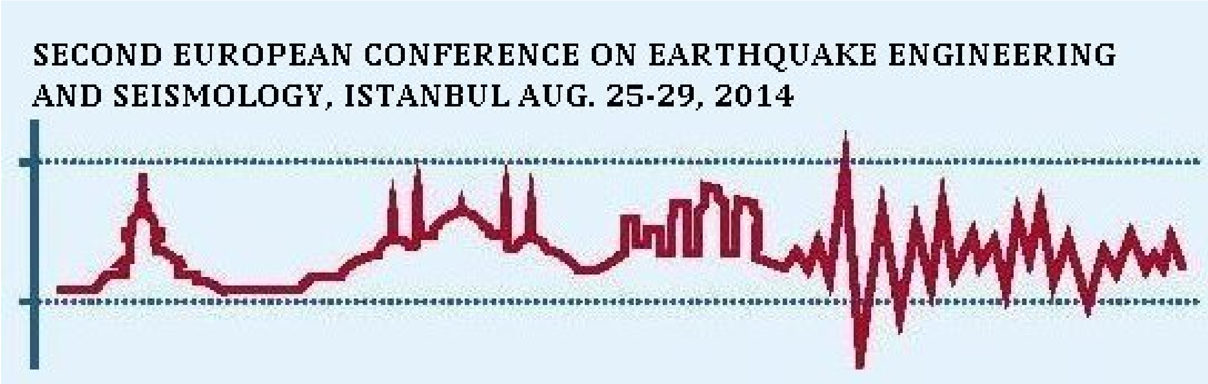

RELATIONSHIP BETWEEN THE COLLAPSE RISK AND THE REINFORCED CONCRETE FRAME STRUCTURE

Jure ŽIŽMOND[1], Damjan PODGORELEC[2] and Matjaž DOLŠEK[3]

ABSTRACT

A risk-based procedure for determination of design seismic action, which was recently developed, is briefly described and applied to a reinforced concrete frame structure in order to assess the relationship between the collapse risk and characteristics of the structure. The relationship was determined by means of an example of an eight-storey reinforced concrete frame building, which was designed for four values of target collapse risk. The assumptions in the design were checked by assessing buildings' seismic performance using the N2 method. It was found that the escalation of target collapse risk by a factor of 50 resulted in an increase of the total design seismic forces by about a factor of 10. This was reflected in the amount of material required for the construction of the structure. The amount of reinforcement and the volume of the concrete were increased by approximately 5 and 2 times, respectively. It has been shown that acceptance criteria defined on the basis of very low target collapse risk cannot be met by the reinforced concrete frame structure.

INTRODUCTION

Eurocode 8 (CEN, 2005a) prescribes that structures should be designed on the basis of a design seismic action, which is defined for a given earthquake recurrence intervals associated with the ultimate and serviceability limit state. The building code assumes that the no-collapse requirement is fulfilled if an ordinary structure is designed for a seismic action associated with return period of 475 years. Due to many safety factors in design, structures can withstand an earthquake which have much greater return period compared to that of the design earthquake. However, a small probability that earthquakes will cause collapse of a structure still exists. In general, earthquakes corresponding to a greater or even a lower intensity compared to that of the design earthquake can still cause collapse of the structure due to unfavorable frequency content. Unfortunately, Eurocode 8 does not provide such information since it is not developed to a level that would enable design of structures to a target degree of reliability, which is the basis of Eurocode 0 (CEN, 2004).

This shortcoming of the existing standards for earthquake-resistant design of buildings can be overcome by introducing more scientifically-oriented design procedures. One option is to design the structure using iterative procedure based on non-linear analysis (Lazar and Dolšek 2013). In this case, seismic performance assessment becomes a key component of design procedure. It is hoped that such an approach may be used in a future for more important structures. However, several other possibilities exist to improve the design procedures of current buildings codes. For example, Costa et

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al. (2010) developed the methodology for determination of behavior factor on the basis of ductility of structure and target probability of collapse. Vamvatsikos et al. (2013) proposed yield frequency spectra, which can be used for the direct design of structure to a set of performance objectives. Another option is to redefine just the seismic design action. This can be achieved by so-called risk-targeted seismic design maps (e.g. Luco et al. 2007, Douglas et al. 2013), which adopt a 'constant risk' assumption by assuming the collapse fragility curves. However, it would be better to develop a design procedure, which would enable differentiation of reliability and providing an insight into the factors which affect the collapse risk. Recently, an attempt has been made to define the risk-based design spectrum and risk-based performance spectrum (Žižmond and Dolšek 2014b). They can be used to design a structure for target reliability and to check its performance on the basis of nonlinear method of analysis. The risk-based spectra depends on the seismic hazard curve at a location of the building, reduction factors due to overstrength and ductility, the target collapse risk and the prescribed dispersion corresponding to the intensity measure causing collapse.

In this paper a new procedure for determination of risk-based spectra is briefly described and applied to the reinforced concrete frame structure in order to show the relationship between the collapse risk and the characteristics of the structure. This goal was achieved by designing an eight-storey reinforced concrete frame building to four target values of collapse risk. The assumptions in the design were justified by assessing buildings' seismic performance using N2 method (Fajfar, 2000).

