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APPLICATION OF THE RISK-BASED SEISMIC DESIGN PROCEDURE TO A REINFORCED CONCRETE FRAME BUILDING

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Abstract. *Usually seismic design procedures are based on elastic analysis by using design acceleration spectrum, which implicitly takes into account the ability of inelastic energy absorption of the structural system, and capacity design principles. Thus, current standards for earthquake-resistant design of buildings do not control seismic risk to such an extent that would be acceptable for all types of structures and for all investors. Development is therefore oriented towards advanced design methods, which can be used to achieve well informed decision-making based on target reliability. In this paper, the risk-based seismic design procedure is applied in order to design a reinforced concrete building for a tolerable seismic risk. The procedure is based on the use of nonlinear methods of analysis (pushover-based method and nonlinear time history analysis). Herein the procedure for risk assessment is briefly described. It involves envelope-based pushover analysis procedure and the new closed-form solution for estimating the annual probability of exceeding selected limit states, which provide less conservative results as that proposed by Cornell. The procedure is demonstrated by designing an eight-storey RC frame building for tolerable risk. It is shown that several iterations were needed in order to fulfill the requirement of tolerated seismic risk. However, the final configuration of the structure was checked using nonlinear response history analysis, where it is shown that the envelope-based pushover analysis procedure provided a slightly conservative estimate of collapse risk. The proposed design procedure enables explicit estimation of the seismic risk and verification of the collapse mechanism, which is an advantage in comparison to the design procedure prescribed by Eurocode 8.*

1 INTRODUCTION

Seismic risk of newly designed structures is usually not calculated, since it is assumed that current standards for earthquake-resistant design of buildings (e.g. Eurocode 8 [1], ACI-318-11 [2]), which involve elastic analysis and design acceleration spectrum, guarantee that buildings designed according to their provisions are safe against collapse due to earthquakes. Therefore current standards for earthquake-resistant design of buildings do not control seismic risk to such an extent that would be acceptable for all types of structures and for all stakeholders.

Incorporation of seismic risk assessment in the process of design is difficult since assessment of risk is a complex problem which links seismic hazard analysis, vulnerability analysis of the building and socio-economic consequences that result from strong earthquakes. However, several simplified procedures have been proposed. In the simplest case, seismic risk can be communicated by means of the mean annual frequency of exceedance of a selected limit state, such as the near-collapse limit state. This information incorporates effects of all possible earthquakes that could affect the structure at a defined location and not only the design earthquake, which is the case when the structure is designed according to current standards. By comparing estimated seismic risk with an acceptable risk we can eventually decide whether the newly designed structure meets all safety requirements or not. The simplest, practice-oriented approach, combines probability assessment in closed form [3] with the pushover-based methods (e.g. [4, 5]). Herein a new pushover analysis procedure, called envelope-based pushover analysis procedure [6], is used in order to assess the building's performance. Additionally, the collapse risk is assessed by using a new closed-form formula, which takes into account the lower and the upper ground–motion intensity. It is assumed that the limit state can be exceeded only for the ground-motion intensity which is greater than the minimum ground motion intensity causing the near-collapse limit state. On the other hand it is assumed that the ground-motion intensity cannot exceed the upper-bound intensity, which can be estimated using the results of the hazard analysis. Recently it was observed by the authors of this paper that such an approach provides less conservative estimates of probability of failure as that obtained by assuming integration of the risk equation on the interval $[0,\infty)$.

Several quite comprehensive reliability-based frameworks for the design of structures were already developed. Among others, Wen [7] proposed a design procedure based on minimum lifecycle cost criteria. He concluded that there are capabilities which allow development of risk-based, comprehensive, and yet practical design procedures familiar to engineers. Similar methods for risk-based design were proposed and applied to steel frames by Liu, Wen and Burns [8] and Rojas, Pezeshk and Foley [9]. They used genetic algorithms for determination of an optimal structural configuration. Recently, a procedure for seismic design of reinforced concrete frames based on the observation of the response of structures was proposed [10]. Optimal structural behavior is achieved through redistribution of longitudinal reinforcement with the goal of uniform deformations through the height of the structure. An attempt has been made recently in order to incorporate the basic pushover-based method for risk-based seismic design of building [11]. The proposed process is iterative. The first step involves preliminary design of the structure. Then the seismic risk is estimated for the initial structural configuration and compared to an acceptable risk. If the seismic risk is too high, measures are taken to reduce it. The seismic risk is then re-evaluated for a new and improved structure.

In this paper the iterative risk-based design procedure is demonstrated by means of an example of an eight-storey reinforced concrete building. Since system failure modes of such buildings vary due to ground motions, envelope-based pushover analysis procedure was used for the estimation of the peak ground acceleration associated with the collapse of the building.

