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DEAGGREGATION OF SEISMIC SAFETY IN THE DESIGN OF REINFORCED CONCRETE BUILDINGS USING EUROCODE 8

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Abstract. *Design of earthquake-resistant structures according to Eurocod 8 is not based on the concept of acceptable/tolerable probability of exceedance of the near collapse limit state. Rather than that standard introduces fundamental non-collapse and damage limitation requirements, which are associated with the design seismic action. It is foreseen that the non-collapse requirement is satisfied when the regular structure does not collapse in the case of an earthquake with a return period of 475 years. Probability of such an even in 50 years is 10%. Therefore it is obvious that probability of failure of structures, which would be designed strictly according to the fundamental non-collapse requirement, would be unacceptable for society. Due to factors of safety involved in design the structures withstand much stronger earthquakes in comparison to an earthquake with a period of 475 years. In order to assess which factor of safety have the greatest impact on the overall safety of code-conforming buildings, two multi-storey reinforced concrete buildings were investigated. The strength and the system ductility of the six variants of the structures were evaluated on the basis of the pushover analysis gradually taking into account the requirements of the Eurocode 2 and 8, as well as gradually excluding the design assumptions. Safety in design of the buildings was evaluated by the difference between the calculated and prescribed behaviour factor, by the ratio between the design ground acceleration and that associated with the near collapse limit state, which was assessed using the N2 method, and by the escalation of safety in terms of probability of exceedance of the near collapse limit state. The results of this analysis are discussed in the paper. For the investigated buildings it is shown that the design seismic action has the greatest impact on the yield strength of the structure and the peak ground acceleration, which cause the near-collapse limit state. On the other hand, the partial factors of material strength contribute around 50% to the return period of the near-collapse limit state, whereas the contribution of the capacity design principles to overall safety is minor.*

1 INTRODUCTION

Current standard for earthquake design of buildings Eurocode 8 [1] which is used in Slovenia prescribes that buildings should be design to withstand two fundamental requirements: no-collapse requirement and damage limitation requirement. It is assumed that the non-collapse requirement is satisfied when the structure is designed according to Eurocode 8 provisions taking into account an earthquake with a return period of 475 years. Usually design procedures involve factors of safety since the design is based on simplified 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 concept of reduction (behavior) factor and capacity design procedure.

In ATC 19 [2] it is discussed based on ATC-3-06 that the reduction factors were intended to reflect reduction in design force vales that were justified on the basis of risk assessment, economics, and nonlinear behavior. Therefore determination of the reduction factors is not trivial. In the basic formulation of the reduction factor it can be shown that it can be defined as the product of equivalent global ductility factor and overstrength factor [3]. However, some other authors, as discussed in ATC 19 [2], defined the reduction factor as the product of period-dependent strength factor, the period-dependent ductility factor, the redundancy factor and in some cases also as the product of the viscous damping factor. A comprehensive evaluation of proposals for strength reduction factors (R_μ) for earthquake-resistant design was done by Miranda and Bertero [4]. They concluded that the ductility-, period- and site-dependent strength reduction factors, together with estimates of the overstrength of the structure, can lead to a more rational and transparent seismic design approach.

However, current building code involves different factors of safety which affect structural configuration (dimensions of structural elements and corresponding reinforcement). In order to identify, which factors have the greatest impact on the seismic safety of code-conforming buildings, seismic safety deaggregation was performed for two reinforced concrete frame buildings which are located in the region with moderate seismicity and designed for ductility class medium. In the paper, the pushover-based method for estimation of the failure probability, which were used in the analysis, are briefly described. Then the factor of safety are defined and assessed for two investigated buildings.

2 SUMMERY OF PUSHOVER-BASED METHOD FOR ESTIMATION OF FAILURE PROBABILITY OF BUILDING STRUCTURES

The basic pushover-based methods are often used to approximately assess risk of near-collapse state of the building (e.g. [5, 6]), which is expressed by the mean annual frequency of exceedance as follows:

$$P_{NC} = \int_0^\infty P\left[NC \mid A_g = a_g\right] \cdot \left|\frac{dH\left(a_g\right)}{da_g}\right| da_g \quad (1)$$

where $P\left[NC \mid A_g = a_g\right]$ is the fragility, i.e. the conditional probability of violating the near-collapse limit state for a given level of ground motion intensity a_g, $H\left(a_g\right)$ is the hazard function approximately representing the probability of exeedance of a_g per year, and NC stands for the near collapse limit state, which can be defined in various manners. The fragility is often defined by assuming lognormal distribution of ground motion intensity which causes the near-collapse limit state. In this case the fragility can be expressed by using standard normal probability integral $\Phi(\cdot)$:

$$P\left(NC \middle| A_g = a_g\right) = \Phi\left(\frac{\ln\left(a_g\right) - \ln\left(\tilde{a}_{g,NC}\right)}{\beta_{NC}}\right) \tag{2}$$

where $a_{g,NC}$ is the median value of ground motion intensity causing the violation of NC limit state and β_{NC} is the logarithmic standard deviation of $a_{g,NC}$. In the case if the fragility is defined by means of Eq.(2) and if the hazard is assumed linear in log-log domain, the Eq.(1) can be solved in closed-form [7, 8]:

$$P_{NC} \approx H\left(a_{g,NC}\right) \cdot e^{\frac{1}{2} \cdot k_{NC}^2 \cdot \beta_{NC}^2} \tag{3}$$

where k_{NC} is the slope of the hazard curve associated with the near-collapse limit state.

In general case the fragility parameters ($a_{g,NC}$, β_{NC}) are estimated by using nonlinear response history analysis. Such approach is computationally demanding, especially if used in the designs process, which requires several iterations in order to obtain the final structural configuration. Therefore approximate methods are often employed for estimation of the fragility parameters [5, 6, 9]. In the simplest approach, the $a_{g,NC}$ is determined according to the N2 method [10], which involves pushover analysis, often performed by assuming the invariant distribution of lateral forces corresponding to first vibration mode. Based on the results of pushover analysis, the equivalent SDOF model is defined utilizing the transformation factor Γ and the mass of the equivalent SDOF model m^* [10]:

$$\Gamma = \frac{m^*}{\sum_{j=1}^{n} m_j \phi_{1,j}^2}, \quad m^* = \sum_{j=1}^{n} m_j \phi_{1,j} \tag{4}$$

where m_j is the jth storey mass of the structure and $\phi_{1,j}$ is the jth component of the first natural vibration mode, which is normalized to a roof displacement equal to 1. In order to determine the force-displacement relationship of the equivalent SDOF model, the pushover curve has to be idealized. There are many options how the pushover curve is idealized. Recently a web-based application was developed, which enables quadrilateral idealization of the pushover curve and prediction of the approximate relationship between the ground motion intensity measure and the displacement of the equivalent SDOF model [11]. In the most basic case, the pushover curve is idealized by elasto-plastic force-displacement relationship. This approach enables use of inelastic spectra. Consequently the reduction factor due to ductility $R_{\mu,NC}$, i.e. due to the hysteretic energy dissipation of ductile structures [10], corresponding to the near-collapse ductility μ_{NC} can be rapidly estimated as follows [10]:

$$R_{\mu,NC} = \begin{cases} (\mu_{NC} - 1)\dfrac{T^*}{T_C} + 1 & \cdots \quad T^* < T_C \\ \mu_{NC} & \cdots \quad T^* \geq T_C \end{cases}, \quad \mu_{NC} = \frac{D_{NC}}{D_y} \tag{5}$$

where D_{NC} and D_y are, respectively, the displacement corresponding to the near-collapse limit state and displacement at yielding of the idealized pushover curve, T^* is the period of the equivalent SDOF model and T_C the period between the range of the constant acceleration and constant velocity of the acceleration spectrum. The reduction factor $R_{\mu,NC}$ and the period T^* are defined as follows [10]:

$$R_{\mu,NC} = \frac{S_{a,NC}}{S_{ay}}, \quad S_{ay} = \frac{F_y}{\Gamma \cdot m^*}, \quad T^* = 2\pi \sqrt{\frac{m^* \cdot D_y}{F_y}} \tag{6}$$

where $S_{a,y}$, $S_{a,NC}$, F_y are, respectively, the spectral acceleration causing 'yielding' of the equivalent SDOF model, the spectral acceleration causing the near-collapse limit state, and the yield displacement of the idealized force-displacement relationship.

When fragility parameters are assessed by using the above-described procedure, the β_{NC} has to be predetermined according to previous studies (e.g. [6, 9, 11]). However, the $a_{g,NC}$ can be calculated based on the shape of the elastic acceleration spectrum used for the assessment of the $S_{a,NC}$, which is determined for the known μ_{NC} and by employing Eq.(5) and Eq.(6).

3 FACTORS OF SAFETY ASSOCIATED WITH EARTHQUAKE-RESISTANT DESIGN OF BUILDINGS

The objective of this study is deaggregation of the seismic safety against near-collapse performance of buildings designed according the Eurocode 8 [1]. It is worth to emphasize that discussion regarding the adequacy of seismic safety of code-conforming buildings is not the topic of this study. Therefore it is assumed that the reinforced concrete buildings designed in compliance with all provisions of Eurocode 2 [12] and Eurocode 8 are safe, although this is a subject for debate.

