16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1659

Registration Code: S-U1463125412

THE FOMULATION OF RISK-TARGETED BEHAVIOUR FACTOR AND ITS APPLICATION TO REINFORCED CONCRETE BUILDINGS

J. Žižmond(1) and M. Dolšek(2)

(1) *PhD student, University of Ljubljana, Faculty of Civil and Geodetic Engineering, jure.zizmond@fgg.uni-lj.si*
(2) *Professor, University of Ljubljana, Faculty of Civil and Geodetic Engineering, matjaz.dolsek@fgg.uni-lj.si*

Abstract

The derivation of risk-targeted behaviour factor which can be used for the force-based design of structures is presented. The risk-targeted behaviour factor is formulated as a product of the conventional reduction factor and two factors which account for the return period of design seismic action, the target collapse risk and the difference between the shape of acceleration spectra from the demand and capacity domain. Consequently, the behaviour factor is expressed as a function of the target collapse risk, the parameter of seismic hazard, the uncertainty of the seismic response, the ability of the structure to deform in the collapse range and the overstrength factor. The aim of the proposed definition of the behaviour factor is to explain why it is necessary that the behaviour factor be smaller than that estimated from pushover analysis using conventional definition of the behaviour factor. Therefore the proposed definition of the behaviour factor makes it possible to explain the "empirical component" of the values of behaviour factors defined in Eurocode 8. The proposed approach for estimation of the behaviour factor for the force-based design is demonstrated by means of an example of an 8-storey reinforced concrete building. It is hoped that the definition of a risk-targeted behaviour factor may help engineering practitioners and scholars to better understand the origin of the value of behaviour factor and the target response of the structures.

Keywords: risk-targeted behaviour factor; reinforced concrete frames; risk-based design; risk-targeted design intensity; collapse risk.

1. Introduction

The force-based design using linear-elastic analysis is still the most widespread approach for earthquake-resistant design of structure. One of the most important parameters in the design is the reduction factor (in Eurocode 8 [1] termed as behaviour factor) which takes into account the ability of the inelastic behaviour of structure during strong earthquakes. The evaluation, calibration and redefinition of the reduction factor have been going on ever since the concept of reduction of seismic forces is known. Fischinger and Fajfar [2] for example defined the reduction factor as a product of overstrenght and ductility reduction factor. A similar proposal was made by Uang [3]. The determination of the values of reduction factor was also the case of studies of other researchers [4,5]. A comprehensive overview of models for the definition of reduction factors was presented by Miranda and Bertero [6]. At that time the available models of reduction factors were calibrated for the purpose of deterministic analysis, whereas, later on, when risk assessment had become an increasingly acceptable concept, reduction factors were estimated by means of probabilistic frameworks (e.g. [7,8]).

According to the Eurocode 8 the behaviour factor (*q* factor) is used to reduce the seismic forces corresponding to design peak ground acceleration which is, in case of ordinary buildings, defined as a seismic intensity with return period of 475 years. The conventional definition of the behaviour factor is based on the deterministic approach. This may lead to misunderstanding of the concept of the reduction of seismic forces for force-based design since such a definition cannot be used for the interpretation of the concept of the reduction of seismic forces in conjunction with the target collapse risk, although the protection of human lives with an adequate reliability is the fundamental objective of each building code.

To solve this shortcoming, the risk-targeted behaviour factor was recently proposed [9]. It depends on the target collapse risk, the dispersion of limit-state intensities, the difference between spectral shapes of design spectrum and that of the hazard-consistent ground motions, the overstrength reduction factor and the ductility reduction factor. In this paper, the derivation of the risk-targeted behaviour factor is summarized in two steps. In the first step, the risk-targeted design peak ground acceleration, which was already presented in previous study [10], is presented. In the second step, the risk-targeted behaviour factor q is defined. The use of the presented definition of the behaviour factor is then demonstrated by estimating risk-targeted behaviour factor for an 8-storey reinforced concrete frame building.

2. Formulation of structure-specific risk-targeted seismic intensity for force-based design

2.1 The risk-targeted peak ground acceleration causing collapse

The risk-targeted seismic intensity for the performance assessment of structures is introduced by the risk-targeted peak ground acceleration causing collapse a_{gC}. This measure defines target capacity of a structure if it is expressed by the peak ground acceleration. It is actually a parameter of the target collapse fragility function. Note that peak ground acceleration was selected as intensity measure since current seismic hazard maps used in Eurocode 8 are based on peak ground acceleration. However, a similar formulation can be defined for any intensity measure. The derivation of a_{gC} starts with the assumption that the no-collapse requirement is fulfilled when the probability of collapse of a structure P_C is less than the target probability of collapse

$$P_C \le P_t \tag{1}$$

The probability of collapse is defined by the risk equation:

$$P_C \approx \lambda_C = \int_0^\infty P\left(C \middle| A_g = a_g\right) \cdot \left| \frac{dH\left(a_g\right)}{da_g} \right| \cdot da_g \tag{2}$$