2 RISK-BASED DESIGN PROCEDURE

In this paper an iterative risk-based design procedure [11] was used to design an 8-storey building. This design procedure involves definition of the acceptable risk, which is discussed later in the paper (Section 4). The next step involves determination of an initial structural configuration, which can be achieved by using standards for earthquake-resistant design of buildings or by engineering judgment, which is also often used by experienced structural engineers. In the example presented in Section 5, a simple initial structural configuration was determined based on design of reinforcement for vertical loads with consideration of the maximum allowable axial force in columns and the design criteria associated with the minimum allowable size of elements and the amount of reinforcement. For the initial structural configuration seismic risk is estimated and compared to the predefined acceptable risk. If the seismic risk which corresponds to the initial structural configuration exceeds the acceptable risk, measures are taken to reduce it and the seismic risk is re-evaluated for a new and improved structure. In the proposed design procedure (see Fig. 1) the last three steps are repeated until the estimated seismic risk is less than the acceptable risk. As opposed to current standards, e.g. Eurocode 8, the proposed process does not involve the design earthquake. The main advantages of the proposed design procedure in comparison to that prescribed in Eurocode 8 are explicit simulation of structural damage due to earthquakes and explicit estimation of seismic risk.

In the simplest case, measures for the reduction of collapse probability ("improvements") can be based on trial and error procedure. However, guidelines can be specified for different types of buildings in order to assess the impact of variation of input parameters, such as the amount of reinforcement in columns and/or beams, on the most basic global parameters of the building (e.g. maximum base shear, global ductility, collapse mechanism), which can be used for the estimation of variation in the probability of collapse.

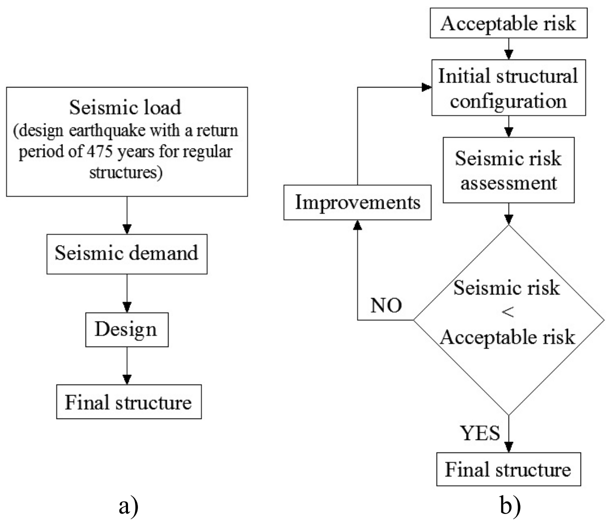


Figure 1: Flowchart showing the process of seismic design according to a) Eurocode 8 and b) proposed risk-based design procedure [11].

3 SEISMIC RISK ASSESSMENT

In the simplified approach seismic risk assessment provides estimates of the mean annual frequency (MAF) of limit-state exceedance. For the most severe limit states such as collapse, the MAF of limit state exceedance is almost equal to the annual probability of exceeding a selected limit state (*LS*), and can be determined as follows

$$P_{LS} \approx \lambda_{LS} = \int_0^\infty P(LS|IM = im) \cdot \left|\frac{\mathrm{d}H(im)}{\mathrm{d}im}\right| \cdot \mathrm{d}im\,, \tag{1}$$

where $P(LS|IM = im)$ is the fragility or probability of exceeding the limit state LS given the intensity measure (im) and $H(im)$ is the hazard, i.e. the mean annual frequency that the ground motion intensity exceeds im. If the hazard is assumed to be linear in log-log coordinates ($H=k_0 \cdot im^{-k}$) and if the fragility is expressed by means of the standard normal probability integral $\Phi[(\ln im - \ln im_{LS,50})/\beta_{LS}]$, Eq. (1) can be approximated by the simple formula 3:

$$P_{LS} \approx H(im_{LS,50}) \cdot e^{\frac{1}{2} \cdot k_{LS}^2 \beta_{LS}^2}\,, \tag{2}$$

where P_{LS} is the mean annual frequency of exceeding a given limit state, $im_{LS,50}$ is the median value of the IM -based capacity (i.e. the median ground-motion intensity which causes the given limit state), β_{LS} is its logarithmic standard deviation and k_{LS} is the slope of the hazard curve close to $im_{LS,50}$.

