

The mass of the i-th storey of the building was estimated based on the building's gross floor area A_G and the average mass per unit floor area m_A. The latter can be considered building typology-specific and was assumed to be $1.2t/m^2$ for reinforced concrete buildings and $1t/m^2$ for masonry buildings, based on previous studies (Dolšek et al. 2020; Snoj and Dolšek 2020). The HAZUS (FEMA 2022) and POTROG (Lutman et al. 2013; 2016; 2018) models were used to estimate T of reinforced concrete and masonry buildings, respectively.

The deformation shape vector $\boldsymbol{\phi}$ was considered to be inverted triangular due to the absence of data about building irregularity in elevation.

3.2 The essential load-bearing capacity parameters for reinforced concrete and masonry buildings in Slovenia

This section presents the models for BS_C, q_S, q_R and $F_{Y,min}$ that can be applied to reinforced concrete and masonry buildings in Slovenia when the level of knowledge about the buildings is low.

The BS_C varies significantly for different periods. According to Fajfar (2017), the minimum horizontal seismic action for the design of buildings in Slovenia was introduced in 1948 and was not changed until 1963. For buildings with reinforced concrete walls and ceilings (roofs), the design base shear was assumed at 1% of the weight, accounting for the dead load of the building and 50% of the live load. For buildings with reinforced concrete walls and lightweight ceilings, the design base shear was equal to 1.2% of the weight, while for buildings with lightweight reinforced concrete walls and ceilings, the ratio between the design base shear and the weight was equal to 1.5% (Fajfar 2017). It was a requirement that the design base shear be increased by 50% or 100% depending on the location of the building and the corresponding seismicity level (Fajfar 2017). For simplicity, the value of BS_C was set at 0.02 for all buildings constructed before 1963. Nevertheless, it should be noted that the load-bearing capacity of buildings constructed before 1963 had not always been governed by seismic design action, as evidenced by subsequent models for $F_{Y,min}$.

In 1963, the first Slovenian code for earthquake-resistant design was introduced (UL SRS 1963), but it was superseded a year later by the first Yugoslav code (UL SFRJ 1964), which adopted the methods defined in the Slovenian regulation for structural analysis (Fajfar 2017). The next iteration of building codes was introduced in 1981, following a brief period (1978–1981) of revised regulations from 1963. The changes in the legislation followed major earthquake events in Europe, which was typical for many other countries as well. The Eurocode has been used in Slovenia for earthquake-resistant design of buildings since 2008. Given the multiple changes in the definition of BS_C in the past 70 years, it is defined as a parameter that is dependent on the building construction year (CY). Considering the rules from previous building codes (Fajfar 2017) and Eurocode 8 (CEN 2004), BS_C can be modelled as:

$$BS_C = \begin{cases} 0.02; CY \leq 1963 \\ \begin{cases} 0.04 \cdot \gamma_{I,1}; T < T_C \\ 0.13 \cdot \gamma_{I,1}; T > T_C \end{cases} ; 1963 < CY \leq 1970 \\ \begin{cases} 0.02 \cdot \gamma_{I,1}; T < T_C \\ 0.06 \cdot \gamma_{I,1}; T > T_C \end{cases} ; 1970 < CY \leq 1981 \\ K = K_0 \cdot K_s \cdot K_d \cdot K_p; 1981 < CY < 2008 \\ S_{ad}\left(PGA_{475}, S, T_B, T_C, T_D, q\right) \cdot \lambda \cdot \gamma_I; 2008 \leq CY \end{cases} \tag{18}$$

where $\gamma_{I,1}$ is the building's importance factor, which is set to 1.5 for buildings deemed as important, and to 1.0 for buildings of ordinary importance. K is the seismic coefficient that was in use between 1981 and 2008 and consists of four components.K_0 accounts for the building category and was set to 1.5 for buildings of the first category (i.e., important buildings) and 1.0 for other buildings. K_s is the seismic intensity coefficient that is based on the building's location. For example, buildings in Ljubljana, including those investigated in Sect. 4, fell under seismic zone VIII according to the seismological map in use at the time.

For such buildings, K_s was between 0.04 and 0.06. K_d is the dynamic factor, which depends on the natural period of the structure and soil type at the building's location, and K_p is the factor that accounts for the damping and ductility of the building, which was considered dependent on the material of the structure. The value of K_p was 1.6 for masonry buildings and 1.0 for reinforced concrete buildings.

The last model in Eq. (18), which is considered for $CY \geq 2008$, is the Eurocode 8 model (CEN 2004). This model incorporates the design acceleration spectrum S_{ad}, which depends on many parameters, including peak ground acceleration corresponding to a 475-year return period and the behaviour factor q. This factor accounts for both the overstrength of the structure and its ability to dissipate the energy from strong ground motions. The structural system is not precisely defined because the level of knowledge about the building is low, so $q = 3$ was assumed for the reinforced concrete buildings and $q = 2$ for the confined masonry buildings (CEN 2004). The Eurocode 8 model also considers the soil type, factor S, which was evaluated based on a soil class database established for a previous study (Dolšek et al. 2020). To estimate BS_C according to Eurocode 8, the correction factor that accounts for the effective mass λ should also be implemented. For simplicity, λ is assumed to be equal to 1.0 or 0.85, depending on the period of vibration and the number of storeys, as prescribed in Eurocode 8 for the lateral force method of seismic analysis (CEN 2004). Finally, γ_I is the building's importance factor. For example, γ_I is set to 1.2 for the school and faculty buildings, which are analysed in the case study.