THE RISK-BASED DESIGN AND PERFORMANCE SPECTRUM FOR DESIGN OF BUILDINGS TO TARGET RELIABILITY

The so called risk-based design and performance spectra (Žižmond and Dolšek 2014b) are briefly presented. For brevity of presentation it is assumed that spectra has the same shape of that prescribed by Eurocode, but the peak ground acceleration depends on the target reliability and the purpose of the analysis. The risk-based design spectrum should be used within the force-based design procedure prescribed by Eurocode 8 in order to achieve the target collapse risk. The assumptions in force-based design should then be checked by assessing the seismic performance of the structure on the basis of risk-based performance spectrum.

The starting point of the definition of the risk-based design and performance spectra is the selection of the target mean annual frequency of limit-state exceedance $P_{LS,t}$. The design seismic action should then be defined in such a way that the resulting structures fulfil the following criterion:

$$P_{LS} \leq P_{LS,t} \tag{1}$$

where P_{LS} is the assessed value of the mean annual frequency of limit-state exceedance once the structure is designed by utilizing the risk-based design spectrum.

The P_{LS} can be obtained by the following integral (e.g. (Pinto 2004, Bradley and Dhakal 2008, Vamvatsikos 2013, Jalayer 2003, Eads et al. 2012)):

$$P_{LS} = \int_0^\infty P\left(LS \middle| IM = im\right) \cdot \left| \frac{dH\left(im\right)}{d\left(im\right)} \right| \cdot d\left(im\right), \tag{2}$$

where the fragility function $P(LS|IM = im)$ is the probability of exceeding the limit state (LS) if the intensity measure (IM) takes on a value equal to im and the hazard curve $H(im)$ is the annual rate of exceedance of im. The fragility function can be simply defined as the cumulative distribution function of the limit-state intensity $P(IM_{LS} < im)$. If it is assumed that the limit-state intensity is log-normally distributed, the fragility function can be expressed by means of the standard normal probability integral (Cornell, 1996):

$$P\left(IM_{LS} < im\right) \approx \Phi\left[\frac{\ln\left(im\right) - \ln\left(im_{LS}\right)}{\beta_{im,LS}}\right], \tag{3}$$

where im_{LS} and $\beta_{im,LS}$ are the median limit-state intensity and the corresponding standard deviation of the natural logarithms and $\Phi(\cdot)$ is the cumulative distribution function of the standard normal variable. In the case if the fragility is defined by means of Equation (3) and if the hazard is assumed linear in log-log domain, the Equation (2) can be solved in closed-form (Cornell 1996, McGuire 2004):

$$P_{LS} = k_0 \cdot im_{LS}^{-k} \cdot e^{\frac{k^2 \cdot \beta_{im,LS}^2}{2}} = H\left(im_{LS}\right) \cdot e^{\frac{k^2 \cdot \beta_{im,LS}^2}{2}} \tag{4}$$

where k and k_0 are parameters of hazard curve which is defined as $H(im_{LS})=k_0 \cdot im_{LS}^{-k}$. If Eq. (4) is inserted into Eq. (1), the median intensity im_{LS}, which causes the violation of the limit state LS in order to achieve the target reliability $P_{LS,t}$, can be expressed as follows:

$$im_{LS} \geq \left(\frac{k_0 \cdot e^{\frac{k^2 \cdot \beta_{im,LS}^2}{2}}}{P_{LS,t}} \right)^{\frac{1}{k}} \tag{5}$$

The threshold value of im_{LS} according to Eq.(5) represents an intensity measure which is used for definition of the risk-based performance spectrum. For defining the risk-based design spectrum, the beneficial effect of the overstrength factor and available ductility have to be taken into account. It should be noted that the threshold value of im_{LS} based on Eq.(1) can be obtained iteratively by numeric integration of Eq.(2). Such an iterative procedure eliminates errors associated by the analytic solution of the risk equation.

For simplicity, the risk-based design spectrum is herein derived only for targeting the reliability against collapse (C), e.g. to fulfil the fundamental “no-collapse” requirement of the Eurocode 8. The peak ground acceleration $a_{g,C}$ is selected for the intensity measure im_C, since Eurocode 8 associates this parameter with an earthquake return period. However, simulation of collapse of structure is very uncertain, especially for complex structures which often have to be designed. Therefore, the seismic performance assessment of structure would be based on so called near-collapse (NC) limit state, since it is estimated that the current nonlinear models of structures are sufficiently accurate to simulate structure behaviour in the near-collapse range. Due to the difference between the limit states, which are used for in the seismic performance assessment procedure and the definition of fundamental non-collapse requirement, the so-called correction factor R_C is introduced:

$$R_C = \frac{a_{g,C}}{a_{g,NC}} \tag{6}$$

where $a_{g,C}$ and $a_{g,NC}$ are, respectively, median peak ground acceleration causing collapse and near-collapse limit state. Note that this correction factor should be based on expert elicitation process in order to achieve consistency between the acceptability criteria and the seismic performance assessment procedure.

