Risk-based seismic design – An alternative to current standards for earthquake-resistant design of buildings

N. Lazar & M. Dolšek
University of Ljubljana, Slovenia

SUMMARY:
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. This can be achieved with the proposed concept for risk-based design of buildings, which requires use of nonlinear methods of analysis and risk assessment, whereas decision on the adequate structural configuration depends on the definition of acceptable risk. Despite the fact that the development of the design concept is in the initial phase, an eight- and a fifteen-storey reinforced concrete building were designed according to the proposed procedure by using PBEE Toolbox in conjunction with OpenSEES. It is shown that estimated collapse probability for the final structural configurations is slightly smaller than the acceptable probability of collapse, which was not satisfied for buildings designed according to Eurocode 8.

Keywords: risk-based design, probability of collapse, seismic risk, reinforced concrete frame, pushover analysis

1. INTRODUCTION

Current standards for earthquake-resistant design of buildings (e.g. Eurocode 8 (CEN, 2004), ACI-318-11, (ACI, 2011)) prescribe that buildings should be designed to withstand a design seismic action, which is defined for an earthquake recurrence interval associated with a limit state of interest, such as damage limitation or near-collapse limit state. Usually design procedures involve elastic analysis method and design acceleration spectrum, which implicitly takes into account the ability of inelastic energy absorption of the structural system. Thus, seismic risk of newly designed structures is only implicitly controlled through the q-factor (R-factor) concept and capacity design procedure. 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. This issue can be solved by adequate estimation of seismic risk in the design process of a building, which would probably be the best approach for mitigation of earthquake losses in the future.

In the simple approach, the seismic risk can be described by 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. By comparing estimated seismic risk with an acceptable or tolerated risk, as defined later on in the paper, we can eventually decide whether the newly designed structure met all safety requirements or not. Seismic risk assessment is a complex problem that links seismic hazard analysis, vulnerability analysis of the building and socio-economic consequences that result from strong earthquakes. Therefore several simplified procedures have been proposed. The simplest, practice-oriented approach, combines probability assessment in closed form (Cornell 1996, Cornell et al., 2002) with the pushover-based methods (e.g. Dolšek and Fajfar 2007, Fajfar and Dolšek 2012).

Several reliability-based frameworks for design of structures were proposed. Wen (2001) 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 (2004) and Rojas, Pezeshk and Foley (2011). 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 (Hajirasouliha et al., 2011). Optimal structural behaviour is achieved through redistribution of longitudinal reinforcement with the goal of uniform deformations through the height of the structure.

In the paper we present a simplified procedure for design of buildings based on acceptable risk. The process is iterative and requires the use of nonlinear methods of analysis. The first step of the proposed procedure involves preliminary design of the structure. Then the seismic risk is estimated 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. The proposed design procedure is demonstrated by an example of an eight- and a fifteen-storey reinforced concrete building.

2. PROPOSED DESIGN PROCEDURE

The design procedure proposed in this paper is iterative. The first step involves decision associated with the acceptable risk or structural reliability, which can be adopted based on models of acceptable risk (Section 4). The next step involves preliminary design of the structure. It 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. For example, a simple initial structural configuration can be 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. Once the initial structural configuration is defined, seismic risk is estimated and compared to the acceptable risk. If it is too high, measures are taken to reduce it. The seismic risk is then re-evaluated for a new and improved structure. In the proposed design procedure (see Fig. 2.1) the last three steps are repeated until the estimated seismic risk is less than the acceptable risk. The proposed process does not involve the design earthquake, which is the case of the usual design according to standards, e.g. Eurocode 8 (see Fig. 2.1). The main advantage of the proposed design procedure in comparison to that prescribed in Eurocode 8 are more accurate prediction of structural behavior during earthquakes and the explicit estimation of seismic risk.