The MAF of limit-state exceedance according to Eq. (2) is a result of integrating the risk equation (Eq.(1)) in the range from 0 to ∞. However, this is not physically-consistent since all structures designed according to standards have a quite large collapse capacity. Therefore, there is no ground motion which would cause collapse of a structure and have the intensity measure lower than im_1. The upper integration limit im_2 also exists, since ground motions are constrained with several physical phenomena [12]. Due to these facts, the MAF assessed according to Eq.(2) can become quite conservative. Authors of this paper recently derived the closed-form solution of Eq.(1) if it is integrated in the interval [im_1, im_2]. If a lower-bound truncated lognormal distribution is assumed for the fragility function $P(LS|IM = im)$, Eq.(1) can be expressed as follows:

$$P_{LS,im12} = P_{LS} \cdot \frac{\mathrm{erf}\left[\frac{1}{\sqrt{2}\beta_{LS}}\left(k_{LS}\beta_{LS}{}^2 + \ln\frac{im_2}{im_{LS,50}}\right)\right] - \mathrm{erf}\left[\frac{1}{\sqrt{2}\beta_{LS}}\left(k_{LS}\beta_{LS}{}^2 + \ln\frac{im_1}{im_{LS,50}}\right)\right]}{1 + \mathrm{erf}\left[\frac{1}{\sqrt{2}\beta_{LS}}\ln\frac{im_{LS,50}}{im_1}\right]}, \tag{3}$$

where erf[x] is the error function and is determined as follows:

$$\mathrm{erf}\left[x\right] = \frac{2}{\sqrt{\pi}}\int_0^x \exp\left[-\tau^2\right] d\tau\,. \tag{4}$$

The fragility parameters $im_{LS,50}$ and β_{LS} can be assessed by means of different methods. The theoretically most accurate approach for assessment of fragility parameters would be based on nonlinear response history analysis. However, this type of analysis is computationally demanding. As an alternative it is possible to use pushover-based procedures which involve pushover analysis of an entire structure and seismic demand analysis using an equivalent single degree of freedom (SDOF) model, e.g. the N2 method [13]. However, this basic procedure does not take into account the impact of higher modes of vibration, which makes it less accurate for taller buildings. Several procedures [14, 15] were proposed in order to overcome this shortcoming including the extended N2 method [16]. In this study an envelope-based pushover analysis procedure (EPA) [6] was used in order to assess the fragility parameters.

The EPA procedure utilizes several pushover analyses, which are performed for invariant horizontal force distributions corresponding to the first three vibration modes, and the inelastic response history analysis of a modal-based and so-called failure based SDOF models. Each engineering demand is then obtained by enveloping results associated with the three system failure modes. Such approach enables approximate simulation of system failure modes caused by ground motions. More details about the method can be found elsewhere [6]. An overview of the method is also given in the paper No. 1451 presented at this conference.

4 ACCEPTABLE COLLAPSE PROBABILITY

In this paper the acceptable risk or target reliability is defined by the probability of collapse. Several models can be used to estimate the target reliability, but there is no consensus regarding the most appropriate model. It should be noted that in this study we distinguish between the acceptable and tolerated risk. Tolerated risk is associated with loss of human life, whereas the acceptable risk is associated with the remaining types of consequences, for example, with the collapse of the structure. Tolerated risk can be estimated by multiplying the acceptable risk with the fatality rate, which is the conditional probability of loss of life given the collapse of the structure. For ductile reinforced concrete frames, which were investigated in this study, the fatality rate amounted to 0.15 [17].

When an acceptable risk is based on the acceptable collapse probability P_C, then the acceptable probability is often expressed by the reliability index β [18]. The relationship between these quantities [18] is as follows

$$P_C = \Phi(-\beta), \tag{5}$$

where Φ is the cumulative distribution function of a standardized normal variable. The acceptable collapse probability P_C or reliability index β can obtained from codes and guidelines (e.g. [1, 19, 20]) or from other models of acceptable/tolerated risk, such as equations proposed by Allen and CIRIA [21] or by Helm's model [22] of tolerable risk. These models and corresponding equations of acceptable risk have been described elsewhere [11]. Herein Helm's model will be used to define acceptable risk (Section 5).

Helm [22] divided risk into four regions; negligible, ALARP (as low as reasonably possible) region, possibly unjustifiable and unacceptable risk, as shown in Fig. 2. If the possible number of fatalities or the number of occupants N is known and if the acceptable region of risk is chosen, the frequency of N or more fatalities, herein equated to the tolerable risk, can be obtained. The acceptable risk (probability of collapse), as defined in this study, is obtained by dividing the tolerated risk with the fatality rate.