The risk-based design peak ground acceleration is defined as the ratio between the $a_{g,NC}$ and the reduction factor R

$$a_{g,d} = \frac{a_{g,NC}}{R} \tag{7}$$

It can be shown that the reduction factor is product of overstrength factor R_S and ductility factor R_μ (e.g. Fischinger and Fajfar 1990). The risk-based design and performance peak ground acceleration can now be expressed on the basis of Eq. (5) by taking into account Eq. (6) and (7):

$$a_{g,d} = \frac{a_{g,NC}}{R} = \frac{a_{g,C}}{R \cdot R_C} \geq \left(\frac{k_0 \cdot e^{\frac{k^2 \cdot \beta_{a_{g,C}}^{\ 2}}{2}}}{P_{C,t}} \right)^{\frac{1}{k}} \cdot \frac{1}{R_s \cdot R_\mu \cdot R_C} \tag{8}$$

$$a_{g,NC} \geq \left(\frac{k_0 \cdot e^{\frac{k^2 \cdot \beta_{a_{g,C}}^{\ 2}}{2}}}{P_{C,t}} \right)^{\frac{1}{k}} \cdot \frac{1}{R_C} \tag{9}$$

The $a_{g,d}$ and $a_{g,NC}$ can be used to define the entire risk-based spectra, which is herein for simplicity assumed to be consistent with the shape of the elastic spectrum according to Eurocode 8. Once the spectra are defined, the Eurocode 2 and 8 provisions can be used for design and the performance assessment of the structure. However, the accuracy of the proposed procedure depends on the accuracy of the selection of standard deviation $\beta_{ag,C}$, which have to be predetermined by the default values, and the reduction factor R. Both prescribed parameters vary with respect to the material of a structure, the structural system, the quality of construction and the quality of structural detailing. It should be emphasize that the typical values of these parameters are not addressed in this paper. The reader is therefore referred to work of Žižmond and Dolšek (2014a). Additionally, it is important to note, that reduction factor R has to express actual (realistic) values of overstrength factor and available ductility. This means that the values of R are much higher than the Eurocode's 8 behavior factor, since the behavior factor implicitly accounts for the reliability (Žižmond and Dolšek 2013, 2014a).

The risk-based design and performance spectra are presented in Fig. 1 in order to demonstrate their definition and the difference between them. Spectra were obtained by prescribing the target probability of collapse 10^{-4}, the standard deviation $\beta_{ag,C}$=0.5 and the parameters of the hazard k=2.9 and k_0=6.41·10^{-5}, which may be typical for region with moderate seismicity Based on this information the risk-based performance spectrum was defined, whereas for the risk-based design spectrum, the reduction factor R=12 was assumed, which may be appropriate for RC frame buildings.

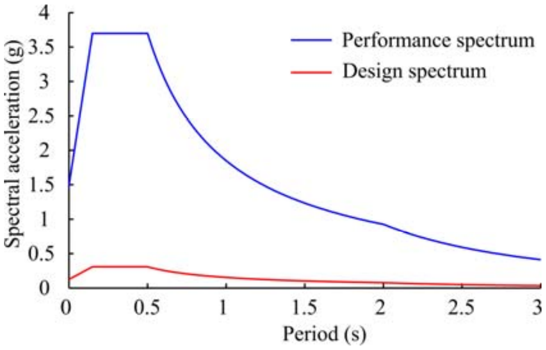

Figure 1. Risk-based design and performance spectra for target collapse risk of 10^{-4}, k=2.9, k_0=6.41·10^{-5}, $\beta_{ag,C}$=0.5 and reduction factor 12

DESIGN AND SEISMIC PERFORMANCE ASSESSMENT OF AN EIGHT-STOREY RC FRAME BUILDING ACCORDING TO EUROCODE 8

An eight-storey structure was designed according to provisions of Eurocode 8 (hereinafter called variant EC8) in order to obtain a point of comparison with respect to variants of structures designed for target collapse risk, and to approximately assess the reduction factor for this type of structures.

The elevation and plan view of the structure are presented in Fig. 2. The building was designed for ductility class M, peak ground acceleration at rock outcrop 0.25 g and soil type C. The behavior factor was assumed 3.9, the quality of reinforcing steel was prescribed S500B whereas the concrete C30/37 was used. The slab depth was 20 cm. The gravity and seismic load combinations were taken into account. Effective beam widths were determined according to Eurocode 2 by assuming the zero moment point at the half-length of the element. The total mass of structure amounted to 2338 t. The first two periods in X and Y directions were, respectively, 1.23 s and 1.28 s, whereas the ratio between the design base shear and the weight is equal to 7.6% and 7.3% for X and Y direction, respectively. More information about design, reinforcement and assessment of the variant EC8 can be found elsewhere (Podgorelec, 2013).

An iterative procedure was used in order to determine an appropriate relationship between size of the cross-sections of the columns and beams and the corresponding amount of reinforcement. The following principles were applied:

- ratio of longitudinal reinforcement in the columns based on seismic demand from linear elastic analysis should not exceed 2%
- ratio of longitudinal reinforcement in the columns based on additional criterion associated with capacity design principles should not exceed 2.5%
- the reinforcement ratio at tension zone of the beams should not exceed the value ρ_{max} given by EN 1998-1:2005 5.4.3.1.2 (4b) (CEN, 2005a)
- the square cross-sections are used for the columns, which do not change over height of the building.
- the thickness of the web of the beam (b_w) and the height of beam (h_b) are equal for the entire structure.