The overstrength factors q_R and q_S are also period dependent. For buildings constructed after 2008, q_R was estimated based on the working version of the draft of the new Eurocode 8 (CEN 2021). For reinforced concrete buildings, a q_R corresponding to a dual wall-equivalent structural system was adopted because this structural system is the most typical in Slovenia (Table 1). The average of the two values corresponding to the layouts with a significant coupling effect was considered for masonry buildings (Table 1). For older buildings, a lower q_R was defined based on expert judgement. Moreover, q_S was assumed equal to 1.5 for all buildings, which is the default value in Eurocode 8 (CEN 2004).

3.2.1 The minimum load-bearing capacity model for reinforced concrete buildings

To assess the minimum load-bearing capacity of reinforced concrete structures, we used a method prescribed by the Japanese standard for seismic evaluation of existing buildings (JBDPA 2001), which was also considered in the study by Sinkovič et al. (2016). The first level of complexity of the Japanese approach was adopted because little was known about the buildings. The Japanese approach was further simplified to estimate the base shear for reinforced concrete and masonry buildings that had not been designed for earthquake resistance.

Table 1 Adopted values of components of the overstrength factor for different construction years

Construction period	Masonry buildings		Reinforced concrete buildings	
	q_R	q_S	q_R	q_S
≤1963	1.05	1.5	1.10	1.5
1964–2007	1.10	1.5	1.15	1.5
≥2008	1.30	1.5	1.2	1.5

The base shear, according to the Japanese approach, is estimated through the strength index (JBDPA 2001):

$$C_g = \frac{\sum_j \tau_j \cdot A_{s,g,j}}{W}, \tag{19}$$

where index g determines the group of the vertical elements of the load-bearing structure ($g = w$ for walls, $g = c$ for columns, and $g = sc$ for short columns), $A_{s,g,j}$ is the cross-sectional area of the j-th element of the g-th group of vertical elements under consideration, τ_j represents the corresponding limit value of shear strength in terms of stress, and W is the weight of the entire building based on its mass $\sum m_i$. In the original method (JBDPA 2001), τ was estimated for reinforced concrete buildings in Japan. However, Sinkovič et al. (2016) calibrated τ for the European region based on 208 experimentally tested columns, and proposed the values of 1.5 MPa for short columns, 0.9 MPa for standard columns, and 0.4 MPa for very slender columns. For walls built before and after the 1981 building code, Sinkovič et al. (2016) proposed $\tau = 1.0$ MPa and 2.0 MPa, respectively. It should be noted that the proposed limit values indirectly account for the effect of ductility on the building's ultimate capacity. However, in the parametric pushover curve model proposed in this study, structural ductility is directly considered. Therefore, to avoid double-counting of the beneficial effect of ductility, the limit values of shear strength proposed by Sinkovič et al. (2016) were reduced. The reduction for all columns and walls built before 1981 was set to 50%, thus assuming that one-half of these elements' ultimate capacity originated from their ductility. The same shear strength limits were considered for walls built after 1981, as for those built before 1981, assuming that the 1981 code mostly improved the ductility of elements, while the strength was not affected. Therefore, the limit values of shear strength were: 0.75 MPa for short columns, 0.45 MPa for normal columns, 0.2 MPa for very slender columns, and 0.5 MPa for walls, regardless of the CY period.

Equation (19) cannot be used directly if there is limited knowledge of the buildings because the cross-sectional areas of the structural elements are not available. So, instead of considering a cross-sectional area of each element, the total cross-sectional area of the g-th group of elements $A_{s,g}$ is estimated as:

$$A_{s,g} = A_T \cdot \rho_g, \tag{20}$$

where ρ_g is the proportion of the total cross-sectional area of the g-th group of elements in the bottom storey and A_T is the area of the bottom storey. Because of the low level of knowledge about the buildings, A_T is estimated as the total area of the building divided by the number of storeys. The model foresees the use of default values of ρ_g. To estimate the ρ_g values, several existing reinforced concrete buildings constructed before 1981 were examined. It was realised that ρ_g for walls and columns can be assumed to be 0.55%. However, there are fewer columns and more walls for the period after 1981. For buildings constructed between 1982 and 2007, the adopted ρ_g corresponds to the required proportion of walls in the regulations in force at the time (UL SFRJ 1981), which recommended a minimum value of 1.5%. This same value was deemed applicable to the period after 2008. The default ρ_g values are presented in Table 2.

Based on the adopted default values of structural-element-dependent τ and CY-dependent ρ_g, the minimum base shear for reinforced concrete buildings was estimated by considering the strength indexes (Eq. 19) for walls and columns:

Table 2 Ratios between the cross-sectional area of the walls or columns and the area of the storey controlling the minimum base-shear capacity of the building

ρ_g	≤1981	1982–2007	≥2008
wall ratio (%	0.55	1.5	1.5
column ratio (%)	0.55	0	0

$$F_{Y,min,RC} = \left(C_W + \alpha_C C_C\right)W, \tag{21}$$

where α_C is the strength utilisation ratio of columns at the moment when the maximum base shear is achieved in the walls, which was assumed to be 0.7 (Sinkovič et al. 2016).

3.2.2 The minimum load-bearing capacity model for masonry buildings

The estimation of the minimum load-bearing capacity for masonry buildings is challenging because of the large variety of masonry and construction techniques used in different construction periods. However, due to the typical geometry of load-bearing walls, the material properties of the masonry, and the fact that the majority of masonry buildings were constructed before 1964, it can be argued that shear failure is the most prevalent and representative mechanism. In this mechanism, the primary tensile stresses resulting from the combination of vertical and horizontal forces in the central area of the wall exceed the tensile strength of the masonry, resulting in characteristic inclined cracks (Tomaževič 2009; Turnšek and Čačovič 1971).