where A_g is a random variable representing the peak ground acceleration, $P\left(C \middle| A_g = a_g\right)$ is the collapse fragility function, i.e. the probability that a ground motion where $A_g = a_g$ will cause collapse of a structure, and $H\left(a_g\right)$ is the hazard function which expresses the annual rate of exceedance of a_g. If it is assumed that the hazard function $H\left(a_g\right)$ is linear in the log-log domain ($H\left(a_g\right) = k_0 a_g^{-k}$) and that the collapse intensity is log-normally distributed, then Eq. (2) can be solved in closed form (e.g. [11,12]):

$$P_C = k_0 \cdot a_{gC}^{-k} \cdot e^{\frac{k^2 \cdot \beta_C^2}{2}} = H\left(a_{gC}\right) \cdot e^{\frac{k^2 \cdot \beta_C^2}{2}} \tag{3}$$

where a_{gC} is the risk-targeted peak ground acceleration causing collapse, β_C is the corresponding standard deviation of the natural logarithms, k is the slope of the hazard function in the log-log domain, and k_0 is the annual rate of exceedance of a_g =1 g. In general, the risk-targeted intensity a_{gC} can be assessed iteratively by solving Equation 2, taking into account the assumption that $P_C = P_t$. In this approach there is no need to approximate the hazard function obtained from the probabilistic seismic hazard analysis (e.g. [7,13,14]). However, the shape of the probability distribution curve corresponding to the collapse intensity has to be assumed. In more common cases, when the collapse intensity is represented by a lognormal distribution, the only parameter whose value has to be assumed is the standard deviation β_C. Such a solution is quite general, but it does not provide any insight into the importance of the seismic hazard parameters or into the characteristics of the structure which affect the values of a_{gC}. For this purpose it is convenient to express a_{gC} in closed form by using Eq. (3):

$$a_{gC} = \left(\frac{k_0 \cdot e^{\frac{k^2 \cdot \beta_C^2}{2}}}{P_t} \right)^{\frac{1}{k}} \tag{4}$$

It should be emphasized that risk-targeted peak ground acceleration causing collapse a_{gC} is actually the median value of the peak ground accelerations causing collapse of a structure, and therefore can be classified in the so-called "capacity" domain which actually represents a measure of the capacity of a structure when the latter is expressed in terms of seismic intensities. In order to be able to claim that a structure is safe against collapse, the actual median peak ground acceleration causing collapse of a structure should be greater than a_{gC}.

2.2 Derivation of the risk-targeted intensity for the design of structures using linear elastic analysis

Several different factors have to be considered, whose purpose is to reduce a_{gC} to the risk-targeted peak ground acceleration for the force-based design a_{gD}. All these factors are taken into account by means of a reduction factor *r*, which can be expressed as follows:

$$r = \frac{a_{gC}}{a_{gD}} \tag{5}$$

The acceleration a_{gD} could be estimated from Eq. (5):

$$a_{gD} = \frac{a_{gC}}{r} \tag{6}$$

if an appropriate value of the reduction factor *r* were to be known.

This formulation of a_{gD} is general. In this case the reduction factor *r* has to be estimated according to Eq. (5) by a trial and error procedure, which involves designing of archetype structures from a building class using an assumed value of the factor *r* and a seismic risk assessment of structures until the collapse risk P_C becomes equal to the target collapse risk P_t. Such an approach was recommended in FEMA P695 [7], and its use has also been demonstrated in the case of reinforced concrete frames.

Although the above-mentioned procedure is general, it is useful to have additional insight into the *r* factor. For this reason, the *r* factor is decomposed:

$$r = r_{dc} \cdot r_C \tag{7}$$

where r_{dc} is the demand-to-capacity spectral acceleration ratio, and r_C is a so-called conventional reduction factor, which is formulated in the conventional derivation of the reduction factor using a deterministic approach. Consequently, the risk-targeted peak ground acceleration for the force-based design of structures a_{gD} (Eq. (6)) can be expressed by taking into account Eq. (4) and (7), in the following form:

$$a_{gD} = \frac{a_{gC}}{r_{dc} \cdot r_C} = \left(\frac{k_0 \cdot e^{\frac{k^2 \cdot \beta_C^2}{2}}}{P_t} \right)^{\frac{1}{k}} \cdot \frac{1}{r_{dc} \cdot r_C} \tag{8a,b}$$

As mentioned above the intensity a_{gC} represents the median value of the risk-targeted peak ground accelerations causing collapse of a structure, so that it can be classified in the "capacity" domain (this should be understood as a "target capacity" if capacity of a structure is expressed in terms of seismic intensity) of the structure. In order to define the design value of seismic intensity it is necessary to transform the seismic intensities from the "capacity" to a so-called "demand" domain, which represents the seismic intensities aimed at the force-based design or the selection of hazard-consistent ground motions which are used to estimate the performance of a structure. This transition is defined by reducing a_{gC} to a_{gD}, which is done by means of r_{dc} and r_C.