Standards for earthquake-resistant design of structures involve several provisions, which have an impact on the seismic safety of the structure. Additionally, the actual amount of reinforcement in structural elements is larger than that obtained from design. However, it has been assumed that the following factors have a direct or indirect impact on the seismic safety of the facility:

a) seismic design action
b) minimum requirements of Eurocode 2 for detailing and dimensioning of structural elements
c) minimum requirements of Eurocode 8 for detailing and dimensioning of structural elements
d) ratio between the actual (mean) and design strength of material
e) ratio between the actual and required amount of reinforcement in structural elements
f) capacity design principles prescribed in Eurocode 8.

In order to quantify how the above-mentioned factors affect the seismic safety of the code-conforming building, the overall factor of safety and so-called partial factor of safety are defined as follows:

$$FS = \frac{y_{all}}{y_0} \tag{7}$$

$$\delta FS_k = \frac{y_k}{y_{k-1}} \tag{8}$$

where y_{all} is the parameter of the code-conforming structure or its seismic performance, y_0 is the same parameter but assessed for the case when the seismic design action is the only considered factor which affects structural configuration, and the y_i corresponds to the case when first *k*th number of factors from the list above are considered in the design of a building. According to this definition the overall factor of safety is defined as the product of the partial factor of safety:

$$FS = \prod_{k=1}^{n} \frac{y_k}{y_{k-1}} \tag{9}$$

where *n* is number of all factors which affect the seismic safety of the code-conforming buildings.

For simplicity reasons, the above-defined factors of safety are assessed for the global structural parameters (F_y, μ_{NC}), which are obtained from the idealized pushover curves, and for factors of safety associated with structural performance, such as $a_{g,NC}$, P_{NC} (or $T_{R,NC} = 1/P_{NC}$) and q_{NC}, which can be understood as actual (realized) behaviour factor (Eurocode 8). The q_{NC} is herein defined according to Fischinger and Fajfar [3] as follows:

$$q_{NC} = R_s \cdot R_{\mu,NC} \tag{10}$$

where R_s is the reduction factor due to overstrength and $R_{\mu,NC}$ the reduction factor due to the hysteretic energy dissipation of ductile structures (Eq.(6)). Note that a value of a global structural parameter and a value of performance parameter depend on the safety measures, which are considered in the design. However, the overstrength reduction factor is defined as ratio between actual F_y and design lateral strength F_d:

$$R_s = \frac{F_y}{F_d} \tag{11}$$

Design lateral strength F_d is herein determined according to the modal analysis taking into account the effect of first vibration mode $\{\phi_1\}$ and corresponding spectral acceleration $S_a(T_1)$:

$$F_d = [M] \cdot \{\phi_1\} \cdot \Gamma \cdot S_a(T_1) \tag{12}$$

4 VARIANTS OF PARTIALLY CODE-CONFORMING STRUCTURES

Six variants of partially code-conforming structures were defined (Table 1) in order to gradually take into account some of factors, which have an impact on the seismic safety of buildings designed according to Eurocodes. The variant 0 ideally satisfy only the fundamental ‘non-collapse’ requirement associated with the design seismic action. This is actually artificial variant of the structure, since it is assumed that it does not have redundancy and overstrength, but only the available ductility, which is equal to the behaviour factor assumed in the design. Consequently, this structure meet the near-collapse limit state in the case of the reference seismic action associated with a reference probability of exceedance, which is herein assumed 10% in 50 years.

All additional structural variants were indeed designed and its performance was assessed with the N2 method (Section 1). Therefore the effect of redundancy is automatically considered in the assessment of factors of safety. Note that each subsequent structural variant takes into account the additional design safety measure, as presented in Table 1. Such escalation of safety measures was defined in order to achieve gradual increase of buildings strength and near-collapse ductility between most of structural variants.

The largest difference between the reinforcement in the columns and beams was observed for variants 1 and 2 due to significantly greater requirements of the Eurocode 8 for minimum amount of longitudinal and transverse reinforcement in the columns and beams. The difference in the reinforcement of columns due to capacity design principles was observed only for transverse reinforcement in bottom stories of the building, since the minimum amount of the reinforcement was adequate for most parts of columns of the 8-storey and 11-storey building. This was not the case of the beams, as expected, since consideration of the capacity design principles caused larger amount transverse reinforcement in all stories. Additional observation was made for variants 3 and 4. In this case the difference in the amount of reinforcement in the columns of the structural variants is practically negligible since the utilization rate (i.e. the ratio between the actual and calculated amount of reinforcement) of the columns was almost equal to 1. However, utilization ratio for the beams was slightly smaller. Therefore actual amount of reinforcement in the beams slightly exceeded the amount of required reinforcement.