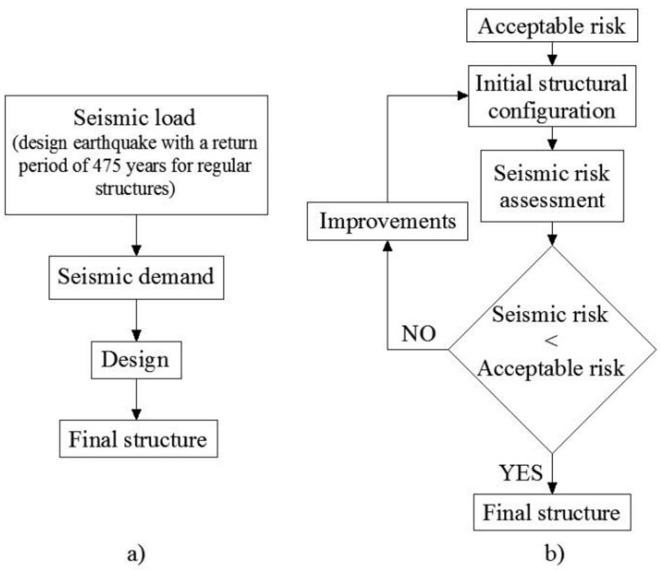


Figure 2.1. Flowchart showing the process of seismic design according to a) Eurocode 8 and b) proposed risk-based design procedure.

In the simplest case, measures for reduction of collapse probability ("Improvements") can be based on trial an 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 estimation of the variation in the probability of collapse.

A sensitivity study was performed based on similar buildings as designed herein in order to define simple guidelines for improvement of the structural configuration, which can be used in combination with the capacity design concept prescribed in standards. Eurocode 8 was used for determination of the basic structural configuration of examined buildings. Among other things, we found, that the increase of longitudinal reinforcement in columns had in most cases beneficial effects on the probability of collapse. Probability of collapse was efficiently reduced if the amount of reinforcement of columns was increased in more than 40% of the storeys counted from the first to the top storey (Fig. 2.2). The opposite effect was observed if the amount of reinforcement was increased only in the beams in a certain part of the building, since such a measure does not have beneficial effect on the global ductility. Thus the increase of the strength of beams should be controlled by the strength of columns as it is treated in the capacity design procedure.

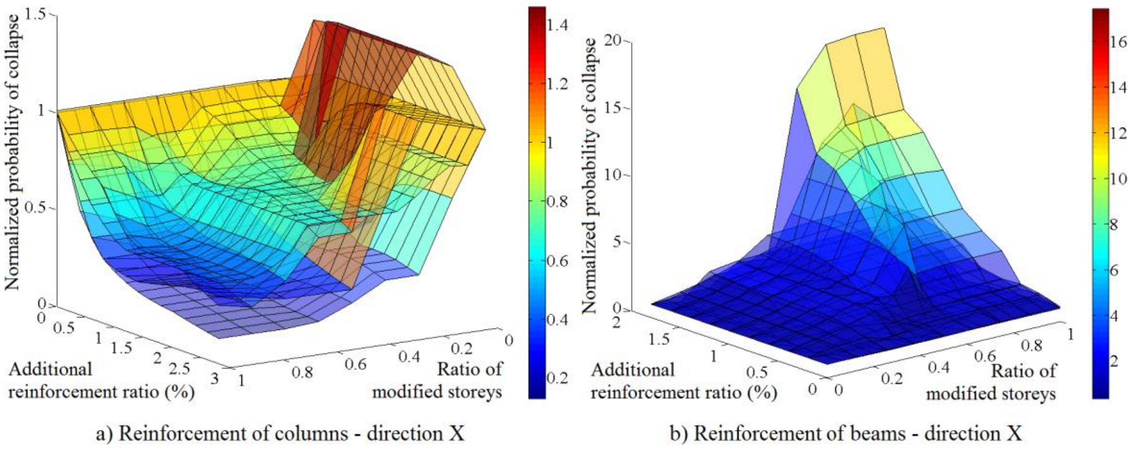


Figure 2.2. Normalized probability of collapse as a function of the ratio of modified storeys (from bottom to top) and the increase of reinforcement ratio of a) columns and b) beams. Surfaces correspond to buildings which had a different number of storeys.

3. SEISMIC RISK ASSESSMENT

In its simplest form, seismic risk assessment can provide estimates of the annual probability of exceeding a selected limit state (LS), e.g. collapse, and can be determined as follows

$$P_f = \int_0^\infty P(LS|I_M = i_m) \cdot \left|\frac{dH(i_m)}{di_m}\right| \cdot di_m, \tag{3.1}$$