The shear resistance of the masonry wall was thus estimated as follows (Tomaževič 2009; Turnšek and Čačovič 1971):

$$R_{S,W} = A_W \cdot \frac{f_t}{b} \cdot \sqrt{\frac{\sigma_0}{f_t} + 1}, \tag{22}$$

where f_t denotes the mean tensile strength of the masonry wall, A_W is the cross-sectional area of the wall oriented in the direction of the earthquake ground motion, σ_0 is the average compressive stress in the wall cross-section that results from the constant vertical load and b is the cross-sectional shear stress distribution factor, which depends on the ratio between the height and the length of the wall (Tomaževič 2009).

The mean value of the tensile strength of the masonry f_t for buildings constructed before 1981 was assumed based on the draft of the new Eurocode 8–3 (wdEN 1998-3:2019), which prescribes a value of 0.114MPa. It was assumed that most buildings constructed after 1982 were built from hollow block masonry. Thus, the average tensile strength was determined based on the characteristic tensile strength for this type of masonry (0.20 MPa) estimated from experimental data (Tomaževič 2009), and by applying the coefficient of variation proposed in the working version of the new Eurocode 8–3. This yields $f_t = 0.30$MPa, which is the same value as proposed elsewhere (MIT, 2009).

Due to incomplete building data, the cross-sectional shear stress distribution factor b was assumed to be 1.25, which is a value typical for masonry walls in Slovenia, and A_W was estimated by analogy to Eq. (20) as:

$$A_W = A_T \cdot \rho_W, \tag{23}$$

Table 3 The ratio ρ_w and the mean tensile strength for different periods of building construction

Construction period	≤ 1963	1964–1981	≥ 1982
ρ_W[%]	5.40	4.80	3.50
f_t[MPa]	0.114	0.114	0.30

Table 4 Proposed values of the factor of reduced connectivity k_{np} for different types of buildings

Construction type	k_{np}
up to 3 storeys, wooden or arched floors	0.75
up to 3 storeys, reinforced concrete floors at all levels	0.90
4–7 storeys, wooden or arched floors	0.675
4–7 storeys, reinforced concrete floors at all levels	0.90

where A_T is the area of the storey and ρ_W is the ratio between the total cross-sectional area of the walls oriented in the direction of ground motion and the area of the storey. The ρ_W for buildings constructed before 1963 was estimated based on the analysis of eight buildings that are part of the University of Ljubljana's building stock (Siebenreich 2015; Bosiljkov 2018; GI-ZRMK 2019). The proportion of walls in those buildings, on average, amounted to 5.40%. For buildings constructed between 1964 and 1981, seven buildings that had been damaged in the Posočje earthquake of 1998 (DTP 2009) were analysed to estimate ρ_W, which amounted to 4.80%. However, for masonry buildings constructed after 1982, it was assumed that the cross-sectional area of walls is smaller compared to previous periods. The ρ_W was thus assumed to be 3.5%, which is also prescribed as the minimum ratio by Eurocode 8 (CEN 2004). The applied values of tensile strength and wall ratios for different periods are shown in Table 3.

To apply Eq. (22) for the calculation of the minimum load-bearing capacity of masonry buildings, the average value of compressive stress in the wall cross-sections was estimated as follows:

$$\sigma_0 = \frac{W}{A_{W,ALL}}, \tag{24}$$

where $A_{W,ALL}$ was, in the absence of more detailed data, assumed $2A_W$.

It should be noted that Eq. (22) is based on the assumption that a parallel system of equal walls can simulate the base shear of a masonry building. However, this assumption does not apply in general. Buildings constructed before 1964 typically have wooden floors and arches, which does not justify the assumption of rigid diaphragms for floor structures. As this directly affects the building's base shear, the minimum base shear of the building was reduced by the building connectivity factor k_{np} (Tomaževič et al. 1991). The factor k_{np} depends on the wall connectivity parameter and the number of storeys in the building. The wall connectivity parameter can be determined based on the type of floor structure and the age of the building (Lutman 2003). Due to the consideration of incomplete building data, the determination of k_{np} was further simplified. The values are presented in Table 4.

The minimal load-bearing capacity of masonry buildings was finally determined as:

$$F_{Y,min,M} = \begin{cases} k_{np}R_{S,W}; CY < 1964 \\ R_{S,W}; CY \geq 1964 \end{cases}. \tag{25}$$

3.3 The essential deformation capacity parameters for reinforced concrete and masonry buildings in Slovenia

If there is insufficient information about the building, the modelling of C_U, θ_U and μ_{0M} must be simplified. An insight into C_U for typical reinforced concrete buildings in Slovenia was obtained by analysing the results of previous studies for eight pre-1981 buildings with a dual structural system (Celarec 2012; Sinkovič et al. 2016). In all cases, C_U was found to be close to 1.0, leading to the decision that a value of $C_U = 1.0$ can be adopted for reinforced concrete buildings with a dual structural system, which are the most common in Slovenia. For masonry buildings, a predominantly soft-storey failure mechanism was assumed in the estimation of C_U. However, it was assumed that 20% of the deformations occurring in the bottom storey would also be present in the storey above the soft storey. Therefore, C_U was assumed to be $1.2/n_e$ for masonry buildings, where n_e corresponds to the number of storeys above ground level. The equation is only applicable to buildings with more than one storey above ground level. For single-storey buildings, the value of $C_U = 1.0$ applies by definition, regardless of the type of building.