In order to explain the reduction from a_{gC} to a_{gD} as easy as possible let us introduce a median risk-targeted capacity spectrum which is classified into the "capacity" domain and a risk-targeted design spectrum which is classified into the "demand" domain. The latter type of spectrum is used for force-based design. It is defined by acceleration a_{gD} and shape of spectrum which is often represented by the Newmark-Hall type spectrum (Fig. 1). The median risk-targeted capacity spectrum is defined by acceleration a_{gC} and by the shape of the spectrum (see Fig. 1), which generally differs to that used in the design [9]. It is out of the scope of this paper to precisely define the median risk-targeted capacity spectrum. However, its definition can be simply explained if it is assumed that the structure is already fully defined and that the collapse risk of this structure is equal to target collapse risk. Then the median risk-targeted capacity spectrum is the median spectrum of spectra of ground motions which would cause the damage very close to the collapse of the structure. However, it can be shown that the shape of this spectra is under some assumptions equal to the median spectra from a hazard-consistent set of ground motions used for seismic performance assessment of a structure or in the case of incremental dynamic analysis [9]. In the simplest case, it can also be assumed that the shape of the risk-targeted capacity spectrum and the shape of the spectrum used for design are equal. In this case the r_{dc} is equal to 1. In general, shapes of these spectra differ, as presented in Figure 1.

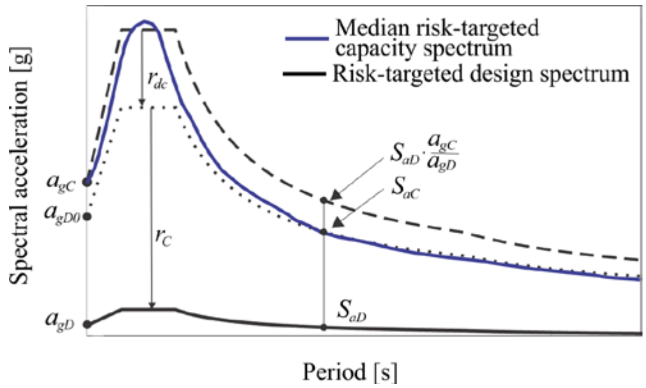

Fig. 1 – The median risk-targeted capacity spectrum (from the "capacity" domain), the risk-targeted design spectrum (from the "demand" domain), and a schematic representation of the gradual transition from the "capacity" domain to the "demand" domain.

In order to explain how the r_{dc} factor can be estimated in the design phase, let us imagine that the risk-targeted design spectrum is first normalized to a_{gC} (the black dashed line shown in Fig. 1). A comparison between the risk-targeted design spectrum and the median risk-targeted capacity spectrum if they are both normalized to a_{gC} reveals the difference between the shapes of these spectra. This difference is accounted for by r_{dc} factor, which is defined as the ratio between the spectral acceleration corresponding to first vibration period obtained from the risk-targeted design spectrum when normalized to a_{gC} and the spectral acceleration corresponding to first vibration period obtained from the median risk-targeted capacity spectrum (see Fig. 1):

$$r_{dc} = \frac{S_{aD}\dfrac{a_{gC}}{a_{gD}}}{S_{aC}} \tag{9}$$

where S_{aC} is the spectral acceleration corresponding to the first vibration period of a structure, which is obtained from the median risk-targeted capacity spectrum, and the S_{aD} is the spectral acceleration corresponding to the same vibration period, which is obtained from the risk-targeted design spectrum. It should be noted that, in the special case, when the seismic intensity measure is to be represented by a spectral acceleration corresponding to the first vibration period of a structure, the formulation can be simplified, since $r_{dc} = 1$ as discussed elsewhere [9].

The peak ground acceleration a_{gD0} (see Fig. 1) is obtained if a_{gC} is divided by r_{dc}. The risk-targeted design spectrum scaled to a_{gD0} (the black dotted line shown in Fig. 1) would have to be used for the design of the structures if structures would not have any ductility and overstrength. Since this is not the case, the a_{gD0} can be further reduced. This reduction is performed using r_C. From Fig. 1 it can be now realized that r_C is defined as the ratio between S_{aC} and S_{aD}. If it is assumed that the relationship between spectral acceleration, forces and displacement can be described by using equivalent SDOF model, then it can be shown [9] that the reduction factor r_C is also a product of an overstrength reduction factor r_s and a ductility reduction factor r_μ [2]:

$$r_C = \frac{S_{aC}}{S_{aD}} = r_\mu \cdot r_s = \frac{\mu_C}{C_1} \cdot \frac{F_y}{F_{d,1}} \tag{10}$$

The ductility reduction factor r_μ can be defined as the ratio between the available system ductility μ_C (i.e. the ratio between the displacement of the structure at the collapse and the corresponding yield displacement) and the inelastic deformation ratio C_1 ([15,16]), whereas the r_s can be interpreted as the ratio between the yield strength (F_y) and the design base shear associated with the first vibration mode ($F_{d,1}$). Note that the C_1 from Eq.(10) should be estimated as the ratio between the displacement corresponding to the collapse of the nonlinear SDOF model and the displacement of the linear elastic SDOF model when subjected to the risk-targeted seismic intensity causing collapse of the nonlinear SDOF model.