Variant of structure	Design safety measures					
	a) design seismic action	b) minimum requirements Eurocode2	c) minimum requirements Eurocode 8	d) actual material strength	e) actual amount of reinforcement	f) capacity design principles
0	x					
1	x	x				
2	x	x	x			
3	x	x	x	x		
4	x	x	x	x	x	
5	x	x	x	x	x	x

Table 1: Description of structural variants associated with gradual consideration of design safety measures.

5 EXAMPLES

5.1 Description of buildings and structural models

The 11-storey and 8-storey reinforced concrete frames (Figures 1 and 2) were designed according to provisions of Eurocode 2 and 8. Both buildings were designed for ductility class M and reference peak ground acceleration of 0.25 g. It was assumed that the 11-storey and the 8-storey building are located on soil type B and C, respectively. The behaviour factor was assumed 3.9. The quality of reinforcing steel was S500B, whereas the concrete C35/45 and C30/37 was used, respectively, for the 11- and 8-storey building. Some global characteristics of structures and design parameters are presented in Table 2.

The structural model in general follows the Eurocode 8 requirements for the modelling of structures as discussed elsewhere [13]. The beam and column flexural behaviour is 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 bi-linear relationship. A linear negative post-capping stiffness is assumed after the maximum moment is achieved. The axial force due to gravity loads is taken into account when determining the moment-rotation relationship for the columns, while in the case of the beams zero axial force and the rectangular 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 [14]. The parameter γ_{el} was assumed to be equal to 1.5. For the structural variant 1 (no seismic detailing), the ultimate rotations were multiplied by a factor of 0.825 [14]. 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. For structural variants No. 3, 4 and 5, the seismic performance assessment based on actual (mean) strength of material. For these variants the mean concrete strength was assumed 8 MPa greater than the characteristic compressive strength, whereas for the steel, the mean strength (570 MPa) was assumed 14 % greater than the characteristic steel strength.

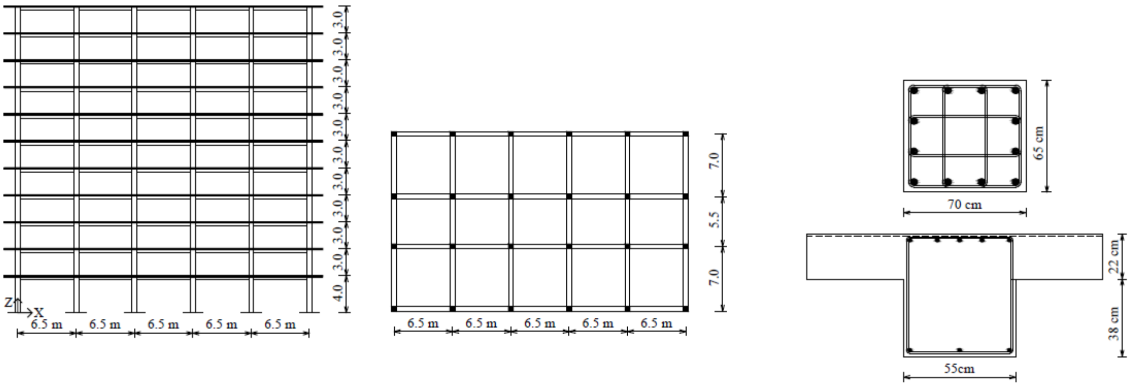

Figure 1: The elevation, plan view and reinforcement in typical columns and beams of code-conformed 11-storey buildings (variant 5).