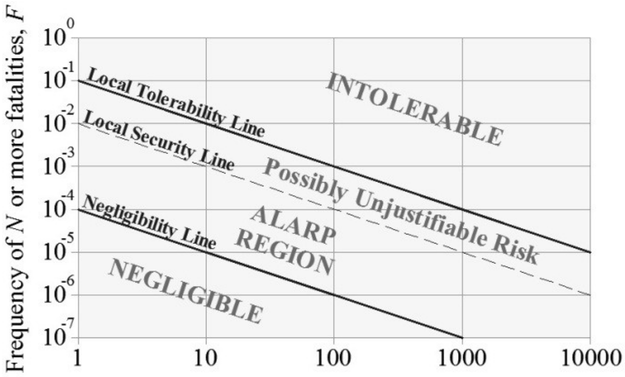


Figure 2: Helm's Frequency – Fatality curve according to [22].

5 EXAMPLE

The proposed design procedure is demonstrated by means of an 8-storey reinforced concrete building. It is assumed that the building is located in Ljubljana, i.e. in a moderate seismic region on soil type B. For illustration, the peak ground acceleration for a 475-year return period amounted 0.24 g. The initial structural configuration is based on Eurocode's provisions for minimum/maximum reinforcement ratio of the primary beams and columns corresponding to the ductility class medium. Additionally, the beams were rapidly designed for gravity loads only and the strong-column weak-beam concept was checked using the input results of the nonlinear structural model. For comparison reasons, the MAF of collapse was determined according to Eqs. (2) and (3), whereas the median peak ground acceleration at collapse $a_{g,C,50}$ and the corresponding standard deviation β_C was obtained by fitting a lognormal distribution to the sample of the $a_{g,C}$ (i.e. capacity points) determined based on the EPA procedure.

5.1 Description of the initial structure and structural model

The observed eight-storey frame building is a parking garage (Fig. 3). The height of the first and second storey is 5 m, whereas the height of other storeys is only 3.1 m, which makes this building irregular in elevation. The slab thickness is 20 cm. Concrete C30/37 and reinforcing steel S500, class B, were adopted. All columns and beams of the initial structural configuration have the same dimensions and amount of reinforcement. The only exceptions are the beams in the first and second storey, which have a greater amount of the top longitudinal reinforcement (6ϕ20) in comparison to that initially designed for other beams (4ϕ20) (see Fig. 3c and 3d). The longitudinal reinforcement in all columns amounted to 1% of the cross-section area. Stirrups were based on the criteria of the minimum concrete confinement. The exception was the columns at the base, where a greater amount of confinement is required according to Eurocode 8. In this case a 5 cm distance between the stirrups was prescribed, whereas for all other elements the distance between stirrups was 15 cm. The total mass of the building was 3856 t and the fundamental period amounted to 1.76 s.

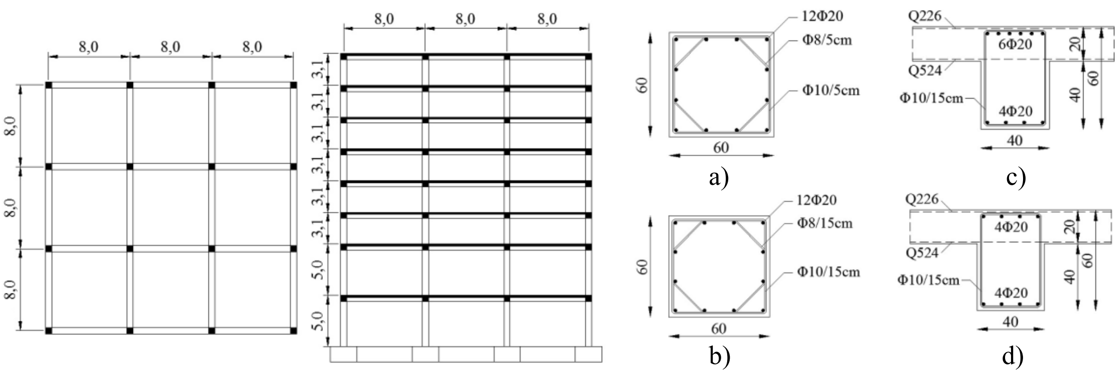


Figure 3: Plan, elevation view and reinforcement of beams and columns for the initial structural configuration of the eight-storey building.

It should be noted that the so determined initial structural configuration of the 8-storey building is practically equal to the structural configuration obtained by designing the building for the seismic action associated with a 475-year return period and behavior factor q=3.9, since the internal forces in the beams due to gravity loads are greater than those corresponding to the seismic action and since the minimum amount of longitudinal reinforcement in the columns is sufficient. Therefore a direct comparison between the structural configuration obtained by Eurocode 8 provisions and the proposed preliminary design procedure is possible.