Taking into account Eq. (10) the risk-targeted peak ground acceleration can be formulated as follows:

$$a_{gD} = \frac{a_{gC}}{r_{dc} \cdot r_\mu \cdot r_s} = \frac{a_{gC}}{r_{dc}} \cdot \frac{C_1}{\mu_C} \cdot \frac{1}{r_s} \tag{11}$$

3. Formulation of risk-targeted behaviour factor

For simplicity of the derivation of the behaviour factor q on the basis of a probabilistic approach, it is assumed that the behaviour factor applies to the entire range of the acceleration spectrum, and not only to periods greater than the period T_B, which represents the lower limit of the period of the constant spectral acceleration branch [1]. The peak ground acceleration for design a_{gD} can thus be expressed as the ratio between the hazard-targeted peak ground acceleration determined from seismic hazard maps a_{gTR}, where T_R is the return period of the seismic action (e.g. 475 years in the case of ordinary buildings, according to Eurocode 8), and the behaviour factor q:

$$a_{gD} = \frac{a_{gTR}}{q} \tag{12}$$

In the probabilistic approach, a_{gD} is the risk-targeted peak ground acceleration defined, for example, by Eq. (8a,b). The behaviour factor can then be expressed as the ratio between the two intensity levels multiplied by the reduction factor:

$$q = \frac{a_{gTR}}{a_{gC}} \cdot r_{dc} \cdot r_C \tag{13}$$

The ratio of the peak ground acceleration determined from the seismic hazard map a_{gTR}, and the risk-target peak ground acceleration causing collapse of a structure a_{gC}, defines the correction factor due to the risk-targeted definition of the behaviour factor:

$$C_P = \frac{a_{gTR}\left(H\left(a_g\right), T_R\right)}{a_{gC}\left(P_t, H\left(a_g\right), \beta_C\right)} \tag{14}$$

The correction factor C_P can be formulated in different forms. For example, if a_{gC} is expressed using Eq. (4) and a_{gTR}, which is associated with return period $T_R = 1/H\left(a_{g,TR}\right)$, is determined from the approximate hazard function ($H\left(a_g\right) = k_0 a_g^{-k}$) as follows:

$$a_{gTR} = \left(k_0 T_R\right)^{\frac{1}{k}} \tag{15}$$

then the correction factor C_P can be formulated in the closed-form solution:

$$C_P = \left(T_R \cdot P_t\right)^{\frac{1}{k}} \cdot e^{-\frac{k\beta_C^2}{2}} \tag{16}$$

If correction factor C_P is introduced into Eq. (13) and by taking into account that $r_C = r_\mu \cdot r_s$, then the risk-targeted behaviour factor, as defined by Eq.(13), can be expressed in the following form:

$$q = r_{dc} \cdot r_\mu \cdot r_s \cdot C_P \tag{17}$$

From Eq. (17) by taking into account, for example, Eq. (16), it can be seen that the behaviour factor, when defined with consideration of a probabilistic framework, does not depend just on the product of the ductility reduction factor r_μ and the overstrength factor r_s but it is also affected by the return period T_R, the target collapse risk P_t, the seismic hazard function (in this simplest case by the slope of the hazard curve k), the standard deviation of the logarithm of the peak ground accelerations causing collapse β_C, and r_{dc} factor, which is in a special case equal to 1.

4. Example of calculation of risk-targeted behaviour factor

4.1 Description of investigated building

The geometry of the 8-storey reinforced concrete frame building is presented in Fig. 2a. The building is located in Ljubljana (Slovenia) on soil type A, where the peak ground acceleration corresponding to return period of 475 years is equal to 0.25 g. The building consists of three bays in the X direction and two bays in the Y direction. The columns of the middle frame in the X direction (columns C5-C8) have a cross-section of 55/55 cm, whereas the others (columns C1-C4, C9-C12) are 50/50 cm. The quality of reinforcing steel was prescribed as S500B, whereas concrete class of C30/37 was used. The slab depth was 20 cm. The total mass of structure amounted to 2338 t. The first vibration periods in the Y and X directions, respectively, were 1.28 s and 1.23 s.

4.2 Target collapse risk and the corresponding risk-targeted behaviour *q*

The calculation of risk-targeted behaviour factor q involves five steps: (1) Define the target collapse risk P_t and the reference return period T_R, (2) Define the seismic hazard at the site of the building, (3) Calculate risk-targeted peak ground acceleration causing collapse a_{gC}, determine a_{gTR} from the seismic hazard maps and calculate correction factor C_P, (4) Assume values for overstrength r_s and ductility of structure μ_C and calculate r_{dc}, (5) Calculate behaviour factor q.

Step 1: The target collapse risk was set to P_t $=5\cdot10^{-5}$ (0.25% in 50 years). Note that this risk is 4 times lower than that defined by the US building code, but around 4 times greater than that estimated on the basis of a survey about the tolerable probabilities of collapse for ordinary structures in Slovenia [17]. The return period was set to T_R=475 years since design peak ground according to the current version of Eurocode 8 [1] corresponds to that return period.

Step 2: The hazard curve (Fig. 2b) was calculated by using a methodology which was used for determination of seismic hazard maps for Slovenia [18]. The hazard curve was then fitted by means of a linear function in the log domain using appropriate acceleration intervals [0.20 g 1.80 g]. The obtained parameters of the hazard function correspond to k=5.8 and k_0=$1.4\cdot10^{-6}$. It has to be noted that there is no need to use linear hazard function as discussed in the following.