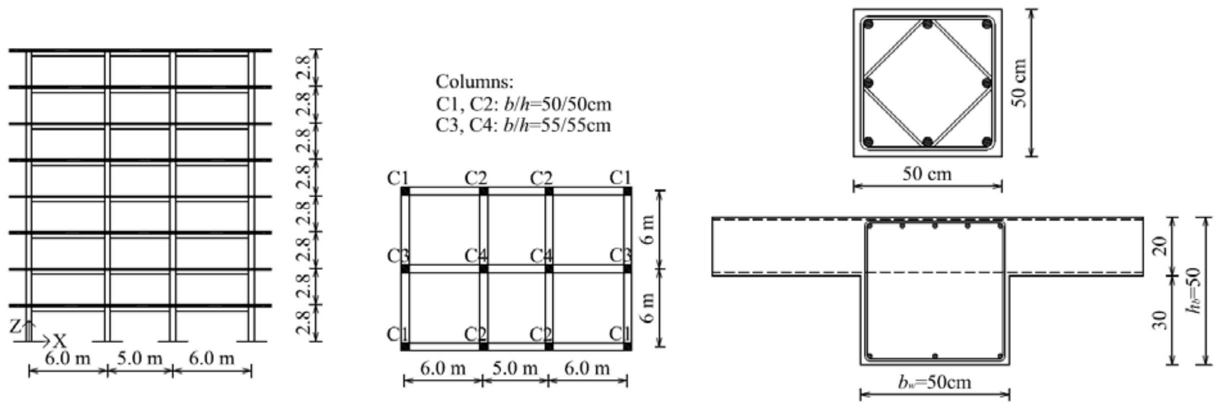

Figure 2. The elevation, plan view and reinforcement in typical columns and beams of the EC8 variant of 8-storey building

Structural model

The structural model for nonlinear analysis was based on the Eurocode 8 requirements as discussed elsewhere (Dolšek, 2010). The beam and column flexural behavior 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 was based on the assumption of an inflexion point at the midpoint of the element. The gravity load was 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 was used for major axis bending only. For the columns, two independent plastic hinges for bending about the two principal axes were used. The moment-rotation relationship before strength deterioration was modelled by a bi-linear relationship. A linear negative post-capping stiffness was assumed after the maximum moment had been achieved. The axial force due to gravity loads was taken into account when determining the moment-rotation relationship for the columns, while in the case of the beams zero axial force and the T cross-sections were assumed. The ultimate rotation Θ_u in the columns and beams at the near collapse (*NC*) limit state, which corresponds to a 20% reduction in the maximum moment, was estimated by means the EC8-3 formulas (CEN 2005b). The parameter γ_{el} was assumed to be equal to 1.0, since mean values of near collapse rotation were used. Mean (actual) concrete (38 MPa) and steel (570 MPa) strength were assumed. Post-capping negative stiffness was calculated by assuming the ratio between the rotation at zero strength and the rotation corresponding to the maximum moment equal to 3.5. Positive (4.24 cm^2/m) and negative (2.26 cm^2/m) slab reinforcement on the entire width of the effective beam (same width as for design) were taken into account for determination of the yield moments of the beams.

All the analyses were performed with OpenSees (PEER, 2013), using the PBEE toolbox (Dolšek, 2010), which is a simple yet effective tool for the seismic performance assessment of reinforced concrete frames by using simplified nonlinear models.

Seismic performance assessment of structure

The N2 method (Fajfar, 2000) was used for assessing the seismic performance of a structure. Conventional pushover analyses were performed. The invariant force vectors corresponded to product of the storey masses and fundamental vibration modes (X and Y direction). The pushover curves and corresponding idealized pushover curves for X and Y directions are shown in Fig. 3. Note that the idealized pushover curves were assumed to be valid until the near collapse (NC) limit state. Since Eurocode 8 does not provide definition about the NC limit state of RC frame buildings, it was assumed that the NC limit state at the structural level was attained at the base shear corresponding to the 80% of maximum strength, if measured in the post-capping range of the pushover curve. The yield strength F_y (which corresponds to maximum strength) of idealized system were 2298 kN and 2340 kN for X and Y direction, respectively. The yield displacement d_y and near-collapse displacement d_{NC} were equal to 5.7 cm and 48.4 cm for X direction and 6.4 cm and 51.1 cm for Y direction, respectively. Based on the N2 method it was assessed that the peak ground accelerations which causes the NC limit state were equal to 0.82 g for both directions.

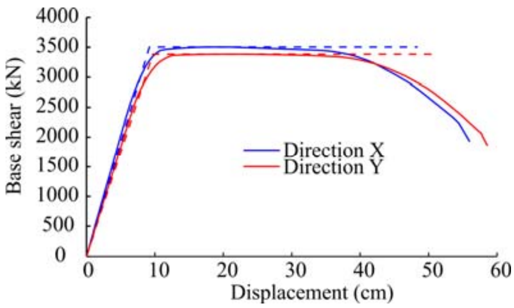

Figure 3. The pushover curves of the EC8 variant of building

The results of the seismic performance assessment according to the N2 method were used to assess the reduction factor R:

$$R = \frac{a_{g,NC}}{a_{g,d}} = \frac{0.82}{0.074} = 11.1 \tag{10}$$

where $a_{g,d}$ was obtained by taking into account the behavior factor based on Eurocode 8. It is calculated as follows:

$$a_{g,d} = \frac{S \cdot a_{g,R}}{q} = \frac{1.15 \cdot 0.25}{3.9} = 0.074g \tag{11}$$

Note that the reduction factor R is equal for both directions in the case of the eight-storey building since the near-collapse peak ground acceleration is almost equal for both directions of seismic action. It is obvious that the reduction factor is significantly higher than the behavior factor as prescribed in Eurocode 8. For simplicity, the reduction factor based on variant EC8, will be assumed for all other variants (1 to 4), although it can slightly vary with respect to the target collapse risk.