The storey-drift angle θ_U of masonry walls at the NC limit state was estimated according to Appendix C of Eurocode 8–3 (CEN 2004), which suggests that the ultimate drift angle of 0.0107 and 0.0053 for individual walls is governed by the flexural and shear failure mechanism, respectively. For buildings constructed after 1964, the θ_U was estimated to be 0.008, as an average value of the values referring to the flexural and shear failure mechanisms. However, for structures built before 1963, we assumed that the shear failure mechanism prevails, so we adopted a limit storey-drift angle of 0.0053 at the NC limit state.

For reinforced concrete buildings, the values of limit storey-drift angles were adopted based on results from past studies (Kosič, 2014; Sinkovič et al. 2016). Sinkovič et al. (2016) analysed buildings with dual structural systems built before 1981. Based on eight analysed buildings, an average value of the limit storey-drift angle of 0.0125 was estimated. Kosič (2014) analysed buildings designed according to Eurocode 8 (CEN 2004) and showed that the NC limit state of a wall building was attained at the storey-drift angle from 0.030 to 0.036. An average value of $\theta_U = 0.033$ was used in the definition of the pushover curves for reinforced concrete buildings constructed after 2008. For reinforced concrete buildings built between 1982 and 2007, θ_U was assumed as 2/3 of the limit storey-drift angle defined for buildings built after 2008, because the building code from 1982 guaranteed local ductility of structural elements to some extent.

The values of post-capping ductility were based on the results of previous studies (Kosič, 2014; Sinkovič et al. 2016; Snoj and Dolšek 2020). The adopted values of all the parameters introduced in this section are presented in Table 5.

Table 5 The estimated values of C_U, θ_U and μ_{0M} for buildings in Slovenia using incomplete data

Construction period	Masonry buildings			Reinforced concrete buildings		
	C_U	θ_U	μ_{0M}	C_U	θ_U	μ_{0M}
≤ 1963	$1.2/n_e$	0.0053	1.2	1.0	0.0125	3.5
1964–1981		0.0080			0.0125	
1982–2007		0.0080			0.0220	
≥ 2008		0.0080			0.0330	

4 Case study: seismic performance assessment of existing building stock relative to new building stock

The parametric pushover curve model is used to estimate pushover curves of the building stock of the University of Ljubljana (UL), which is considered critical infrastructure in Slovenia, and the corresponding limit-state peak ground accelerations are based on the N2 method implemented into Eurocode 8 (CEN 2004). Because estimated pushover curves are based on limited knowledge about the building, the results of the seismic performance assessment of the critical infrastructure are not communicated in absolute terms, but in relative terms, so a new building stock scenario was defined. It reflects the same buildings as the existing building stock but assumes that all buildings were designed and constructed after 2008. The seismic performance assessment of the new building stock scenario is then performed by analogy to the existing building stock. The strength, deformation capacity D_U and the limit-state peak ground corresponding to D_Y and D_U of the existing building stock relative to the new building stock scenario are discussed.

4.1 Description of the existing building stock and the new building stock scenario

The UL building stock under consideration comprises 109 buildings that are used for research and/or teaching, and have an estimated real estate value exceeding €70,000. The net area of the so-determined building stock is over 300,000 m^2, which represents about 95% of the net area of all facilities. Most of the 109 buildings are in Ljubljana, with only a few in other municipalities.

The parametric pushover curve model introduced in Sects. 2 and 3 was used to analyse the building stock. The building stock data were primarily obtained from a publicly accessible real estate register (GRS 2022). The building importance category was determined based on expert opinion and criteria from building codes, while the soil type for the location of each building was estimated using the soil type database established for the seismic stress test of building stock in the Republic of Slovenia (Dolšek et al. 2020).

The analysis of building stock data revealed that the UL building stock is old. More than 50% of the buildings were constructed before 1964, that is, before the introduction of the first building code for earthquake-resistant design in Slovenia. Of the 55 buildings from that period, 44 are masonry (MSN) buildings, and 11 are reinforced concrete (RC). Only six buildings have been constructed since the introduction of Eurocode 8, all of which are RC buildings. Table 6 presents a more detailed classification of the UL building stock by construction year and predominant structural material.

Table 6 The number of buildings of the UL building stock for different *CY* periods and structural materials (RC Reinforced concrete, MSN Masonry)

Construction period	Number of buildings		
	Total	RC	MSN
≤ 1963	55	11	44
1964–1981	25	18	7
1982–2007	23	16	7
≥ 2008	6	6	0

About 30% of the total building stock area is masonry buildings (Fig. 2, Table 7). They are mostly older than the RC buildings, with 75% of all the masonry buildings in the UL building stock having been constructed before 1964. They range from one to six storeys with the majority of masonry buildings from that period having between two and four