Step 3: The correction factor C_P was assessed iteratively using Eq. (14) and on the basis of an approximate closed-form solution (Eq. (16)). In the first case, the a_{gC} was assessed iteratively by solving Eq. (2), taking into account the assumption that $P_C = P_t$ and the entire non-fitted hazard curve, and by assuming a lognormal distribution function for the collapse fragility function $P\left(C \middle| A_g = a_g\right)$. The standard deviation of the logarithm of the peak ground accelerations causing collapse was assumed to be equal to β_C =0.60 ([19,20]). For this particular example a_{gC} was estimated to amount to 1.23 g. Note that a_{gC} represents target median value of the peak ground accelerations causing collapse of a structure. The acceleration a_{gTR} = 0.25 g was obtained directly from seismic hazard curve. Note that this value of a_{gTR} is equal to the value of the peak ground acceleration from seismic hazard map for Ljubljana (Slovenia) for return period 475 years. The correction factor C_P=0.19 was then calculated as a ration between a_{gTR} and a_{gC}. Slightly smaller value of C_P was obtained on the basis of an approximate closed-form solution (Eq. (14)):

$$C_P = \left(T_R \cdot P_t\right)^{\frac{1}{k}} \cdot e^{-\frac{k\beta_C^2}{2}} = \left(475 \cdot 5 \cdot 10^{-5}\right)^{\frac{1}{5.8}} \cdot e^{-\frac{5.8 \cdot 0.60^2}{2}} = 0.18 \tag{18}$$

It can be observed that there is small difference between the two values of correction factor C_P. The difference is a consequence of the linearization of seismic hazard curve. Hereinafter the value of correction factor obtained by iterative solution of Eq. (14) (C_P=0.19) will be used.

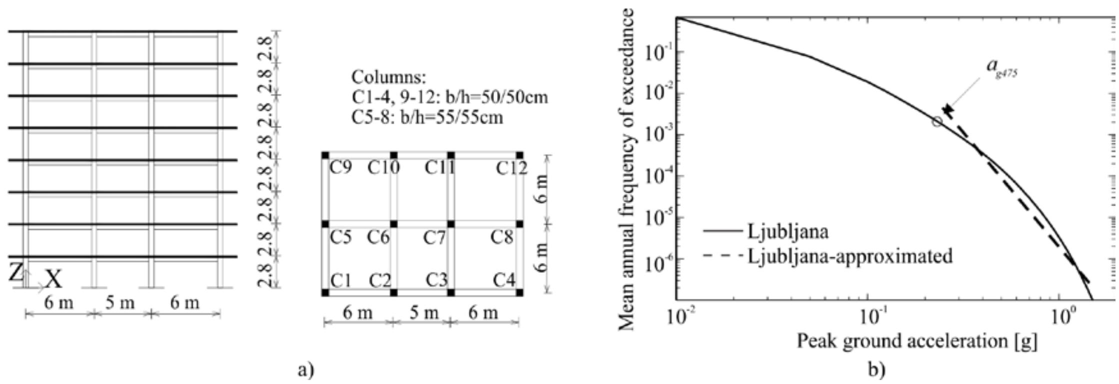

Fig. 2 – a) Elevation and plan views of the investigated 8-storey building, and b) the hazard function for Ljubljana (Slovenia) and the approximated linear hazard function in the log domain.

Step 4: An overstrength and ductility of structure was assumed as r_s=2 and μ_C=8. Such a values were estimated on the basis of the results of assessments of structures in previous studies, where it was found that, in the case of multi-storey reinforced concrete frame buildings designed according to Eurocodes 2 and 8, typical values of r_s vary from around 2 to 3 whereas the values of μ_C vary from around 6 to 8 ([21,22]).

The ductility of structure μ_C has to be transformed to ductility reduction factor r_μ. Different approaches can be used. Since the period of the investigated building is greater than T_C, the value of ductility reduction factor r_μ can be assumed equal to μ_C since it is well known that for such structures the displacements from linear elastic models and those corresponding to the nonlinear models are approximately equal. However, hereinafter the ductility reduction factor r_μ will be obtain as a ration between ductility μ_C and inelastic displacement ratio C_1. In order to estimate the inelastic displacement ratio, the nonlinear SDOF model has to be defined and the hazard consistent set of ground motion, which is intended to be used for seismic performance assessment, has to be selected. It is assumed that the period of the SDOF model is equal to the first fundamental period of structure, the ductility of SDOF model will correspond to assumed ductility μ_C and that the collapse intensity of SDOF model will be in the order of magnitude of a_{gC} (Fig. 4a).

The hazard-consistent set of ground motions were selected using conditional spectrum approach [23,24] taking into account mean magnitude M= 6.7, mean distance R= 7 km, the peak ground acceleration a_{gC} = 1.23 g and Sabetta&Pugliese attenuation relationship [25]. Note that since a_{gC} is used as an intensity measure the conditional period corresponds to T^*= 0 s. All of the 30 ground motions (Fig. 3a) corresponds to events with magnitudes of between 4.5 and 7, and source-to-site distances of between 5 and 50 km and were recorded on soil having a shear-wave velocity in the upper 30 m upper to 600 m/s.