DESIGN OF AN EIGHT-STOREY RC FRAME FOR FOUR VALUES OF TARGET COLLAPSE RISK

Four values of target collapse risk were selected (Table 1) in order to investigate the relationship between the collapse risk and the characteristics of an 8-storey reinforced concrete frame structure. The values of target collapse risk were based on arbitrarily selected values of target risk of loss of life ($P_{LL,t}$ see Table 1). The fatality rate 0.15, as defined by (Jaiswal and Ward, 2010) for the ductile reinforced concrete frames, was adopted in to obtain the relationship between the target risk of loss of life and the collapse risk. Note that consequently the target collapse risk $P_{C,t}$ was larger than the $P_{LL,t}$ (see Table 1). However, the target collapse risk was selected in such a way that the collapse risk for variants 3, 2 and 1 was, respectively, 5, 10 and 50 times smaller than that assumed for the variant 4.

Table 1. The target risk of loss of life $P_{LL,t}$, the target collapse risk $P_{C,t}$, and the ratio between the target risk for different variants of structures

Variant	1	2	3	4
$P_{LL,t}$	$1\cdot10^{-6}$	$5\cdot10^{-6}$	$1\cdot10^{-5}$	$5\cdot10^{-5}$
$P_{C,t}$	$6.7\cdot10^{-6}$	$3.3\cdot10^{-5}$	$6.7\cdot10^{-5}$	$3.3\cdot10^{-4}$
$P_{Ci,t}$ / $P_{C4,t}$	*1/50*	*1/10*	*1/5*	*1*

Additionally to the target risk, several structural parameters have to be prescribed in order to determine the risk-based spectra for design and seismic performance assessment. The dispersion of collapse intensity $\beta_{ag,C}$ was assumed from results of previous studies. Less information were available for assuming an appropriate value for the reduction factor *R*. Thus, the reduction factor was based on the results of seismic performance assessment of variant EC8.

Determination of the risk-based design and performance spectra for the four values of collapse risk

The tolerable median peak ground accelerations which cause collapse were calculated using Eq. (5). The $\beta_{ag,C}$ was assumed 0.60 (Dolšek, 2012). The slope of the hazard curve (k=2.9) was obtained from previous study (Lazar and Dolšek 2013). However, parameter $k_0=5.67\cdot10^{-5}$ was determined by assuming equal hazard for the design peak ground acceleration according to Eurocode 8 ($a_{g,R}$=0.25 g, S=1.15 $H(a_g)$=1/475). Such decision cause slight adjustment of the hazard curve, which does not affect the conclusions of this study. The reduction factor R=11.1, which corresponded to variant EC8, was used for all variant of structure. Since N2 method cannot be used to estimate the global dynamic instability, the correction factor R_C=1.2 was adopted from previous study (Brozovič and Dolšek 2011). Note that the procedure for determination of the amount of reinforcement was based on Eurocodes 2 and 8 by assuming the ductility class medium (DCM).

The procedure for determination of the risk-based design peak ground acceleration (and spectrum) is presented only for variant 4 ($P_{C,t}=3.3\cdot10^{-4}$) (see Eq. (12) and (13)). Results corresponding to other values of target collapse risk are presented in Table 2. Note that the ratio between the risk-based design peak ground acceleration of *i*-th variant and that of variant 4 is also presented in the last row of Table 2. It can be observed that the risk-based peak ground accelerations of variant 1 are greater for around factor of 3.9 in comparison to those obtained for variant 4, while the target collapse risk differs for a factor of 50.

$$a_{g,C} \geq \left(\frac{k_0 \cdot e^{\frac{k^2 \cdot \beta_{a_{g,C}}^2}{2}}}{P_{C,t}} \right)^{\frac{1}{k}} = \left(\frac{5.67 \cdot 10^{-5} \cdot e^{\frac{2.9^2 \cdot 0.6^2}{2}}}{3.3 \cdot 10^{-4}} \right)^{\frac{1}{2.9}} = 0.91g \tag{12}$$

$$a_{g,NC} = \frac{a_{g,C}}{R_C} = \frac{0.91g}{1.2} = 0.76g\ , \quad a_{g,d} = \frac{a_{g,NC}}{R} = \frac{0.76g}{11.1} = 0.07g \tag{13}$$

Table 2. The risk-based performance peak ground acceleration associated with the collapse and near-collapse limit state, and the risk-based design peak ground accelerations for the four values of target collapse risk.

Variant	1	2	3	4
$a_{g,C}$ [g]	3.53	2.02	1.59	0.91
$a_{g,NC}$ [g]	2.94	1.69	1.33	0.76
$a_{g,d}$ [g]	0.26	0.15	0.12	0.07
$a_{gi,d}$ / $a_{g4,d}$	*3.9*	*2.2*	*1.7*	*1.0*

Based on the risk-based design and performance peak ground acceleration, the corresponding spectra can be obtained according to Eurocode 8 provisions (Fig. 4). For comparison, the design and elastic spectrum for variant EC8 are also presented in Fig. 4. It can be observed that the design spectrum based on Eurocode 8 is very similar to the risk-based design spectrum corresponding to the target collapse risk of $3.3 \cdot 10^{-4}$ (variant 4). The difference between the elastic spectrum based on Eurocode 8 and the risk-based design spectrum is only minor in the case if the target collapse risk is set to $6.7 \cdot 10^{-6}$ (variant 1). However, the difference between the design and performance spectra is large, since these spectra are intend to be used for different purposes. It should be emphasize that the shape of the spectra was assumed constant due to simplicity.