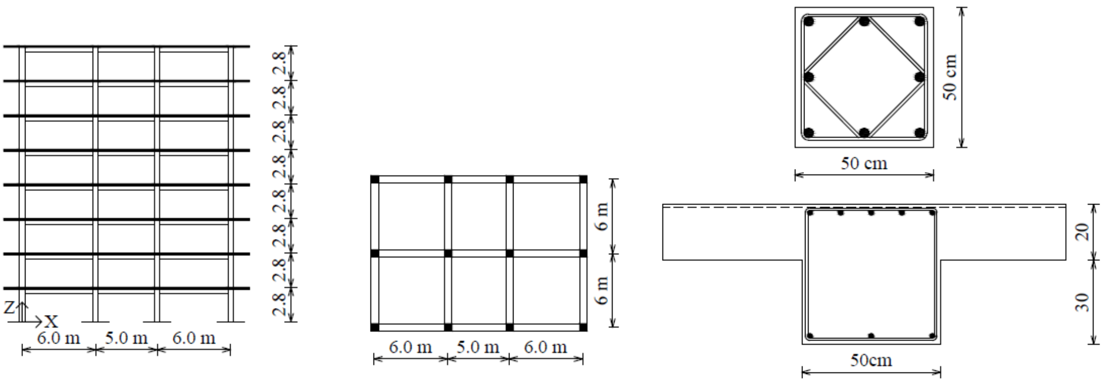

Figure 2: The elevation, plan view and reinforcement in typical columns and beams of code-conformed 8-storey buildings (variant 5).

	Total mass [t]	Period [s]	$a_{g,d,R}$	Soil factor	F_b/W	$F_{b,1}/W$
8-storey	2326	1.45	0.25	1.15	6.5%	6.0%
11-storey	10218	1.92	0.25	1.2	4.6%	3.9%

Table 2: The mass, the first vibration period, the reference peak ground acceleration of on type A ground, the soil factor S, the ratio between the design base shear and the weight and the ratio between the 'first-mode' base shear and the weight of 8-storey and 11-storey buildings.

All the analyses were performed with OpenSees [15], using the PBEE toolbox [13], which is a simple yet effective tool for the seismic performance assessment of reinforced concrete frames by using simplified nonlinear models. The PBEE toolbox includes different functions for the calculation of the moment-rotation relationship of the plastic hinges in the columns and beams, functions for the generation of the tcl input code for OpenSees, functions for the post-processing of the analysis results, and functions for structural performance assessment. More details regarding the PBEE toolbox can be found elsewhere [13].

It should be emphasized that the ultimate rotations in the beams and columns as modelled in this comparative study are lower than the mean values since γ_{el} was assumed to be equal to 1.5. Consequently, the deformation capacity is also underestimated. Additionally, the strength and the stiffness of the beams are also underestimated since the slab effective width was neglected (rectangular sections). Therefore, the expected strength, deformation capacity and the $a_{g,NC}$ of the investigated buildings are likely to be greater than those obtained in this study.

5.2 Pushover analyses

The pushover analyses were performed for X (longitudinal) direction (Figure 1 and 2) using the invariant force vector were which corresponded to product of the storey masses and the first vibration mode ($\Phi_{1,X,11\text{-storey}}$=[0.12 0.24 0.37 0.49 0.60 0.70 0.79 0.86 0.92 0.96 1], $\Phi_{1,X,8\text{-storey}}$=[0.11 0.28 0.44 0.60 0.73 0.84 0.91 1]). The pushover curves and corresponding idealized force-displacement relationship are presented in Figures 3 and 4. Note that the idealized force-displacement relationship ends at displacement which corresponds to the near-collapse limit state, which is obtained when all the rotation in all the columns or beams in one storey exceeds the corresponding ultimate rotation. It should be emphasized that the pushover curve for variant 0 is not available, since it is assumed that variant 0 does not have redundancy and the overstrength, but only the 'available' ductility, which is equal to the behaviour factor assumed in the design.

For the investigated buildings it can be observed that the capacity design principles do not have the impact on the buildings' strength and just small influence on the deformation capacity. On the other hand, it seems that the greatest factor-of-safety can be associated with the material safety factors (see difference between pushover curve corresponding to variant 2 and 3). Moderate impact on the buildings' strength is observed due to the minimum requirements for reinforcement according to Eurocode 8 (variants 1 and 2) and the ratio between the actual (selected) and required (calculated) amount of reinforcements (variants 3 and 4).

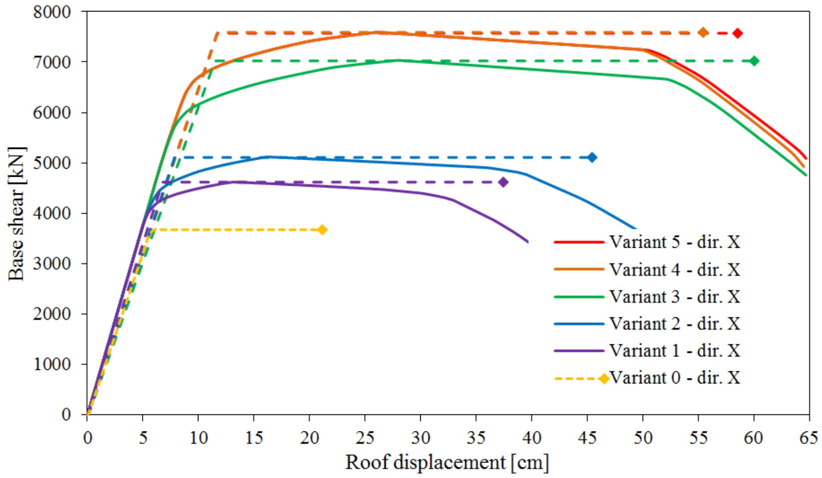

Figure 3: The pushover curves and corresponding idealized force-displacement relationship of the six variants of the 11-storey building.