A simplified nonlinear structural model was used, which in general follows the Eurocode 8 requirements for the modeling of structures. The beam and column flexural behaviour was therefore modelled by one-component lumped plasticity elements, composed of an elastic beam and two inelastic rotational hinges (defined by a moment–rotation relationship). The element formulation is based on the assumption of an inflexion point at the midpoint of the element. The gravity load is represented by the uniformly distributed load on the beams and by the concentrated loads at the top of the columns. For the beams, the plastic hinge is used for major axis bending only. For the columns, two independent plastic hinges for bending about the two principal axes are used. The moment–rotation relationship before strength deterioration is modelled by a bilinear relationship. A linear negative post-capping stiffness is assumed after the maximum moment is achieved. The PBEE Toolbox [23] in conjunction with OpenSees [24] was used to generate the simplified nonlinear models and to perform the nonlinear analyses, i.e. the pushover analyses of the entire building and the nonlinear response history of the equivalent SDOF models in the design phase, and the response history analysis in order to verify the final structural configuration.

5.2 Seismic hazard and ground motions

The risk-based design procedure requires the seismic hazard curve, which was obtained by using EZ-FRISK [25, 26]. This software contains a seismicity model for the central European region (Europe III). It should be noted that the seismicity model built in EZ-FRISK is not consistent with the seismicity models used for determination of the seismic hazard maps for Slovenia [27], which are used to define the seismic action according to Eurocode 8. However, according to results of the probabilistic seismic hazard analysis, the hazard parameter k=2.9 was obtained by fitting a straight line to the hazard curve in log-log coordinates with the method of least squares.

The ground-motion records were selected using an algorithm proposed by Jayaram et al. [28], which involves generation of the response spectra based on target spectrum and its variance by utilizing the Monte Carlo simulation and then selection of the records from the NGA database [29]. The minimum difference between the target spectrum and the spectrum corresponding to the selected ground motions is achieved by a greedy optimization. The target response spectrum for the response history analysis was assumed as the uniform hazard spectrum used for the design of the building, although it should be noted that hazard-consistent procedures (e.g. [30, 31]) exists, but were not applied in this study for simplicity reasons and due to the lack of the data (detailed hazard deaggregation is not available for the region of Slovenia). The ground motions were selected based on the magnitude between 5.5 and 7.5, fault distance between 5 and 50 km and shear wave velocity in the upper 30 m higher than 180 m/s.

The target uniform hazard spectrum, median response spectrum conditioned to the fundamental period of the analyzed structure and the corresponding 16th and 84th percentiles are presented in Fig. 4. In addition to the spectral acceleration at first period of the building ($S_a(T_1)$), the ground motions were conditioned also on the peak ground acceleration (a_g). In this case the median spectral acceleration of the selected ground motion records was also similar to the target spectrum. Therefore it was herein assumed that this set of ground motions is also appropriate for the case when the peak ground acceleration is selected for an intensity measure.

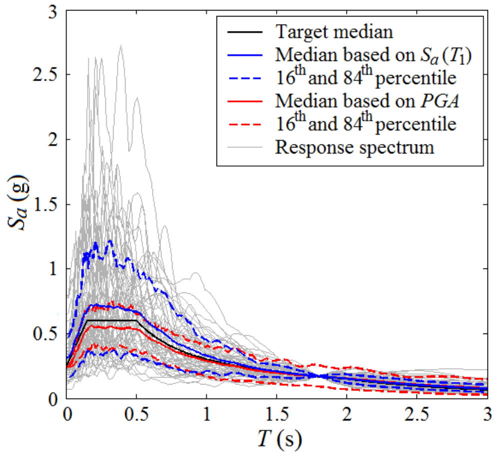


Figure 4: The target uniform hazard spectrum, spectra for each selected ground motion, the median spectrum of the selected ground motions and the corresponding 16th and 84th percentiles.

5.3 Definition of acceptable risk

Firstly, an acceptable risk was assessed using the models of ISO [19], EC0 [1], JCSS [20], CIRIA [21], Allen [21] and Helm [22]. Some models of acceptable risk (e.g. ISO, CIRIA and Allen) take into account the number of people exposed to danger. In this case we assumed that 10 people are exposed in the building at the time of an event. Additionally, a moderate cost of safety measures and small consequences were taken into account. Based on these decisions the social criterion factor K_s according to CIRIA's model was assumed equal to 0.5. For the case of Allen's model, the activity and warning factors were set, respectively, to 3.0 and 1.0. Acceptable risk according to Helm's model was evaluated based on the negligibility line (Fig. 3).