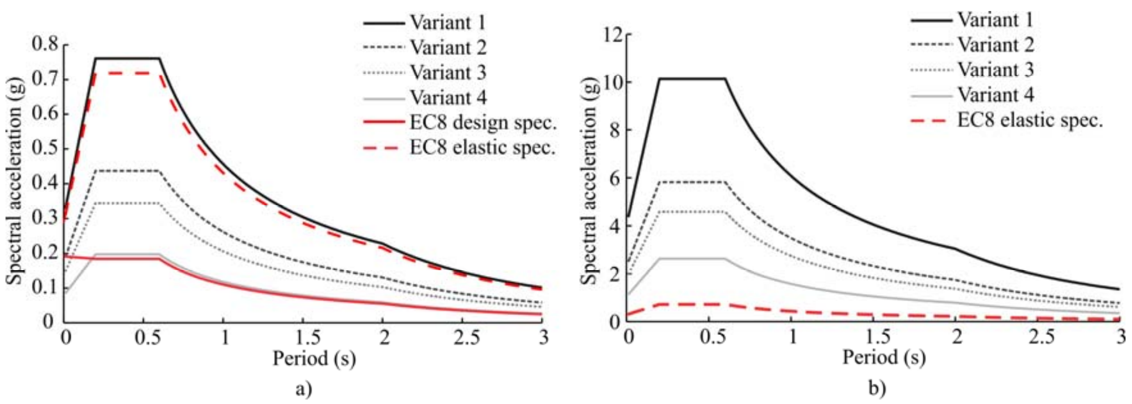

Figure 4. a) the risk-based design spectra and b) risk-based performance spectra corresponding to the four target collapse risk. For comparison, the design and the elastic spectra according to Eurocode 8 are also presented.

Description of the structures designed according to the risk-based spectra

Four variants of the eight-storey building were designed using gravity-load combinations and load combinations associated with the risk-based design spectrum. Note that the gravity load and corresponding storey masses varied with respect to the amount of material, which was required for each variant of the structure. The concrete cross-sections of the columns and beams were estimated utilizing the same iterative process which was explained for the variant EC8. The resulting dimensions of the cross-sections of the beams and the columns are presented in Table 3. Gradual increase of the dimensions of concrete cross-sections can be observed. However, the trend is not linear with respect to the increase of target reliability. An insight into the intermediate results of design makes it easier to understand the relationship between the target reliability and the dimensions of the structure (i.e. the volume of concrete and reinforcement).

Table 3. The dimensions of the concrete cross-sections of the beams and columns for investigated variants of the building. See Figure 3 for labels of the columns.

Variant	1	2	3	4	EC8
Beams b_w=h_b [cm]	75	60	55	50	50
Columns C1 b=h [cm]	80	60	50	50	50
Columns C2 b=h [cm]	90	65	60	50	50
Columns C3 b=h [cm]	90	70	60	50	55
Columns C4 b=h [cm]	100	70	60	55	55

The masses, the fundamental vibration periods and the ratio between base shear F_b and weight W of structure are presented in Table 4. Quite significant differences can be observed between different variants of the structure. It is obvious that the increase of the target reliability of structure caused an increase of design base shear (Table 4). However, the increase of design base shear resulted from the increased mass and increased value of the design spectral acceleration. The later parameter had greater impact. Its increase was partly a consequence of stricter criterion of the target collapse risk and partly a consequence of the decreased fundamental vibration period, since the effect of the increased stiffness (from variant 4 to 1) prevailed the effect of the increased mass. For example, the increase of target reliability by a factor of 50 (from variant 4 to – Table 1) caused an increase of design peak ground acceleration by a factor of 3.9 (Table 2), increase of total mass of structure by a factor of 1.4 (Table 4) and decrease of period of structure from 1.24 s to 0.62 s (from 1.31 s to 0.66 s for Y direction) (Table 4). All the above-mentioned variations of parameters caused an increase of design base shear by a factor of around 10 (from variant 4 to 1). It is interesting to note that the increase of target reliability in comparison to that of variant 4 did not significantly affect the total mass of the variants 2, 3. The total mass of variant 2 is only around 13 % higher than the mass of variant 4. However, the dimensions of the cross-sections of variant 1 were too large and thus unsuitable for construction. Therefore it would make sense to select different structural system in the case of very strict criterion for the collapse risk.

Table 4. The fundamental vibration periods, the mass, the design base shear and the ratios between design base shear and the weight of structure

Variant	1	2	3	4	EC8
T_X [s]	0.62	0.89	1.04	1.24	1.23
T_Y [s]	0.66	0.94	1.11	1.31	1.28
$F_{b,X}$/W	52 %	21 %	15 %	7.1 %	7.6 %
$F_{b,Y}$/W	49 %	20 %	14 %	6.7 %	7.3 %
m [t]	3222	2627	2462	2332	2338
m_i/m_4	1.38	1.13	1.06	1.00	/
$F_{b,X}$ [kN]	16351	5484	3491	1615	1754
$F_{b,X,i}/ F_{b,X,4}$	10.1	3.4	2.2	1.0	/
$F_{b,Y}$ [kN]	15187	5055	3185	1470	1681
$F_{b,Y,i}/ F_{b,Y,4}$	10.3	3.4	2.2	1.0	/

Different criteria controlled the amount of reinforcement in the case of investigated variants. For example, the top reinforcement in the beams of all stories and for all four variants was obtained on the basis of seismic demand while for the bottom reinforcement this was true only for variant 1. For other variants, the minimum amount of reinforcement according to Eurocode 8 (50% of the top (positive) reinforcement), was sometimes the decisive criterion for the bottom (negative) reinforcement of the beams. This requirement was relevant for all the beams of the 7th and 8th storey of variant 2, for the beams above the 5th storey of variant 3 and for all the beams of variant 4. Note that for all the variants, the transverse reinforcement in the beams in all stories was controlled by the capacity design principles.