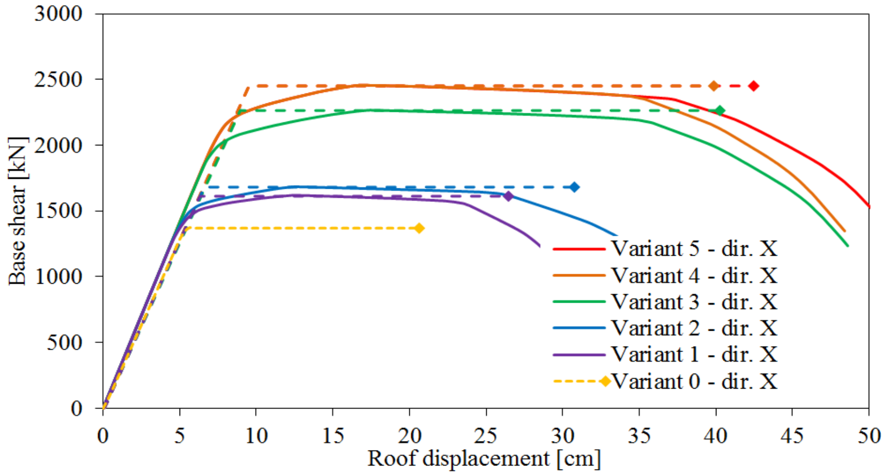

Figure 4: The pushover curves and corresponding idealized force-displacement relationship of the six variants of the 8-storey building.

5.3 Seismic hazard and risk calculation

The seismic hazard at the location of buildings was assessed according to EZ-FRISK [16, 17], which is well-known software for conducting the probabilistic seismic hazard analysis. However, it includes only simplified seismicity model for the central European region (Europe III). This is not consistent with the seismicity models used for the calculation of the seismic hazard maps [18] prescribed by Eurocode 8. Note, that the EZ-FRISK provided greater exceedance rate in this range of peak ground accelerations, whereas the peak ground acceleration associated with the 475-year return period were almost equal to that prescribed for design, which is consistent with the hazard map for 475-year return period. However, since the objective of this study is deaggregation of the seismic safety, the discussion regarding the absolute values of hazard and risk will be omitted.

In addition to the hazard curve, the logarithmic standard deviation β_{NC} of the $a_{g,NC}$ and the slope of the hazard curve k_{NC} associated with the near-collapse limit state has to be assumed.

In this study the standard deviation β_{NC} was assumed equal to 0.6 [9], whereas the the hazard parameter k_{NC}=2.9 was obtained by fitting the hazard curve in log-log coordinates using the method of least.

5.4 Results and discussion

The structural parameters and the performance parameters of the variants of the investigated buildings are presented in Tables 3 and 4, while the corresponding overall factor of safety and the partial factor of safety are presented in Tables 5. The strengths of the structure and the yield displacements gradually increase with respect to the number of safety measures excluded in the performance assessment. However, the gradual increase of structural parameters between the two variants is not general. For example, the available ductility μ_{NC} reduces between the variant 3 and 4, since the increment of the yield displacement is greater as the increment of displacement corresponding to the near-collapse limit state (Figures 3 and 4). However, the effect of the ratio between the actual and the required reinforcement causes positive effect on the strength of the structure and slightly negative effect on the deformation capacity, since the storey drifts associated with the near-collapse limit states of variant 3 (required amount of reinforcement) are more uniformly distributed along the building's height that those corresponding to variant 4. The greatest difference in storey drifts occurred in the central part of the building between 3th and 8th storey and 6th and 7th storey, respectively, for 11-storey and 8-storey building.