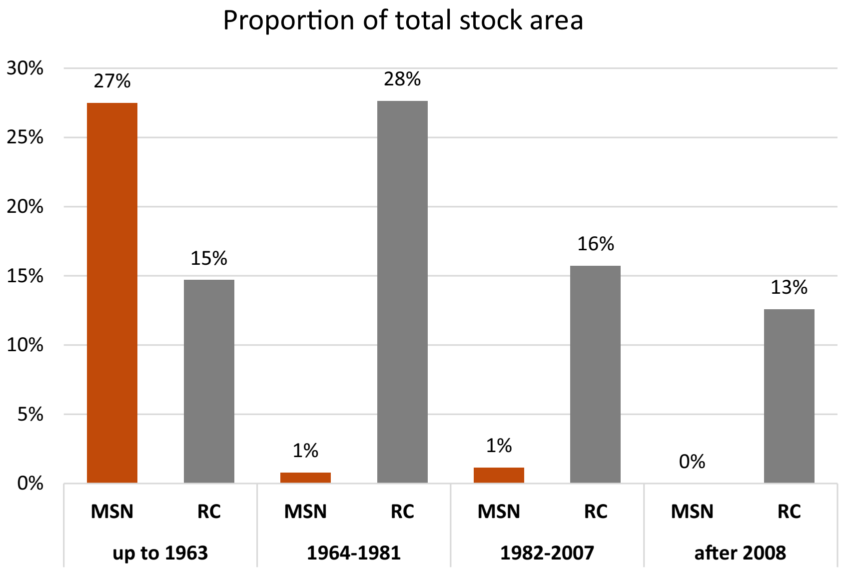

Fig. 2 Percentages of the total net area of the UL building stock for different *CY* periods and structural materials (RC–reinforced concrete, MSN–masonry)

Table 7 The number of masonry (MSN) buildings and their net area as the percentage of the total stock area as a function of construction period (*CY*) and the number of storeys

MSN	≤ 1963		1964–1981		1982–2007		Σ	
n_e	N	A [%]	N	A [%]	N	A [%]	N	A [%]
1	5	0.6	4	0.3	3	0.6	12	1.5
2	14	2.9	1	0.1	3	0.4	18	3.4
3	12	8.3	2	0.4	1	0.2	15	8.8
4	10	12.9					10	12.9
5	2	1.0					2	1.0
6	1	1.8					1	1.8
Σ	44	27.5	7	0.8	7	1.1	58	29.4

storeys. However, masonry buildings constructed between 1964 and 2007 tend to have between one and three storeys.

The remaining 70% of the building stock area is RC buildings (Fig. 2, Table 8). Most RC buildings were built between 1964 and 1981, and range from two to 14 storeys in height. These buildings represent almost 28% of the total stock area. Most newer and older RC buildings have up to six storeys and are relatively evenly distributed across the remaining construction year categories (Table 8). Tables 7 and 8 present more details about the building stock structure related to the number of buildings (N) and their net area as the percentage of the total building stock area (A). In these tables, the buildings are divided into categories by material, year of construction and number of floors above ground level.

In addition to the existing building stock, the new building stock scenario was defined to enable the comparison of the seismic performance of the existing building stock relative to the hypothetically rebuilt building stock. The new building stock scenario comprises the same buildings as the existing building stock, but the structures of the new building stock scenario are consistent with Eurocode 8. In addition, all existing masonry buildings with three or more storeys were replaced in the new building stock scenario with reinforced concrete buildings, while the buildings with one or two storeys remained masonry. Such a decision was made because masonry construction is not considered suitable for high buildings. Consequently, the parametric pushover curves for the new building stock scenario were derived by considering most of the input parameters equal to those of the existing building stock, except for the parameters affecting the strength and deformation capacity if they were determined based on the model for $CY < 2008$. Those parameters were updated due to the requirements for the new structures and, in some cases, due to switching from a masonry to a reinforced concrete building structure.

4.2 Description of seismic performance assessment process

A Python code comprised of several scripts was developed to implement the parametric pushover curve model introduced in Sects. 2 and 3. The input data are divided into the

Table 8 The number of RC buildings and their net area as the percentage of the total stock area as a function of CY and the number of storeys

RC	≤ 1963		1964–1981		1982–2007		≥ 2008		Σ	
n_e	N	A [%]	N	A [%]	N	A [%]	N	A [%]	N	A [%]
1					5	2.2			5	2.2
2	2	1.0	3	1.9	2	1.0	1	0.1	8	3.9
3	2	3.1	3	3.4	5	5.6	2	1.3	12	13.3
4	3	2.5	3	3.9			1	1.2	7	7.6
5	1	1.9	1	3.5	3	6.5	2	10.0	7	21.9
6	1	3.9	2	3.5					3	7.4
7	1	1.7	2	5.0	1	0.4			4	7.1
8			1	3.0					1	3.0
9	1	0.8%	1	0.2					2	1.0
11			1	2.9					1	2.9
14			1	0.3					1	0.3
Σ	11	14.7	18	27.6	16	15.7	6	12.6	51	70.6

building stock data and the data of the models supporting the parametric pushover curve model (models for T, $\boldsymbol{m}$, Γ, BS_C,$F_{Y,min}$, forces and displacements of the pushover curve) as well as those used to determine the limit-state displacements and the corresponding limit-state peak ground accelerations (PGA).

In the first step of the process, the structural vibration period is calculated based on the material of the building structure and the number of storeys above ground level, as described in Sect. 3.1. This is followed by the estimation of the mass of the building based on the number of storeys, structural material, and the total building area.

A subsequent script is used to determine the BS_C (Eq. 18). The $F_{Y,min}$ is then defined as presented in Sect. 3.2.1 and 3.2.2. From the obtained parameters, the forces of the points of the pushover curve of the analysed building are determined. In this process, the F_D, F_P and F_Y are calculated (Eqs. 2 and 3) and divided by the transformation factor (Eq. 10) to obtain the corresponding forces of the equivalent SDOF model. The D_D, D_P, D_Y and D_U are then calculated based on the approach explained in Sects. 2.1 and 3.3. The displacements are then transformed to obtain the force–displacement relationship of the equivalent SDOF model. Finally, limit-state spectral accelerations at the building's fundamental period are predicted by a Eurocode 8 (CEN 2004) procedure, and the corresponding limit-state PGAs are obtained considering the elastic spectra from Eurocode 8 (CEN 2004).