where $P(LS|I_M = i_m)$ is the fragility or probability of exceeding the limit state LS when the intensity measure I_M is equal to a value i_m and $H(i_m)$ is the hazard i.e. the mean annual frequency that the ground motion intensity will exceed i_m. Since the occurrence of significant earthquakes can be described by a Poisson process, the hazard of the intensity measure can be modelled with the expression $H(i_m) = 1 - e^{-(i_m/\lambda)^{-k}}$. If the hazard is assumed to be linear in log-log coordinates and denoted as $k_0 \cdot i_m^{-k}$, and if the fragility is expressed by means of the standard normal probability integral $\Phi[(\ln i_m - \ln i_{m50,LS})/\beta_{LS}]$, Eqn. 3.1 can be approximated by the simple closed-form solution (Cornell, 1996):

$$P_f \approx H(i_{m50,LS}) \cdot e^{\frac{1}{2} \cdot k_{LS}^2 \cdot \beta_{LS}^2}, \tag{3.2}$$

where P_f is the mean annual frequency of exceeding a given limit state, $i_{m50,LS}$ is the median value of the I_M - based capacity (i.e. the median intensity measure 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 $i_{m50,LS}$.

The fragility parameters $i_{m50,LS}$ and β_{LS} can be determined by means of different methods. The choice of nonlinear dynamic analysis is, among the other problems, still the subject of available computational resources and time, especially if it is used for design purposes. Thus the $i_{m50,LS}$ was herein estimated by the N2 method (Fajfar, 2000), whereas β_{LS} was assumed from previous studies.

4. ACCEPTABLE PROBABILITY OF COLLAPSE

The decision regarding the acceptable risk or target reliability, which is herein defined by a certain level of the probability of collapse, can be adopted based on models of acceptable risk, although there is no consensus on an acceptable seismic risk yet. Note that we distinguish between the acceptable and tolerated risk. Tolerated risk is associated with loss of human life, whereas acceptable risk is associated with the remaining types of consequences, for example, with the collapse of the structure. The tolerated risk (herein expressed by the mean annual probability of loss of life) is lower than the acceptable risk. It can be estimated by multiplying the acceptable risk and the fatality rate, which is the conditional probability of loss of life given the collapse of structure. For ductile reinforced concrete frames, which were investigated in this study, the fatality rate amounted to 0.15 (Jaiswal, Ward, 2010).

In the simplest case, an acceptable risk is defined with the probability of failure P_f, which can be expressed by the reliability index β. The relationship between these quantities (Melchers, 1999) is as follows

$$P_f = \Phi(-\beta), \tag{4.1}$$

where Φ is the cumulative distribution function of a standardized normal variable. The acceptable probability of failure P_f or reliability index β obtained from codes and guidelines (ISO ISO2394, 1998; CEN, 2004; JCSS, 2000) or from other models of acceptable/tolerated risk, such as equations proposed by Allen and CIRIA (Bhattacharya et al., 2001) or by Helm's model (1996) of the tolerable risk.

The ISO standard (1998) in Appendix E provides two methods to determine the target reliability. The first method takes into account loss of human life and provides a tolerated risk to individual safety and safety of a large number of people. Another way to determine target reliability, which was not used in our study, defines acceptable risk given the financial consequences. Since the determination of target reliability by the aforementioned two methods is not always possible, the standard states also the reliability index β for the lifetime of the structure, which for buildings and other normal structures amounts to 50 years. Target reliability index depends on the costs of safety measures and consequences of failure.

The target reliability provided in Eurocode 0 (CEN, 2004) is based on three consequence classes. It is defined for several limit states and two reference periods (1 year and 50 years). Determination of acceptable risk is described in detail in Appendix B, where a minimum reliability index β for a given reference period is defined. In a similar manner, the minimum reliability index for the ultimate limit state is defined by the Probabilistic model code (JCCS, 2000). The target reliability is differentiated based on relative costs of safety measures and consequences of failure. Note that target reliability index according to Eurocode 8 is often larger than that defined in the Probabilistic model code.

The Construction Industry Research and Information Association (CIRIA) developed an empirical equation for tolerated annual probability of failure (Eqn. 4.2)

$$P_f = \frac{K_s}{n_r} p' / \text{year}, \tag{4.2}$$

where p' is the accepted individual annual probability of death with the usual value of 10^{-4}, K_s is a social criterion factor, which depends on the willingness of individual risk and n_r is the resistance factor or number of people at risk. Melchers (1999) provided various values of the factor K_s. It can be seen that, by increasing the number of exposed people, acceptable probability linearly decreases.