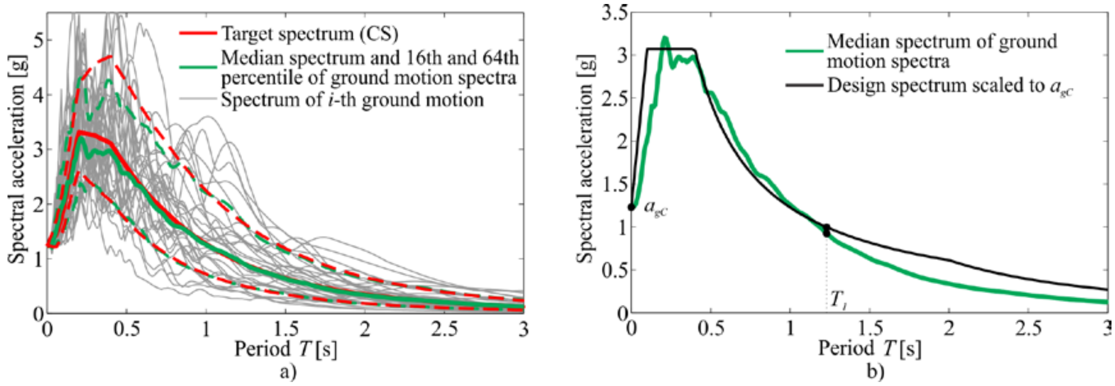

Fig. 3 – a) the target median, 16th and 84th percentile spectra of hazard-consistent ground motions obtained by conditional spectrum approach for an earthquake scenario based on a_{gC} and the corresponding spectra of the selected ground motions from databases, and b) median spectrum of the selected hazard-consistent ground motions compared to design spectrum (Eurocode 8 elastic spectrum) scaled to a_{gC}. The difference between two spectra at spectral acceleration corresponding to first vibration mode define factor r_{dc}.

The inelastic deformation ratio C_1 is defined as the ratio between the displacement at the collapse of the nonlinear SDOF model (D^*_{nC}) and the displacement of the linear elastic SDOF model when subjected to $\tilde{S}_{aC}$ (D^*_{eC}). In this case, $\tilde{S}_{aC}$ has to be estimated as the median value of intensities causing collapse, which are, for example, obtained from incremental dynamic analysis [26] using a nonlinear SDOF model and the hazard-consistent set of ground motions. The displacement of the elastic model is then calculated by using the formula which defines the relationships between the spectral acceleration and the displacement of the elastic system ($S_{aC} = \omega^2 \cdot D^*_{eC}$). The displacement D^*_{nC} can be simply determined from the force-displacement relationship of the SDOF model or from results of IDA.

For this SDOF model (Fig. 4a) and the set of ground motions (Fig. 3a) the observed displacement of the elastic model (D^*_{eC} = 33.0 cm) was slightly greater than the inelastic displacement (D^*_{nC} = 28.9 cm) (Fig. 4b). Consequently the inelastic deformation ratio C_1 was smaller than 1 (C_1=0.88). This phenomenon is a consequence of the use of the conditional spectrum approach for the selection of hazard-consistent ground motions. Once the inelastic deformation ratio C_1 was assessed, the ductility reduction factor can be simply obtained as a $r_\mu = \mu_C / C_1$=8.0/0.88=9.14.

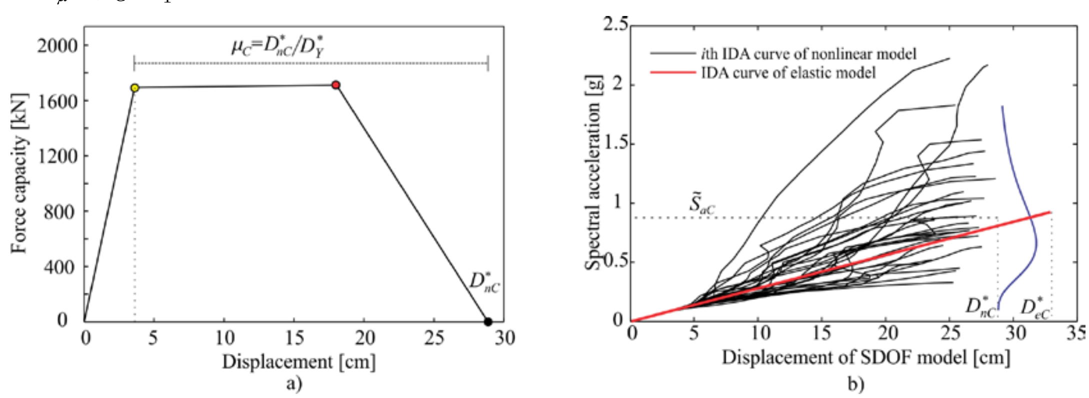

Fig. 4 – a) the force-displacement relationship of SDOF model and b) IDA curves using the nonlinear and elastic SDOF models

The demand-to-capacity spectral acceleration ratio r_{dc}=1.08 was calculated as a ratio between the spectral acceleration corresponding to first vibration mode (T_1=1.23 s) from the median spectrum corresponding to the ground motions that were selected on the basis of the conditional mean spectrum (Fig. 3b) and spectral acceleration corresponding to first vibration mode from risk-targeted design spectrum scaled to a_{gC}.