The acceptable risk according to the six models was calculated based on the above-described decisions and expressed in terms of acceptable collapse probability P_C or reliability index β (Table 1). Since the ISO standard ignores differentiation of the acceptable risk based on activities and the number of people in the building if it is lower than 100, the estimated acceptable risk in comparison to other models is quite low. Eurocode 0 does not prescribe that its model of acceptable risk should be used in the case of seismic hazard. This could be a reason that it provides a relatively low value of acceptable risk. The acceptable risk according to the model of CIRIA and Allen provided a similar value of acceptable risk as that assessed according to Eurocode 0, whereas the acceptable risk according to the JCSS model is too high according to our opinion.

Finally it was decided to design the building for the acceptable risk determined by Helm's model. The target reliability index amounted to 3.8 and the acceptable P_C for a period of 50 years was 0.33%.

It should be noted that the probability of failure, as it is defined in the various models, is often associated with the ultimate limit state, which in our study corresponded to the collapse of the building. Thus we assumed that the acceptable risk is associated to structural collapse and will be hereafter referred to as probability of collapse.

Method	ISO	EC 0	JCSS	CIRIA	Allen	Helm
β	4.4	4.2	3.7	4.0	4.3	**3.8**
P_C	$6.7 \cdot 10^{-6}$	$1.3 \cdot 10^{-5}$	$1.1 \cdot 10^{-4}$	$3.3 \cdot 10^{-5}$	$1.0 \cdot 10^{-5}$	**$6.7 \cdot 10^{-5}$**

Table 1: The acceptable P_C and reliability index β defined with different models of acceptable risk.

5.4 Description of the design procedure

The initial structural configuration was analyzed with envelope-based pushover analysis (EPA) procedure. The estimated collapse probability amounted to $2.9 \cdot 10^{-4}$ if calculated with Eq.(2), which exceeded the acceptable collapse probability by a factor of 4.3. In order to assess the probability of collapse using Eq.(3), the maximum intensity a_{g2} has to be estimated. From a theoretical point of view, the maximum peak ground acceleration can be estimated based on the seismicity model used in hazard analysis and the ground motion prediction model, which are both uncertain, especially in regions of moderate and low seismicity, where the recorded ground motions are rare. Additionally, the truncation of the ground-motion probability is required in order to theoretically assess the upper bound ground-motion intensity. Therefore it is suggested to use different ground-motion prediction (GMP) equations for a given truncation level (e.g. 2 or 3 standard deviations above the mean value) in order to assess the a_{g2}. For the purpose of this study it was assumed that the worst-case scenario is an earthquake with magnitude 7, which is consistent with the seismic hazard analysis for Slovenia [27], and the source-to-site distance R_{jb}=0 km. The truncation level was assumed equal to 2.5. Based on these assumptions, the a_{g2} was calculated for the GMP models proposed by Sabetta and Pugliese [32], Bindi et al. [33], Akkar and Bommer [34] and Peruš and Fajfar [35]. Finally, the a_{g2} was assessed by assuming equal weights for the four GMP models, and amounted to 2.4 g. This is a very large level of peak ground acceleration for moderate seismicity. However, the collapse probability of the initial structural configuration estimated according to Eq.(3) amounted to $2.1 \cdot 10^{-4}$, which is around 40% smaller than that calculated according to Eq.(2), but it still exceeds the acceptable collapse probability by a factor of 3.1. This result indicates that the difference between the collapse probabilities calculated with both equations is not negligible, although the a_{g2} is very high. This confirms that the assessment of the probability of collapse according to Eq.(2) is most probably too conservative.

The initial (first) structural configuration was modified, since the probability of collapse for the initial structural configuration exceeded the acceptable probability of collapse. Decision regarding the improvement of the structural configurations was based on knowledge obtained from a previously performed sensitivity study on similar reinforced concrete frame buildings [11]. Based on this study it was decided that for the 'second' structural configuration the amount of longitudinal reinforcement of the columns in the first four stories was increased for 0.2% of the columns cross-section area. However, additional iterations were needed as presented in Table 2, since the system failure modes observed in the pushover analysis did not provide sufficient system ductility. Note that the label AsC (AsB) refers to the increase of the amount of reinforcement in columns (beams) and the label AcC (AcB) refers to the increase of the cross-section area of columns (beams) for a certain percentage. T_1 is the fundamental period of vibration, $a_{g,C,50}$ is the median ground acceleration causing the collapse of the structure obtained by the EPA procedure, β_C is the lognormal standard deviation and a_{g1} is the minimum observed ground acceleration causing collapse.

In Table 3 the probability of collapse calculated according to Eqs. (2) and (3) is presented for all iterations within the design process. It can be observed that Helm's condition of acceptable risk was satisfied with the fifth iteration. In this specific case the use of Eq.(3) for the determination of the probability of collapse did not affect the final structural configuration.