In 1981 Allen proposed an equation to calculate the acceptable annual probability of failure

$$P_f = \frac{A}{W\sqrt{n_r}} 10^{-5} / \text{year}, \tag{4.3}$$

where n_r is the resistance factor, i.e. the number of people at risk, A is the activity factor defined by the use of structures and W is the warning factor that depends on the type and visibility of failure. A table of factors A and W can be found elsewhere (e.g. Bhattacharya et al., 2001).

Helm (1996) defined tolerable risk based on the relationship between the number of fatalities N and frequency of N or more fatalities. Risk is divided into four regions; negligible, the ALARP (as low as reasonably possible) region, possibly unjustifiable and unacceptable, as shown on Fig. 4.1.

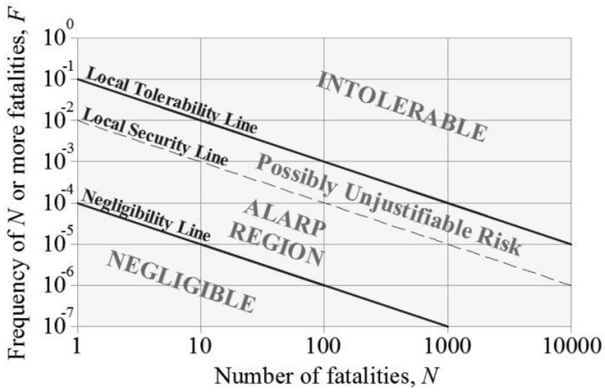


Figure 4.1. Helm's Frequency – Fatality curve

5. EXAMPLE

The proposed design procedure is illustrated by means of two examples of reinforced concrete buildings. Beams of these buildings were preliminary designed for vertical load with consideration of criteria for minimum/maximum reinforcement ratio of the primary beams designed for ductility class medium, whereas columns of initial structural configuration were based on strong-column weak-beam concept by taking into account the Eurocode's criteria of maximum normalized axial load and minimum/maximum reinforcement ratio of cross-section of columns. The MAF of collapse was determined according to Eqn. 3.2, whereas $i_{m,C}$ corresponded to the peak ground acceleration $a_{g,C}$. Note that the median collapse intensity $a_{g,C}$ was assumed 1.2 times the near-collapse limit-state intensity (Brozovič and Dolšek 2011), which was estimated by using the N2 method (Fajfar, 2000). The dispersion measures for randomness (β_{RC}) and uncertainty (β_{UC}) in $a_{g,C}$ were assumed equal to 0.4 and 0.3, respectively, providing a total dispersion β_C=0.5, whereas parameters of hazard (k=3.8 and k_0=2.06·10^{-5}) were determined based on the hazard maps for the location of Ljubljana, Slovenia, and soil type B.

The frame structures were modeled by using PBEE Toolbox (Dolšek, 2010) and analyzed in conjunction with OpenSEES (2009). Modal lateral loads for pushover analyses were applied at the center of mass with positive and negative sings in two horizontal directions. P-Δ effects were taken into account. The nonlinearity was modeled in plastic hinges at both ends of all elements by a tri-linear moment-rotation relationship (Dolšek, 2010). The near-collapse limit state was defined in the softening range and corresponded to 80% of the maximum base shear.

5.1 Description of initial structural configurations

The eight-storey frame building is a parking garage (G8) (Fig. 5.1), whereas the fifteen-storey frame is a residential building (R15) (Fig. 5.2). The height of the first and second storey of the building G8 is 5 m, which makes this building irregular in elevation, since the height of other storeys is only 3.1. For the other building the first storey height is 4 m, whereas other storeys are 3 m high. The slab thickness is 20 and 22 cm, respectively, for building G8 and R15. Concrete C30/37 and reinforcing steel S500, class B, were assumed for both buildings. All columns and beams of the initial structural configurations had the same dimensions and amount of reinforcement (Fig. 5.1 and 5.2). The only exception were the beams from the third to eighth storey of the building G8, where the top longitudinal reinforcement was decreased from 6ϕ20 to 4ϕ20. 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 total mass of the building was 3856 t and 10736 t, respectively, for building G8 and R15. The fundamental period of buildings G8 and R15 was quite high and amounted, respectively, 1.76 s and 2.32 s.