The shape of the risk-targeted design spectrum was assumed the same as the shape of elastic spectrum of Eurocode 8. It is important to emphasized that the value of r_{dc} was assessed on the basis of median spectra from the selected hazard-consistent ground motions. In the design phase this is the only possible approach, which is quite correct since often the same ground motions would be used for estimating the fragility function of the structure [9].

Step 5: Risk-targeted behaviour factor *q* (see Equation 19) is calculated by taking into account C_P=0.19, r_{dc} =1.08, r_s =2 and r_μ =9.14.

$$q = C_P \cdot r_{dc} \cdot r_s \cdot r_\mu = 0.19 \cdot 1.08 \cdot 2 \cdot 9.14 = 3.7 \tag{19}$$

Finally, the risk-targeted design peak ground acceleration can be calculated as follows (Eq. (12)):

$$a_{gD} = \frac{a_{g475}}{q} = \frac{0.25\,g}{3.7} = 0.067\,g \tag{20}$$

The result of the behaviour factor is very similar to that prescribed in Eurocode 8 [1] for this type of structures and ductility class medium (*q*=3.9). For example, the risk-targeted design spectrum as well as design spectrum of Eurocode 8 are presented in Fig. 5. It can be observed that the two design spectra are practically the same for the periods of interest. Consequently, the design seismic forces and the resulted structures would be more or less the same. It is also interesting to note, that in the case if P_t would be set to 10^{-4} (0.5% in 50 years) and if it is assumed that other factors remain practically equal to those used in the example (r_{dc}=1.08, r_s=2 and r_μ=9.14), then the value of the estimated behaviour factor would be equal to 4.6. This value is about 25 % grater then the value of behaviour factor which corresponds to target collapse risk $P_t = 5{\cdot}10^{-5}$.

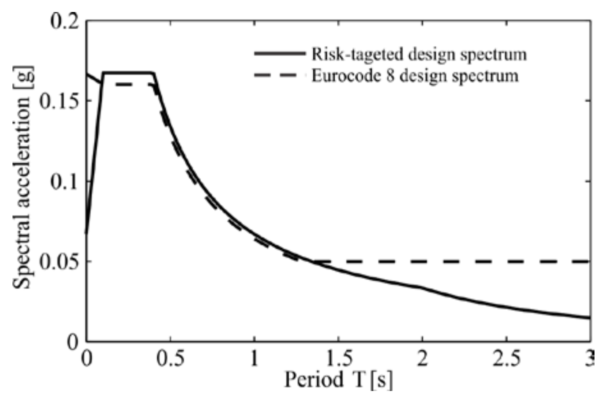

Fig. 5 – The risk-targeted design spectrum and Eurocode 8 design spectrum

4. Conclusions

The behaviour factor using probabilistic framework, as defined in this paper, represents a firm scientific basis for the estimation of the behaviour factor in future generation of building codes or for new structural systems which are not considered in currently applicable building codes. The proposed definition of behaviour factor explicitly takes into account the target collapse risk, the seismic hazard, the ability of structures to deform in the nonlinear range, the overstrength factor, and the uncertainty in the seismic response of structures. In order to simplify the calculation of risk-targeted behaviour factor in practical applications, the web application was recently developed. It can be accessed from www.smartengineering.si.

In the example it was shown that the difference between the behaviour factor, which was estimated in the presented example by selecting target collapse risk equal to 0.25% in 50 years, and that from Eurocode 8 is practically negligible. By double the target collapse risk, the behaviour factor increased for about 25%. Therefore it can be argued that the proposed probabilistic framework for the estimation of the behaviour factor can be used for explaining the relationship between all important factors in the design of structures. From this point of view there is no need to claim that the behaviour factor is determined empirically, as it is usually interpreted (e.g. [27]).

In the presented calculations it is also demonstrated that the target value of median peak ground acceleration causing collapse of structure has to be several times greater than the peak ground acceleration which corresponds to return period of 475 years. Moreover, it can be concluded that the behaviour factor q has to be much smaller than he product of overstrength and ductility reduction factor in order to protect human lives with an adequate level of reliability. Such a conclusion is very important for engineering practitioners and/or scholars that are developing the regulations for new structural types or new standards.

In the more general case, the proposed procedure for the definition of risk-targeted behaviour factor (or design seismic intensities) could be used as a part of an iterative risk-based design [28], where risk-targeted force-based design is used to define an initial structure. Consequently, the accuracy of estimating the risk-targeted behaviour factor (or design seismic intensity) becomes less important, since it affects only the number of iterations which have to be performed within the risk-based design. With some experience and good understanding of the problem it makes sense to use such a procedure for the design of important structures, where it is necessary to check the assumptions of force-based design by using nonlinear methods of analysis.

5. Acknowledgements

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

6. References

[1] CEN (2004): European standard EN 1998–1:2004. Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic action and rules for buildings. European Committee for Standardization, Brussels.

[2] Fischinger M, Fajfar P (1990): On the response modification factors for reinforced concrete buildings. *4th US National Conference on Earthquake Engineering*, Palm Springs, California, USA, pp. 1–10.