All the described steps are repeated for each building from the building stock. The input data, intermediate results and the pushover curves are stored in a file for post-processing.

4.3 The results

The pushover curves of the existing building stock are presented in Fig. 3, and compared to the pushover curves of the new building stock scenario (Figs. 4 and 5). The pushover curves are presented as base shear and roof displacement normalised, respectively, by the total weight of the building and the total height of the building.

In the existing building stock, the oldest masonry buildings have the lowest deformation capacity, and the newest have the highest deformation capacity, as expected (Fig. 3). The same trend applies to the load-bearing capacity of masonry buildings. For RC buildings, the trend of increasing deformation capacity is even more evident, while the trend of increasing load-bearing capacity is less apparent for CY ≥ 2008. It can also be observed that the deformation capacity of RC buildings is greater than that of the masonry buildings for each construction period, but the normalised load-bearing capacity is greater for the masonry buildings.

Figures 4 and 5 show that the buildings from the new building stock scenario have a higher normalised load-bearing capacity and a higher normalised deformation capacity relative to the NC limit state than the existing buildings. This observation refers to both masonry and RC buildings, but it is most pronounced for the masonry buildings that are assumed to be replaced by RC buildings (indicated as MSN-RC in Fig. 4), especially if considering their normalised deformation capacity.

The mean ratios between the normalised load-bearing capacities of the existing building stock and new building stock scenario ($\overline{D_U} = D_{U,existing}/D_{U,new}$) and between the normalised deformation capacities of the two building stocks ($\overline{F_U} = F_{U,existing}/F_{U,new}$), as well as the corresponding standard deviations are presented in Table 9. The ratios are presented also for the limit-state peak ground accelerations corresponding to D_Y and D_U.

Table 9 shows that the average of the ratios for all four indicators is about 0.5. This means that the deformation capacity, load-bearing capacity, and the limit-state PGA for D_Y

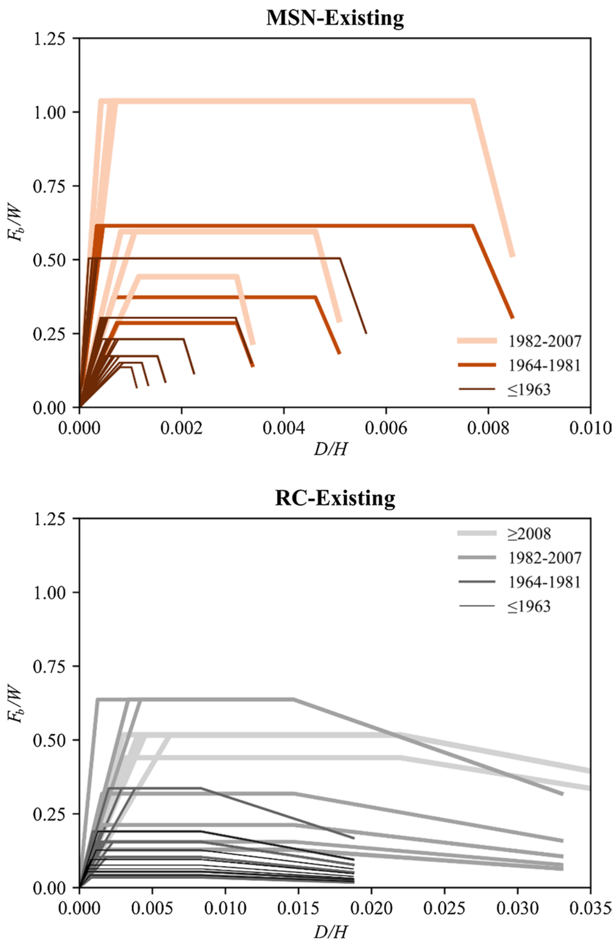

Fig. 3 Normalised pushover curves of existing building stock presented separately for MSN and RC buildings and *CY*

and D_U of the existing building stock are only one-half of what would be expected from the new building stock scenario. The standard deviation of the four parameters is between 0.25 and 0.33. The minimum $\overline{F_U}$ was observed to be about 0.1 for two seven-storey buildings constructed in 1962 and 1972. However, the minimum $\overline{D_U}$ was observed to be only 0.03 for a masonry building constructed in 1921 with six storeys above ground level.

The results in Table 9 are divided into three groups of buildings: the masonry buildings that remain masonry (MSN–MSN) in the new building stock scenario, the masonry buildings that are replaced with RC buildings in the new building stock scenario (MSN–RC) and the RC buildings that remain RC buildings in the new building stock scenario (RC–RC). All three groups were further broken down by the *CY* period. The values obtained for the four indicators are presented in Tables 10, 11 and 12. Period $CY > 2008$ is not included in these tables because the normalised performance indicators for that period are, by definition, equal to 1.