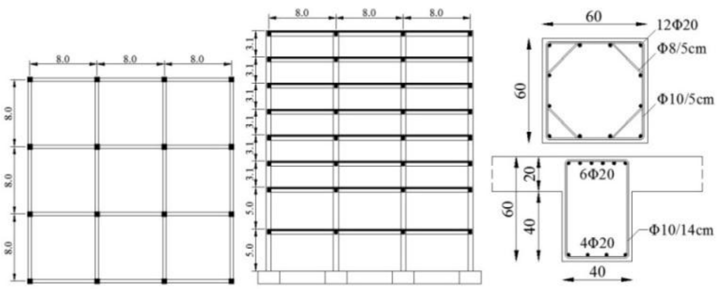


Figure 5.1. Plan, elevation view and reinforcement of beams and columns for the initial design of the eight-storey parking garage.

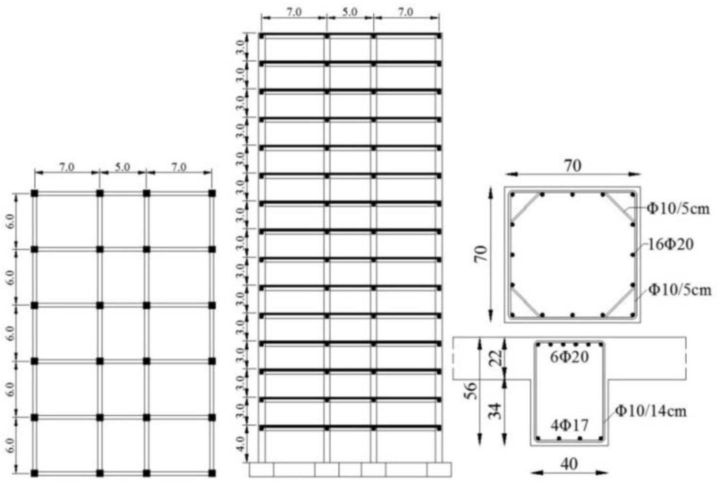


Figure 5.2. Plan, elevation view and reinforcement of beams and columns for the initial design of the fifteen-storey residential building.

5.2 Definition of acceptable risk

The decision regarding acceptable risk was made on the basis of models described in Section 4. In the case when the model of acceptable risk takes into account the number of people exposed to danger, we assumed that an average of 10 people were exposed in the building G8 and 80 people in the residential building R15. When defining acceptable risk, moderate cost of safety measures was taken into account for both buildings, whereas small and moderate consequences were taken into account, respectively, for building G8 and R15. Based on these decisions the social criterion factor K_s (CIRIA's model, see Eqn. 4.2) was assumed 0.5 and 0.05, respectively, for building G8 and R15. For the case of Allen's model, the activity and warning factors were set, respectively, to 3.0 and 1.0 for building G8, whereas for the building R15 both factors were assumed equal to 1.0. Risk according to Helm's model was evaluated based on the negligibility line (Fig. 4.1).

So determined acceptable risk expressed in terms of acceptable probability of failure P_f or reliability index β is presented in Table 5.1. It would be expected that the acceptable P_f can be higher for the building G8. This was not observed in the case of the ISO standard, which ignores differentiation of the acceptable risk based on activities and the number of people in the building. According to our opinion, the acceptable risk based on the CIRIA's model for the R15 building is too low, whereas that of the JCSS model is too high. Eventually we decided to design the two buildings for the acceptable risk based on Helm's model. Target reliability indexes were 3.8 and 4.3, respectively, for building G8 and R15. Thus the acceptable P_f of the residential building for a period of 50 years was 0.04%, whereas for the parking garage was about eight times larger (0.33%).

Table 5.1. The acceptable P_f and corresponding reliability index β defined for buildings G8 and R15 different models of acceptable risk.

Method	ISO	EC 0	JCSS	CIRIA	Allen	Helm
Building G8						
β	4.4	4.2	3.7	4.0	4.3	3.8
P_f	$6.7 \cdot 10^{-6}$	$1.3 \cdot 10^{-5}$	$1.1 \cdot 10^{-4}$	$3.3 \cdot 10^{-5}$	$1.0 \cdot 10^{-5}$	$6.7 \cdot 10^{-5}$
Building R15						
β	4.4	4.7	4.2	4.9	4.7	4.3
P_f	$6.7 \cdot 10^{-6}$	$1.3 \cdot 10^{-6}$	$1.3 \cdot 10^{-5}$	$4.2 \cdot 10^{-7}$	$1.1 \cdot 10^{-6}$	$8.3 \cdot 10^{-6}$

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 building. Thus we assumed that the acceptable risk is associated to structural collapse and will be hereafter referred to as probability of collapse.