The pushover curves and the approximate median IDA curves based on EPA procedure are presented in Fig. 5 in order to better understand how the adopted structural modifications affected the strength and the system ductility of the building and eventually how this affected the $a_{g,C,50}$. It can be seen that the base shear, system ductility and the $a_{g,C,50}$ are increased after each structural modification. The largest increment in base shear and ductility can be observed for the fifth iteration, which corresponds to an increase of the beams' height in the most damaged storeys. A significant increase of ductility can also be observed in the fourth step of the design procedure, where the area of the cross-section of columns was increased. Consequently the largest increment of the a_{g1} and $a_{g,C,50}$ was also observed for the 4th and 5th iteration. In these cases the system failure mode provided significantly largest system ductility in comparison to that observed for the initial structural configuration. In Fig. 6 the deformation shape and damage pattern corresponding to the near-collapse limit state is presented for the structure designed according to Eurocode 8 (i.e. initial structural configuration) and for the final structural configuration obtained with the proposed design procedure. It can be seen that for the structure designed according to Eurocode 8 storey drifts are concentrated in the first and second storey, which was prevented with the proposed design process.

Iteration	Modification	T_1 (s)	$a_{g,C,50}$ (g)	β_C	a_{g1} (g)
1	Initial structural configuration	1.76	1.13	0.68	0.20
2	0.2% AsC first 4 storeys	1.76	1.17	0.68	0.21
3	0.2% AsC first 4 storeys	1.76	1.20	0.67	0.23
4	10% AcC first 4 storeys	1.69	1.44	0.64	0.31
5	5% AcB first storey	1.73	1.56	0.59	0.38

Table 2: Brief description of the safety measures for the five structural configurations, the corresponding first vibration period, parameters of the fragility function and the minimum intensity a_{g1} which causes collapse of the structure.

Iteration	$P_C \cdot 10^{-4}$	$P_{C,1}/P_{C,i}$	$P_{C,im12} \cdot 10^{-4}$	$P_{C,im12,1}/P_{C,im12,i}$
1	2.92	1.0	2.09	1.0
2	2.59	1.1	1.85	1.1
3	2.30	1.3	1.63	1.3
4	1.11	2.6	0.78	2.7
5	**0.65**	4.5	**0.49**	4.2

Table 3: Probability of collapse assessed according to Eqs.(2) and (3) for each structural configuration within the design process.

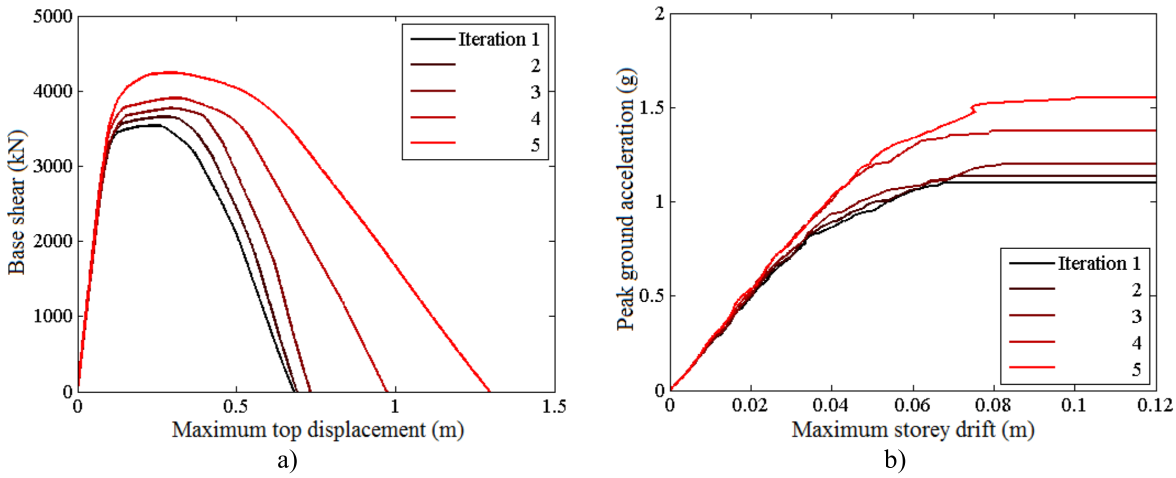
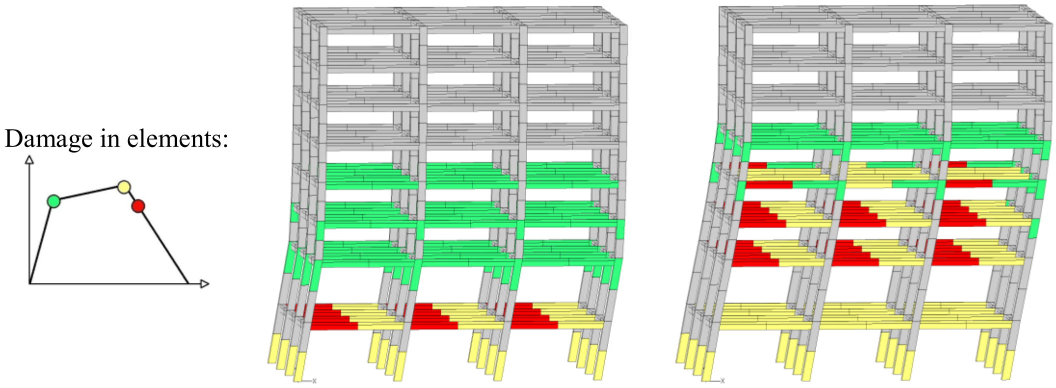