The results presented in Tables 10, 11 and 12, show that almost all the normalised performance indicators increase over the construction period. However, the model neglects the difference in the deformation capacity of the masonry buildings from the

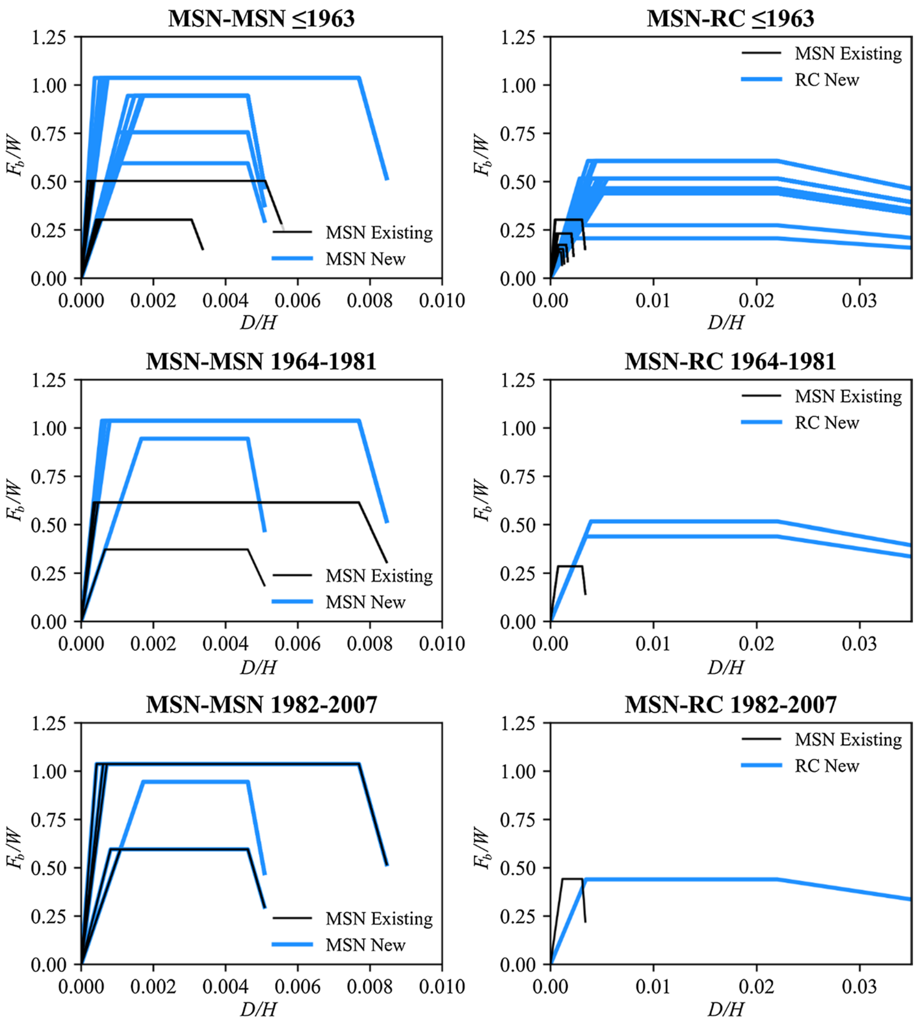

Fig. 4 Normalised pushover curves for existing MSN buildings and the corresponding pushover curves of the new building stock scenario considering MSN or RC buildings depending on the categorised CY

last two construction periods. Consequently, the $\overline{D_U}$ is constant for the construction years from 1964 and 2007 in the case of existing masonry buildings (Tables 10 and 11). The lowest $\overline{D_U}$ is observed for the masonry buildings replaced by the RC buildings (Table 11). This is partly because of the lower deformation capacity of the masonry structural elements and partly because of the difference in the building's deformation shape at the NC limit state. For RC buildings, C_U was assumed to be 1, while for masonry buildings, C_U is significantly less than 1 if there are more than three storeys. However, such an assumption may overestimate the deformation capacity of older RC structures if they have a frame structural system or frame with infills. Such buildings, which were not considered in this study, are often vulnerable to soft-storey effects.

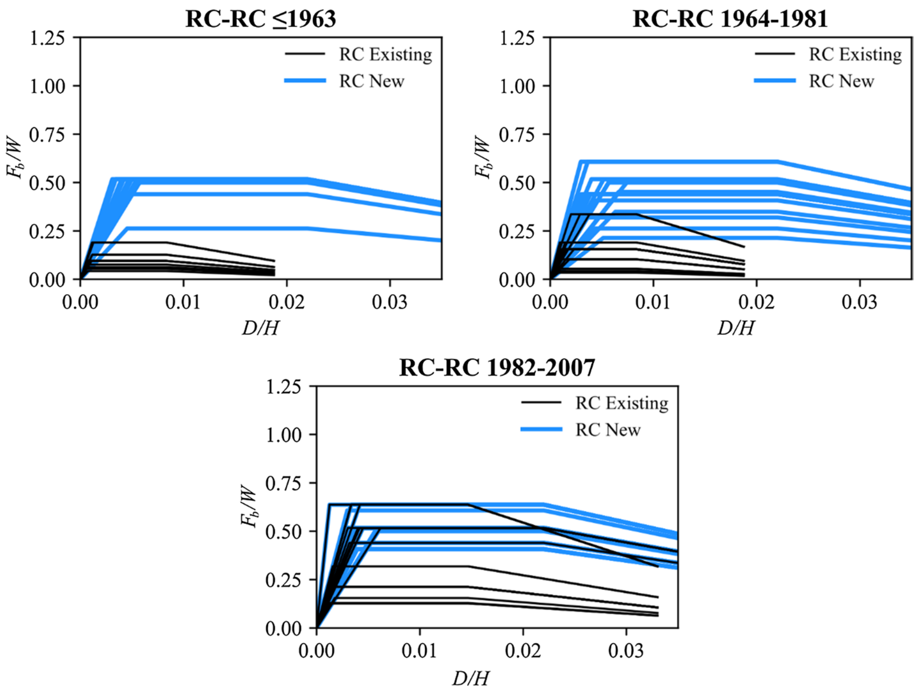

Fig. 5 Normalised pushover curves for RC buildings of the existing building stock and new building stock scenario