The minimum requirement for the amount of the longitudinal reinforcement in the primary columns (1%) was decisive criterion for all columns of variant 4 and for columns in the upper stories of other variants. In other cases, the concept of weak beam – strong column controlled the amount of the longitudinal reinforcement of the columns. The transverse reinforcement in the columns was controlled by the capacity design principles. However, the condition for confinement of concrete core (EN 1998-1: 2005 5.4.3.2.2 (8) and (9)) (CEN, 2005a) of some columns in the bottom storey of variant 4 controlled the amount of traverse reinforcement.

The breakdown list of the mass with respect to the mass of reinforcement and the mass of concrete of investigated structures and their structural components is presented in Table 5. Please note that the mass of concrete was estimated by taking into account only the mass of the columns and beams (with effective beam width) while the mass of reinforcement is based on the calculated reinforcement in the critical region of the element. Thus, it makes sense to compare the amount of estimated material relative to different variants of the structure. Comparison of results from Table 5 showed that the increase of target reliability caused an increase of mass of both the reinforcement and the concrete. The increase of target reliability by a factor of 50 (from variant 4 to 1 – Table 1) caused an increase of total mass of reinforcement by a factor of 4.6 and an increase of total mass of concrete by a factor of 1.9 (Table 5). In general, the maximum increment of the material was observed for the bottom reinforcement of the beam and the longitudinal reinforcement of the columns, since the decisive criterion for the amount of the reinforcement changed in these cases (see paragraphs above). However, according to our opinion the required mass of material is not acceptable in the case of variant 1.

Table 5. The mass of reinforcement and the concrete for the selected structural components of all investigated variants of the 8-storey building.

Mass of reinforcement [t]	Variant			
	1	2	3	4
Beam at top, $m_{TBR,i}$	48	23	17	12
Beam at bottom, $m_{BBR,i}$	37	15	10	7
Beam, transverse, $m_{TrBR,i}$	49	21	15	11
Column, longitudinal, $m_{CR,i}$	39	17	12	7
Column, tranverse, $m_{TrCR,i}$	14	8	6	4
Total mass [t]				
Reinforcement, $m_{R,i}$	187	83	60	41
Concrete, $m_{C,i}$	2086	1437	1254	1106
Ratio of masses				
$m_{TBR,i} / m_{TBR,4}$	4.1	2.0	1.4	1.0
$m_{BBR,i} / m_{BBR,4}$	5.5	2.2	1.5	1.0
$m_{TrBR,i} / m_{TrBR,4}$	4.4	1.8	1.3	1.0
$m_{CR,i} / m_{CR,4}$	5.3	2.3	1.6	1.0
$m_{TrCR,i} / m_{TrCR,4}$	3.7	2.0	1.6	1.0
$m_{R,i} / m_{R,4}$	4.6	2.0	1.5	1.0
$m_{C,i} / m_{C,4}$	1.9	1.3	1.1	1.0

Seismic performance assessment of structures designed for target collapse risk

The seismic performance of the four variants of the structure was assessed by the N2 method in order to verify if structures met the acceptance criteria for target collapse risk. The type of nonlinear structural model was the same as that used in the case of variant EC8. The peak ground acceleration which causes the NC limit state was computed and compared to that of the risk-based performance spectrum. Note that the peak ground acceleration causing collapse was determined multiplying the peak ground acceleration causing NC limit state by a factor of 1.2 (Brozovič and Dolšek 2011). The mean annual frequency of exceeding collapse limit state was calculated using Eq. (4). The standard deviation $\beta_{ag,C}$ was assumed equal to 0.60 (Dolšek 2012).

The results of pushover analysis are presented in Fig. 5. Some selected numerical results, such as the assessed peak ground acceleration, which cause near collapse or collapse limit state ($a_{g,NC,a}$ and $a_{g,C,a}$), the assessed (realized) mean annual frequency of exceeding collapse limit state $P_{C,a}$ are shown in Table 6. Additionally, the overstrength factor and reduction factor due to ductility are presented in Table 7. It can be observed that the available global ductility (Table 7) increases with increasing target reliability whereas the overstrength factor varies since the decisive measure for the amount of the reinforcement also varied with respect to the target reliability. Furthermore it is important to note that the $a_{g,NC,a}$ almost always exceeded the risk-based performance peak ground acceleration (Table 2). An exception can be observed for the variant 3 if analysed in Y direction. Consequently, the actual reliability is slightly greater than the target reliability.