5.3 Analysis and design procedure

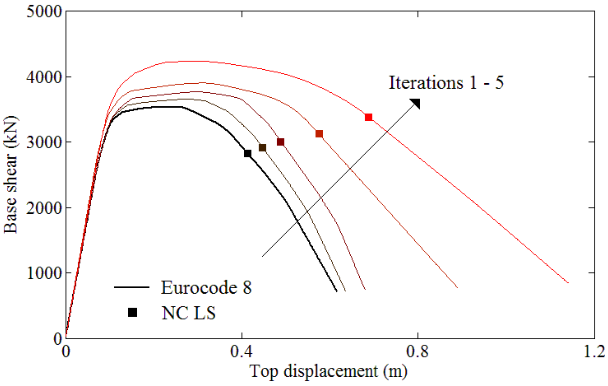
Estimated collapse probability for the initial structural configurations of buildings G8 and R15 (Section 5.1), amounted to $4.24 \cdot 10^{-4}$ and $5.14 \cdot 10^{-5}$, which exceeded the acceptable collapse probability by a factor of 6.3 and 6.2. Decision regarding the improvement of the structural configurations based on knowledge obtained from the previously performed sensitivity study on similar buildings (Section 2). Thus, in the first step, we increased the reinforcement or the size of columns in the bottom half of the building. Further improvements were made according to the damage distribution in the structure. Where a soft-storey mechanism was likely to occur, the columns were strengthened and if the damage was concentrated in beams in only a few storeys, the cross-section height of these beams was increased. A capacity design rule of strong-column and week-beam was satisfied for each structural configuration.

In Table 5.2 the maximum probability of collapse estimated with analyses for two principal directions and risk mitigation measures are presented for each iteration of the design procedure. The label Nst indicates the number of modified storeys counted from the bottom storey, whereas AsC (AsB) and AcC (AcB) refers to increase of the amount of reinforcement and area of the cross-section of columns (beams) for a certain percentage, respectively. It can be observed that in the last iteration, the P_f of buildings G8 and R15 was decreased, respectively, by factors 6.6 and 6.3 if compared to the P_f corresponding to initial structural configuration. The Helm's condition of acceptable risk was satisfied with the fifth iteration for building G8 and ninth iteration for building R15.

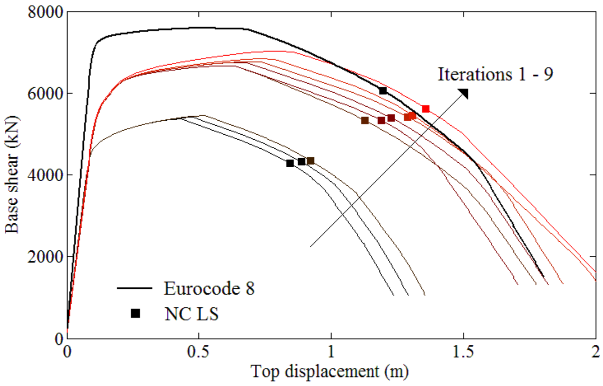
In order to better understand the impact of improvements on the seismic response of the two structures, pushover curves of all structural configurations are presented in Fig. 5.3 and compared to the pushover curves of the structures designed according to Eurocode 8. It can be seen that the base shear and global ductility of both buildings are increased after each structural modification. The largest increment in base shear and ductility can be observed in the fifth iteration for building G8 and fourth iteration for building R15. These iterations corresponded 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 for building G8, where the cross-section of columns was increased. Both types of modification resulted in a more uniform distribution of damage over the height of buildings.

Table 5.2. Probability of collapse in each step of the proposed design procedure

Building G8				Building R15			
Iteration	$P_f \cdot 10^{-4}$	$P_{f,1}/P_{f,i}$	Modification	Iteration	$P_f \cdot 10^{-5}$	$P_{f,1}/P_{f,i}$	Modification
1	4.24	1.00	Initial str. conf.	1	5.14	1.00	Initial str. conf.
2	3.28	1.29	4st-AsC0.2%	2	4.22	1.22	8st-AsC-0.2%
3	2.45	1.73	4st-AsC0.2%	3	3.67	1.40	8st-AsC-0.4%
4	1.21	3.50	4st-AcC10%	4	1.62	3.17	5st-AcB-5%
5	0.64	6.63	1st-AcB5%	5	1.32	3.89	12st-AsC-0.4%
				6	1.18	4.36	1st-AcC-10%
				7	0.98	5.24	12st-AsC-0.2%
				8	0.93	5.53	1st-AcC-20%
				9	0.81	6.34	3st-AsC-0.2% 3st-AcC-10%



a) Building G8



b) Building R15

Figure 5.3. Pushover curves for all structural configurations of a) the building G8 and b) the building R15. Points indicating near-collapse limit state and pushover curves of the structures designed according to Eurocode 8 are also presented.