Table 9 Mean ratios of the normalised load-bearing capacity, normalised deformation capacity, and limit-state PGAs between existing building stock and new building stock scenario, with corresponding standard deviations

	Mean	Standard deviation
$\overline{F_U}$	0.49	0.25
$\overline{D_U}$	0.46	0.33
$\overline{PGA_{DY}}$	0.49	0.25
$\overline{PGA_{DU}}$	0.46	0.27

Table 10 Mean ratios of the normalised load-bearing capacity, normalised deformation capacity, and limit-state PGAs between existing building stock and new building stock scenario in the case of masonry buildings that remain masonry buildings (MSN–MSN) for the three construction periods

MSN–MSN	≤ 1963	1964–1981	1982–2007
	Mean	Mean	Mean
$\overline{F_U}$	0.40	0.55	0.88
$\overline{D_U}$	0.66	1.00	1.00
$\overline{PGA_{DY}}$	0.40	0.55	0.88
$\overline{PGA_{DU}}$	0.55	0.82	0.93

Table 11 Mean ratios of the normalised load-bearing capacity, normalised deformation capacity, and limit-state PGAs between existing building stock and new building stock scenario in the case of masonry buildings that are assumed to be replaced by RC buildings (MSN–RC) for the three construction periods

MSN–RC	≤1963 Mean	1964–1981 Mean	1982–2007 Mean
$\overline{F_U}$	0.46	0.60	1.01
$\overline{D_U}$	0.06	0.10	0.10
$\overline{PGA_{DY}}$	0.45	0.60	1.01
$\overline{PGA_{DU}}$	0.15	0.22	0.23

Table 12 Mean ratios of the normalised load-bearing capacity, normalised deformation capacity, and limit-state PGAs between existing building stock and new building stock scenario in the case of RC buildings that remain RC buildings (RC–RC) for the three construction periods

RC–RC	≤1963 Mean	1964–1981 Mean	1982–2007 Mean
$\overline{F_U}$	0.21	0.34	0.60
$\overline{D_U}$	0.38	0.38	0.67
$\overline{PGA_{DY}}$	0.21	0.34	0.60
$\overline{PGA_{DU}}$	0.37	0.38	0.67

Interestingly, the increment of the load-bearing capacity over time is more pronounced for the RC buildings than for the masonry buildings. On average, the load-bearing capacity of the RC buildings increased by about 40% after the implementation of Eurocode 8, as can be concluded from $\overline{F_U}$=0.60 (Table 12). The corresponding $\overline{F_U}$ for masonry buildings is 0.88 (Table 10), which means that, according to the parametric pushover curve model and the investigated building stock, the load-bearing capacity of the masonry buildings between the last two periods increased by only 12%.

The trends that were discussed for the normalised load-bearing capacity and normalised deformation capacity are reflected in the normalised limit-state peak ground accelerations. The values of $\overline{PGA_{DY}}$ are very well correlated by the $\overline{F_U}$, while the $\overline{PGA_{DU}}$ are very well correlated by the $\overline{D_U}$.

5 Conclusions

A parametric pushover curve model, which includes 12 parameters used to estimate the trilinear pushover curve of a building, is introduced. The proposed model can be applied to various types of buildings for which a trilinear pushover curve can be considered sufficiently accurate.

The accuracy of seismic performance assessment of building stock with the parametric pushover curve models depends significantly on the sub-models that are used to estimate the 12 parameters. In the current research, the parametric pushover curve model was applied only in the case of insufficient knowledge about reinforced concrete and masonry buildings in Slovenia, and used for seismic performance assessment of the University of Ljubljana building stock. It was shown that using the model for practical examples is straightforward. However, due to the uncertainty arising from the lack of knowledge about the buildings and related assumptions, the results of the case study example cannot be considered in an absolute sense but only relative to those estimated for a scenario of the new building stock. Thus the proposed model implemented in the case of a low level of building knowledge can be used for preliminary studies only aimed at raising awareness about seismic risk and making strategic actions towards seismic risk reduction.

The preliminary results of the example from the case study showed that the seismic capacity of the UL building stock is only half of that expected for the introduced scenario of the new building stock. Consequently, it can be claimed that the seismic risk of the investigated building stock is too high, which requires systematic action to improve seismic safety. The primary action should be to enhance the building stock data, perform the seismic risk assessment at a higher level of knowledge about buildings and define scenarios for seismic risk reduction.

Even though the parametrization of the pushover curve, as introduced in this paper, can be considered quite general, the implementation of the model depends on the building stock under consideration, owing to regional and time variations in the construction practice. In particular, the models for strength and deformation capacity considered for the Slovenian case study may not be directly applicable to other regions. Instead, they should be verified or adjusted before being implemented in the seismic performance assessment of building stock in other regions.

Additional research is also needed to gain insight into the accuracy of the proposed model relative to the level of knowledge about the buildings. As a result of further research, a level of knowledge about the buildings should be prescribed with respect to the purpose of the seismic performance assessment studies based on the parametric pushover curve model.

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Data availability The datasets generated and analysed during the current study are available from the corresponding author on reasonable request.

Declarations

Conflict of interest The authors declare that they have no conflict of interest.

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