The maximum ratio between $a_{g,C,a}$ and $a_{g,C}$ from the risk-based performance spectrum is 1.18 (variant 1 and 2 - Table 6). The difference between $a_{g,C,a}$ and $a_{g,C}$ is the consequence of the assumed constant value of reduction factor R. Note that the ratio between $a_{g,C,a}$ and $a_{g,C}$ is equal to the ratio between the reduction factor, which is explicitly obtained from the nonlinear analysis, and the assumed value of the reduction factor R. However, the difference is not so large taking into account that the target reliability varied for a factor of 50. The error in the assumed value of the reduction factor can be eventually expressed in term of the annual frequency of collapse, which, however, never exceeded 60%.

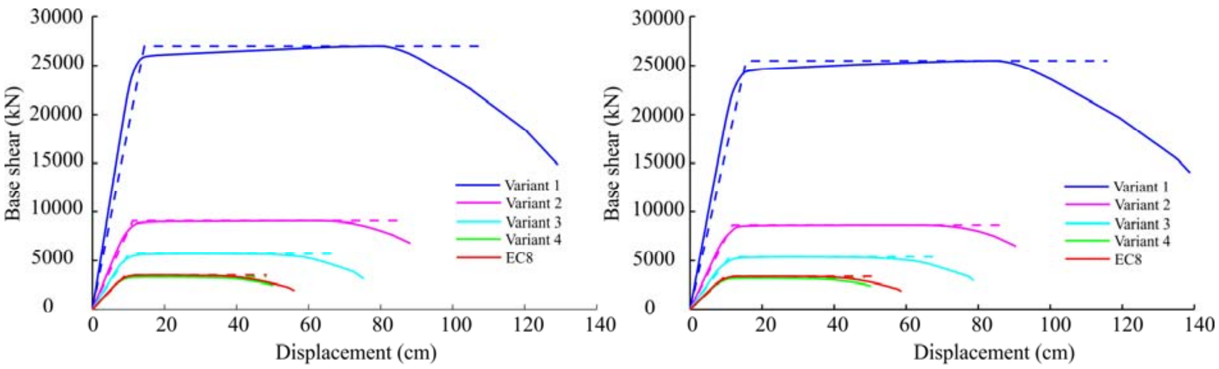

Figure 5. The pushover curves for analysis in X (left) and Y (right) direction in the case of the four variants of the building. The pushover curve of variant EC8 is also presented.

Table 6. The mass, the first vibration modes and ratios between design base shear and weight of structure (left – X direction, right - Y direction).

Variant	1	2	3	4	EC8
$a_{g,NC,a,}$ [g]	3.46	1.99	1.33	0.81	0.82
$a_{g,C,a}$ [g]	4.15	2.38	1.60	0.97	0.98
$r_{a./per.}$	*1.18*	*1.18*	*1.00*	*1.06*	/
$P_{C,a}$	$4.1\cdot10^{-6}$	$2.1\cdot10^{-5}$	$6.6\cdot10^{-5}$	$2.8\cdot10^{-4}$	$2.7\cdot10^{-4}$
$r_{a./per.}$	*1.61*	*1.61*	*1.01*	*1.18*	/

Variant	1	2	3	4	EC8
$a_{g,NC,a}$ [g]	3.46	1.95	1.29	0.77	0.82
$a_{g,C,a}$ [g]	4.15	2.34	1.55	0.92	0.99
$r_{a./per.}$	*1.18*	*1.16*	*0.97*	*1.01*	/
$P_{C,a}$	$4.2\cdot10^{-6}$	$2.2\cdot10^{-5}$	$7.2\cdot10^{-5}$	$3.3\cdot10^{-4}$	$2.7\cdot10^{-4}$
$r_{a./per.}$	*1.60*	*1.52*	*0.93*	*1.02*	/

Table 7. The actual (realized) reduction factor, overstrength and ductility factor (left – X direction, right - Y direction).

Variant	PF1	PF2	PF3	PF4	EC8
$R_{\mu}=\mu$	7.5	7.7	6.6	5.5	5.3
R_S	1.8	1.7	1.7	2.1	2.1
R	13.1	13.1	11.8	11.2	11.1

Variant	PF1	PF2	PF3	PF4	EC8
$R_{\mu}=\mu$	7.4	7.5	6.3	5.1	5.3
R_S	1.8	1.7	1.7	2.20	2.1
R	13.1	12.9	10.9	11.2	11.2

CONCLUSIONS

A new risk-based procedure was used to define seismic action for design and performance assessment of an eight-storey building on the basis of four different values of the target reliability. The value of reduction factor (the product of the overstrength reduction factor and the ductility reduction factor) was estimated based on the structure designed according to Eurocode, and then assumed constant for all four variants of structure design according to the proposed risk-based design procedure. It was found that the escalation of target collapse risk by a factor of 50 resulted in an increase of the total design seismic forces by a factor of 10. This was reflected in the quantity of material required for the structure. The amount of reinforcement was increased by about 4.6 times, while the volume of the concrete was greater by a factor of 1.9. However, it appeared that the frame system was not suitable if the criterion of target collapse risk was very strict. On the other hand the escalation of target reliability for factor 10 (variant 2) and 5 (variant 3) did not have such a great impact on global parameters of structures and their mass. For variant 2, the design base shear increase for a factor 3.4, which increased the total mass of concrete and reinforcement, respectively, by a factor of 2.0 and 1.3. Note that the variant 4 of the investigated building was quite similar to the structure designed according to Eurocodes.

Based on the results of the study it can be concluded that the proposed risk-based design procedure is quite similar to that prescribed by Eurocode 8, since it differs only in terms of definition of the seismic action, which, according to the new procedure, depends on the target reliability rather than earthquake recurrence interval.

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