In Table 5.3 global characteristics of structures designed according to the proposed procedure and Eurocode 8 are compared. It should be noted that the initial structural configuration of building G8 (design iteration 1) is the same as that designed according to Eurocode 8, since reinforcement of the columns and beams was controlled by the minimum requirements of cross-section reinforcement ratio and by the vertical load, respectively. The structural mass and fundamental period of two designs of the building G8 are almost the same, but the estimated near-collapse intensity $a_{g,NC}$ of the building designed according to the proposed procedure is significantly larger than $a_{g,NC}$ of the structure based on Eurocode 8. Consequently, the estimated P_f of the building based on Eurocode 8 is significantly larger (6.6 times) in comparison to that estimated for the building based on the iterative design process. Such large difference was not observed for the fifteen-storey building. The final structural configuration of R15 is more flexible than that based on the Eurocode 8, but the $a_{g,NC}$ is about 20% larger. The deformation shape and damage corresponding to the near-collapse limit state of the building R15 were observed similar for both design procedures (Fig. 5.4 c) and d)). However, for the building G8, which is irregular in elevation, the plastic mechanism was not adequate, if the building was designed according to Eurocode 8, although capacity design rules were applied. For the interested reader movies showing the progression of damage for the four variants of buildings presented in Fig. 5.4 are available on YouTube.

Table 5.3. Comparison between selected parameters of structures, which were designed according to proposed procedure and Eurocode 8

	Design procedure	Mass	T_1	F_b/W	$a_{g,NC}$	$P_f \cdot 10^{-5}$
Building G8	Eurocode 8	3856 t	1.76 s	9.4 %	0.60 g	42.4
	Proposed	3897 t	1.73 s	11.1 %	1.00 g	6.4
Building R15	Eurocode 8	11853 t	1.96 s	6.5 %	1.43 g	1.60
	Proposed	10820 t	2.38 s	6.6 %	1.71 g	0.81

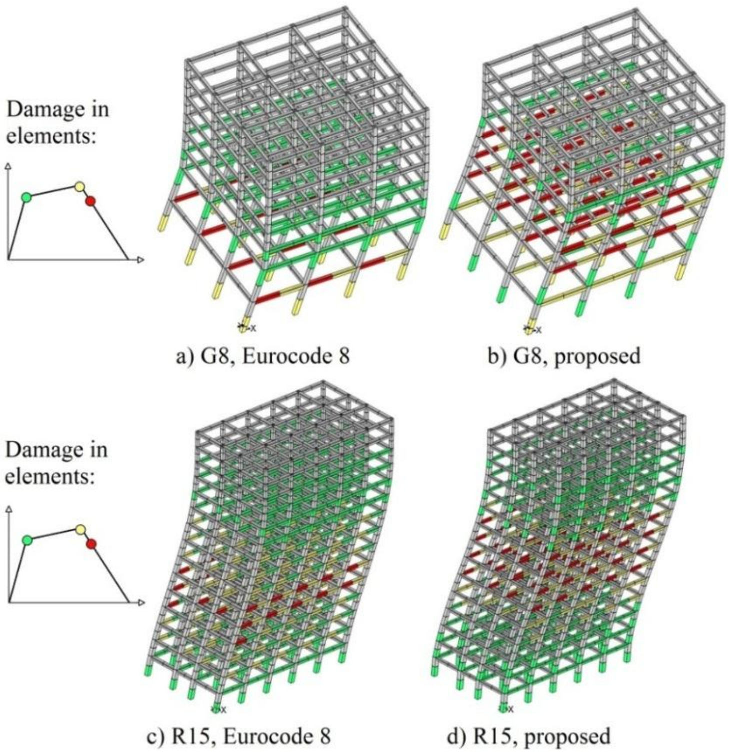


Figure 5.3. The deformation shape and damage distribution at the near-collapse limit state for buildings designed according to the risk-based and Eurocode 8 design procedures.