The variation of $S_{a,NC}$ and the $a_{g,NC}$ is the consequence of the variation of the overstrength factor and ductility at the near-collapse limit state, since the variation of the estimated period due to the six variants of the structure is almost negligible, and the vibration period exceed T_C. Consequently, the overall factor of safety, if expressed as the ratio between the ground-motion intensity which causes the near-collapse limit state and the intensity associated with the design seismic action, can be determined as the product of the FS_{Rs} and $FS_{R\mu}$, which amounted to 2.47 and 2.07, respectively, for the 11-storey and the 8-storey building.

y	Variant					
	0	1	2	3	4	5
F_y [kN]	3961	4620	5119	7036	7592	7591
D_y [cm]	6.1	6.7	7.8	11.4	11.7	11.7
T^* [s]	2.02	1.96	2.01	2.08	2.02	2.02
$R_{\mu,NC}=\mu_{,NC}$	3.90	5.59	5.83	5.26	4.97	5.03
R_s	1.00	1.17	1.29	1.78	1.92	1.92
q_{NC}	3.9	6.5	7.5	9.3	9.5	9.6
$S_{ae,NC}$ [m/s^2]	1.80	3.01	3.49	4.32	4.40	4.46
$a_{g,NC}$ [g]	0.30	0.48	0.58	0.76	0.73	0.74
P_{NC} [10^{-4}]	95.7	24.1	14.5	6.4	7.2	6.9
$T_{R,NC}$	105	416	692	1550	1392	1441

Table 3: The global structural parameters and performance parameters of the six variants of the 11-storey building.

y	Variant					
	0	1	2	3	4	5
F_y [kN]	1370	1620	1685	2267	2458	2458
D_y [cm]	5.3	6.3	6.5	8.9	9.5	9.5
T^* [s]	1.50	1.50	1.51	1.52	1.50	1.50
$R_{\mu,NC}=\mu_{,NC}$	3.90	4.24	4.71	4.53	4.22	4.49
R_s	1.00	1.18	1.23	1.65	1.79	1.79
q_{NC}	3.9	5.0	5.8	7.5	7.6	8.1
$S_{ae,NC}$ [m/s^2]	2.82	3.62	4.18	5.41	5.46	5.82
$a_{g,NC}$ [g]	0.29	0.37	0.43	0.56	0.56	0.59
P_{NC} [10^{-4}]	95.7	46.1	30.1	14.0	14.0	11.7
$T_{R,NC}$	105	217	333	715	714	857

Table 4: The global structural parameters and performance parameters of the six variants of the 8-storey building.

According to definition of the variants, the FS_{Rs} significantly exceeded the $FS_{R\mu}$. The FS_{Rs} was estimated equal to 1.92 and 1.79 for the 11-storey and 8-storey building, whereas corresponding $FS_{R\mu}$ amounted 1.29 and 1.15, respectively. The overall factors of safety can be further decomposed due to the effect of each safety measure. In this case the effect of each safety measure is assessed by the partial factor of safety. Based on results presented in Table 5, it can be concluded that the partial safety factor due to the redundancy and the minimum design requirements according to Eurocode 2 $\delta FS_{1,Rs}$ is around 1.18, whereas the greatest impact on the partially safety factor was observed due to the effect of the ratio between the mean and design strength of material ($\delta FS_{3,Rs} \approx 1.35$). However, it should be emphasized that the values of partial factors of safety can be different if sequence of safety measures corresponding to the variants of structure would be changed.

The factors of safety, which are incorporated in the design process in addition to that corresponding the design seismic action, actually reduces the probability of exceednace of the near-collapse limit state for a factor of around 10 (i.e. $FS_{TR,NC}$ is 13.8 in the case of 11-storey building and 8.2 in the case of 8-storey building). The major increment of safety is the consequence of the redundancy, minimum requirements for reinforcement and the partial material factors.

Deaggregation of yield strength, ground acceleration causing near-collapse limit state and the return period of the near-collapse limit state, are presented in Figure 5 and 6. It is obvious that the major contribution to the strength of the building is the consequence of the design seismic action, follows the contribution of the mean material strength in conjunction with the ratio between the actual and required amount of reinforcement or the contribution of the redundancy in conjunction with minimum requirements of Eurocode 2, whereas the capacity design principles practically does not contribute to the yield strength or the $a_{g,NC}$. However, deaggregation of $T_{R,NC}$ shows different image, since around 50% of safety is controlled by the the partial factors of strength of material and the other 50% is the consequence of other factors of safety.