[3] Uang CM, (1991): Establishing R (or Rw) and Cd factors for building seismic provisions. *Journal of Structural Engineering*, **117** (1), 19–28.

[4] Fischinger M, Fajfar P, Vidic T (1994): Factors contributing to the response reduction. *5th US National Conference on Earthquake Engineering*, Chicago, Illinois, USA, pp. 97–106.

[5] Elnashai AS, Mwafy AM (2002): Overstrength and force reduction factors of multistorey reinforced-concrete buildings. *Structural Design of Tall Buildings*, **11** (5), 329–351.

[6] Miranda E, Bertero VV (1994): Evaluation of strength reduction factors for earthquake-resistant design. *Earthquake Spectra*, **10** (2), 357–379.

[7] ATC (2009): FEMA P695, Quantification of building seismic performance factors, Federal Emergency Management Agency, Washington D.C.

[8] Chryssanthopoulos MK, Dymiotis C, Kappos AJ (2000): Probabilistic evaluation of behaviour factors in EC8-designed R/C frames. *Engineering Structures*, **22** (8), 1028–1041.

[9] Žižmond J (2016): Force-based seismic design of reinforced concrete buildings for target reliability. *Ph.D. Dissertation* (in preparation), Faculty of Civil and Geodetic Engineering, University of Ljubljana.

[10] Žižmond J, Dolšek M (2015): Risk-targeted force-based design and performance check by means of nonlinear analysis. *10th Pacific Conference on Earthquake Engineering*, Sydney, Australia, pp. 1–8.

[11] Cornell CA (1996): Calculating building seismic performance reliability: A basis for multi-level design norms. 11th *World Conference on Earthquake Engineering*, Acapulco, Mexico, pp. 1–8.

[12] McGuire RK (2004): *Seismic hazard and risk analysis*. Earthquake Engineering Research Institute.

[13] Luco N, Ellingwood BR, Hamburger RO, Hooper JD, Kimball JK, Kircher CA (2007): Risk-targeted versus current seismic design maps for the conterminous United States. *SEAOC 2007 Convention proceedings*, Squaw Creek, California, USA, pp. 1–13.

[14] Douglas J, Ulrich T, Negulescu C (2013): Risk-targeted seismic design maps for mainland France. *Natural Hazards*, **65** (3), 1999–2013.

[15] Miranda E (2001): Estimation of Inelastic Deformation Demands of SDOF Systems. *Journal of Structural Engineering*, **127** (9), 1005–1012.

[16] Dolšek M, Fajfar P (2004): Inelastic spectra for infilled reinforced concrete frames. *Earthquake Engineering & Structural Dynamic*, **33** (15), 1395–1416.

[17] Fajfar P, Polič M, Klinc R (2014): Zaznavanje potresne ogroženosti pri strokovnjakih in nestrokovnjakih (Perception of seismic risk by experts and lay people). *Gradbeni Vestnik*, **63** (5) 111–118.

[18] Lapajne J, Šket Motnikar B, Zupančič P (2003): Probabilistic seismic hazard assessment methodology for distributed seismicity. *Bulletin of the Seismological Society of America*, **93** (6), 2502–2515.

[19] Dolšek M (2012): Simplified method for seismic risk assessment of buildings with consideration of aleatory and epistemic uncertainty. *Structural and Infrastructural Engineering*, **8** (10), 939–953.

[20] Lazar N, Dolšek M (2014): Incorporating intensity bounds for assessing the seismic safety of structures: Does it matter?. *Earthquake Engineering & Structural Dynamic*, **43** (5), 717–738.

[21] Žižmond J, Podgorelec D, Dolšek M (2014), Relationship between the collapse risk and the reinforced concrete frame structure. *2nd European conference on earthquake engineering and seismology*, Istanbul, Turkey.

[22] Žižmond J, Dolšek M (2015): Evaluation of factors influencing the earthquake-resistant design of reinforced concrete frames according to Eurocode 8. *Structural and Infrastructural Engineering*, doi: 10.1080/15732479.2015.1117112.

[23] Baker JW (2011): Conditional mean spectrum: Tool for ground-motion selection. *Journal of Structural Engineering*, **137** (3), 322–331.

[24] Jayaram N, Lin T, Baker JW (2011): A computationally efficient ground-motion selection algorithm for matching a target response spectrum mean and variance. *Earthquake Spectra*, **27** (3), 797–815.

[25] Sabetta F, Pugliese A, (1996): Estimation of response spectra and simulation of nonstationary earthquake ground motions. *Bulletin of the Seismological Society of America*, **86** (2), 337–352.

[26] Vamvatsikos D, Cornell CA (2002): Incremental dynamic analysis. *Earthquake Engineering & Structural Dynamic*, **31** (3), 491–514.

[27] Fardis MN, Carvalho EC, Fajfar P, Pecker A (2015): *Seismic Design of Concrete Buildings to Eurocode 8*. CRC Press 1st ed.

[28] Lazar Sinković N, Brozovič M, Dolšek M (2016): Risk-based seismic design for collapse safety. *Earthquake Engineering & Structural Dynamic*, doi: 10.1002/eqe.2717.