6. CONLUSIONS

In the paper the concept of the iterative design procedure based on acceptable risk was proposed. In this initial stage of development, it has several limitations. For example, its efficiency was demonstrated by using pushover analysis, which is an approximate method. It is also necessary to develop clearer guidelines for improving structural configuration aiming at satisfying the condition of acceptable risk with a reasonable number of iterations. In addition, acceptable risk should be defined based on different criteria and not only by the acceptable probability of collapse, which is still subject for debate. However, the concept of risk-based design has advantages in comparison to the classic concept of capacity-based design. For example, the use of a nonlinear method of analysis explicitly provides the plastic mechanism. This enables explicit consideration of force redistribution and design for an adequate seismic response, which is sometimes not the case if buildings are designed according to the capacity design approach. Therefore risk-based design procedure provides well-informed decision-making. This refers to engineers and also to stakeholders, since they can balance between the acceptable risk and costs.

AKCNOWLEDGEMENT

The results presented in this paper are based on work supported by the Slovenian Research Agency. This support is gratefully acknowledged.

REFERENCES

American Concrete Institute (ACI) (2011). Building code requirements for structural concrete and commentary. *ACI 318-11*, Farmington Hills, MI.

Bhattacharya, B., Basu, R., Ma, K. (2001). Developing target reliability for novel structures: the case of the Mobile Offshore Base. Houston, *Marine structures* **14**, 37-58.

Brozovič, M., Dolšek, M. (2011). Computational efficiency of progressive incremental dynamic analysis. *3rd ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*.

CEN (2004). Eurocode 8: design of structures for earthquake resistance. Part 1: general rules, seismic actions and rules for buildings.

Cornell C.A., Jalayer, F., Hamburger R.O., Foutch, D.A (2002). Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines. *Journal of Structural Engineering* **124:4**, 526-533.

Cornell C.A. (1996). Calculating building seismic performance reliability: a basis for multi-level design norms. *Eleventh World Conference on Earthquake Engineering, Mexico City, Mexico*, Paper No.2122.

Dolšek, M. (2010). Development of computing environment for the seismic performance assessment of reinforced concrete frames by using simplified nonlinear models. *Bulletin of Earthquake Engineering* **8:6**, 1309-1329.

Dolšek, M., Fajfar, P. (2007) Simplified probabilistic seismic performance assessment of plan-asymmetric buildings. *Earthquake Engineering and Structural Dynamics*, **36:13**, 2021–2041.

Fajfar, P., Dolšek, M. (2012). A practice-oriented estimation of the failure probability of building structures. *Earthquake Engineering and Structural Dynamics*, **41:3,** 531-547.

Fajfar, P. (2000). Nonlinear analysis method for performance-based seismic design. *Earthquake Spectra* **16:3**, 573-592.

Helm, P. (1996). Integrated Risk Management for Natural and Technological Disasters. *Tephra* **15:1**, 5-19.

International Organization for Standardization (ISO) (1998). ISO 2394:1998(E) – General principles on reliability for structures.

Jaiswal, K.S., Wald, D.J. (2010). Development of a semi-empirical loss model within the USGS Prompt Assessment of Global Earthquakes for Response (PAGER) System. *9th US and 10th Canadian Conference on Earthquake Engineering: Reaching Beyond Borders*.

Joint Committee on Structural Safety (2000). Probabilistic model code – Part 1: Basis of Design.

Liu, M., Wen, Y.K., Burns, S.A. (2004). Life cycle cost oriented seismic design optimization of steel moment frame structures with risk-taking preference. *Engineering Structures* **26:10**, 1207-1421.

Melchers, R.E. (1999). Structural Reliability Analysis and Prediction, Second Edition. Chichester, John Wiley & Sons.

Open System for Earthquake Engineering Simulation (OpenSEES) (2004). Pacific Earthquake Engineering Research Center (PEER), Univ. of California, Berkeley, CA, (http://opensees.berkeley.edu/).

Rojas, H.A., Foley, C., Pezeshk, S. (2011). Risk-Based Seismic Design for Optimal Structural and Nonstructural System Performance. *Earthquake Spectra* **27:3**, 857-880.

Wen, J.K. (2001). Reliability and performance-based design. *Structural Safety* **23:4,** 407-428.