11-storey building	δFS_i					FS
	1	2	3	4	5	
$R_{\mu,NC}=\mu_{NC}$	1.43	1.04	0.90	0.94	1.01	1.29
R_s	1.17	1.11	1.37	1.08	1.00	1.92
q_{NC}	1.67	1.16	1.24	1.02	1.01	2.47
$a_{g,NC}$ [g]	1.61	1.19	1.32	0.96	1.01	2.47
P_{NC} [10^{-4}]	0.3	0.6	0.4	1.1	1.0	0.07
$T_{R,NC}$	3.98	1.66	2.24	0.90	1.04	13.78

8-storey building	δFS_i					FS
	1	2	3	4	5	
$R_{\mu,NC}=\mu_{NC}$	1.09	1.11	0.96	0.93	1.06	1.15
R_s	1.18	1.04	1.35	1.08	1.00	1.79
q_{NC}	1.29	1.15	1.30	1.01	1.06	2.07
$a_{g,NC}$ [g]	1.29	1.16	1.30	1.00	1.07	2.07
P_{NC} [10^{-4}]	0.5	0.7	0.5	1.0	0.8	0.12
$T_{R,NC}$	2.08	1.53	2.15	1.00	1.20	8.20

Table 5: The partial safety factors and the overall safety factor for the global structural parameters and performance parameters of the variants of the 11-storey and 8-storey building.

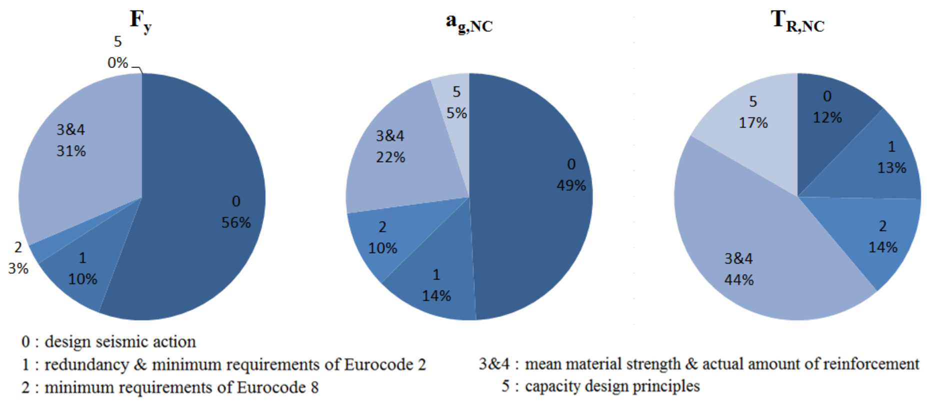

Figure 5: Deaggregation of yield strength, ground acceleration causing near-collapse limit state and the return period of the near-collapse limit state of the 8-storey reinforced concrete building.

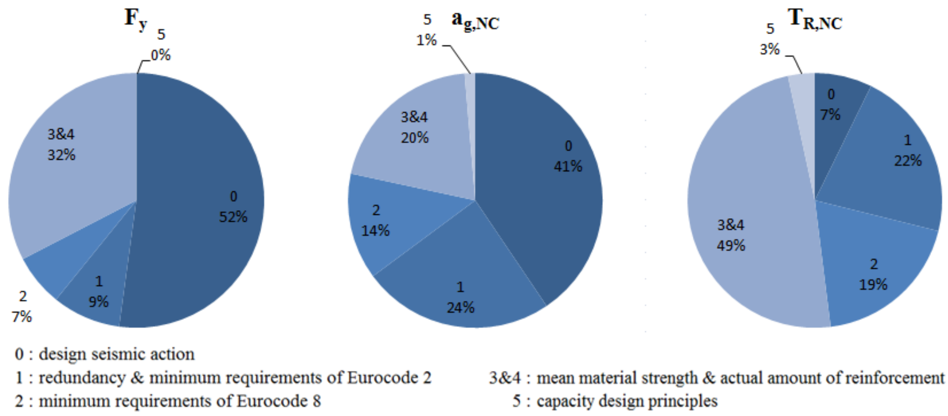

Figure 6: Deaggregation of yield strength, ground acceleration causing near-collapse limit state and the return period of the near-collapse limit state of the 11-storey reinforced concrete building.

6 CONCLUSIONS

Deaggregation of seismic safety in design of two reinforced concrete frame building is investigated in this study on the basis of the pushover analysis gradually taking into account the requirements of the Eurocode 2 and 8, as well as gradually excluding the design assumptions. For these buildings it was shown that the design seismic action has the greatest impact on the yield strength of the structure and the peak ground acceleration which causes the near-collapse limit state. On the other hand, the partial factors of material strength contribute around 50% to the return period of the near-collapse limit state, whereas the contribution of the capacity design principles and the design seismic safety to overall safety is minor. Based on this observation it is argued that it would be better to make the nonlinear model of the building and explicitly design the building for tolerable risk based on several iterations rather to use current standards for earthquake-resistant design, which based on capacity design approach in conjunction with elastic intensity-based assessment.

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