Figure 5: a) Pushover curves for each structural configuration and b) the corresponding approximate median IDA curves based on EPA procedure.



a) b)

Figure 6: Deformation shape and damage pattern corresponding to the near-collapse limit state for the building designed according to a) Eurocode 8 provisions (i.e. initial structural configuration) and b) the final structural configuration (iteration 5).

In order to check the adequacy of the final structural configuration the collapse capacity was assessed using response history analysis for a set of ground motions. The median collapse capacity based on the response history analysis was similar to that provided by the envelope-based pushover analysis procedure (Fig. 7). However, the result obtained by the EPA procedure was slightly conservative. For comparison reasons, the median maximum storey drift ration obtained by taking into account only the modal-based SDOF model corresponding to the first-mode pushover analysis (PA1) is presented in Fig. 7. It can be observed that for the first three iterations higher modes (EPA) have no impact $a_{g,C,50}$ due to the predominant impact of soft storey mechanism that causes collapse. However, for the final structural configuration, where the soft story mechanism is prevented (Fig. 6), a greater impact of the higher modes of vibration can be observed since the difference between the response based on EPA and PA1 gradually increases. From Fig. 7 it can be observed that the median collapse capacity $a_{g,C,50}$ based on the basic pushover-based method is significantly overestimated.

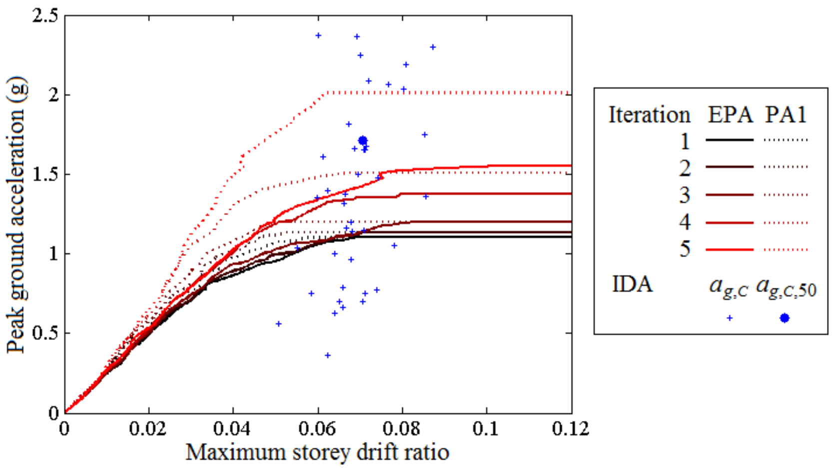


Figure 7: The relationship between the peak ground acceleration and the median value of the maximum storey drift obtained with EPA and the basic pushover-based method (PA1) for all five structural configurations, and the collapse capacity for the final structural configuration obtained by response history analysis.

6 CONCLUSIONS

In the paper an iterative design procedure based on acceptable risk was described and demonstrated by means of an example of a reinforced concrete frame building. The procedure was first presented in [11], where the basic pushover-based method was used to assess structural performance. In this paper structural performance was assessed by using the recently introduced EPA procedure, whereas the probability of collapse was calculated with consideration of the lower and the upper integration limit of the risk equation. Such an approach provided more accurate estimation of the collapse risk due to less conservative assessment of risk and due to more accurate estimation of the $a_{g,C,50}$, since the EPA procedure enables approximate simulation of system failure modes, which are significantly different to that corresponding to the first-mode pushover analysis.

The risk-based design offers the possibility to overcome shortcomings of the standards for earthquake resistant design of structures in order to achieve well-informed decision-making, which is a key element for the future protection of the built environment against earthquakes. However, further research is needed in order to adequately address some underdeveloped components of the proposed design process. Additionally, development of software is also required, which would eventually enable practical applications of the proposed design process.

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