Proceedings of the Tenth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Pacific
6-8 November 2015, Sydney, Australia

Risk-targeted force-based design and performance check by means of nonlinear analysis

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ABSTRACT: The derivation of risk-targeted seismic intensity for the force-based design of structures is presented with emphasis on the target collapse risk and the closed form solution, which provides an insight into the risk-targeted seismic intensity. The latter is expressed as a function of the target collapse risk, the parameters of seismic hazard, the uncertainty of the seismic response, the ability of the structure to deform in the range near to collapse and the overstrength factor. The calculation of risk-targeted peak ground acceleration for force-based design is then demonstrated by means of an example of an 8-storey reinforced concrete building. The structure is designed and its performance is checked by relatively simple nonlinear analysis which confirmed the adequacy of the assumptions made in the proposed design. The proposed design is a simple extension of current state of practice but it allows design for explicitly defined target collapse risk. Definition of a risk-targeted seismic intensity for the force-based design of structures may help engineering practitioners and scholars to better understand the concept of the reduction of seismic forces for the design of structures when using linear elastic analysis.

1 INTRODUCTION

Over the last decade the risk-targeted design of buildings has become increasingly popular, particularly since, in the opinion of authors, designs based on an explicitly defined target risk are conceptually more correct than that commonly used in the current state of practice. In Europe design seismic actions are still based on uniform hazard maps (Eurocode 8 (CEN 2004)), whereas the ASCE 7-10 (2010) already considers risk-targeted seismic design maps. Some attempts have been made in Europe towards the introduction of risk-targeted seismic design maps (e.g. Douglas et al. 2013, Silva et al. 2015). However, even when a structure is designed to a target probability of collapse using direct design approach, it is difficult to claim that the collapse risk of a structure is less or equal to the target collapse risk since direct design approaches are based on several assumptions. For this reason force-based design, as well as other direct seismic design procedures, will always be approximate. The behaviour factor is probably the most uncertain parameter in direct force-based design. However, the force-based design is approximate to some extent also due to certain other assumptions. For example, the effective period of a structure is not precisely known until its strength has been more precisely determined (e.g. Priestley et al. 2007). Thus, especially for more important buildings, it would make sense to check the design by using nonlinear methods of analysis. Pushover-based methods (e.g. Fajfar & Dolšek 2012) are quite convenient for checking design assumptions at least for some types of structures, whereas in the more general case, the design can be checked by using nonlinear dynamic analysis. Recently the concept of intensity-based assessment for risk-based decision-making was introduced and realized with an efficient 3R method (Dolšek & Brozovič 2015), which requires nonlinear dynamic analysis only for a few (e.g. seven) so-called characteristic ground motions. The method is based on a trivial decision model so that the analyst can decide that the structure is safe against collapse if the latter occurs for less than half of characteristic ground motions.

However, it is probably not practical to check performance by nonlinear analysis for all types of structures. Lazar Sinkovič et al. (2015) recently proposed a risk-based design procedure which allows differentiation of the reliability of the design with respect to the importance of the structure observed. Escalation of the reliability of design was related to the method used for analysis or seismic performance assessment. Design Level 0 does not require any analysis of the structure. Therefore the reliability of a design corresponding to Level 0 is too low, and can be used only to define an initial structural configuration in conjunction with other design levels. Design Level 1 is based on linear elastic analysis, whereas pushover-based methods and nonlinear dynamic analysis are required for design Levels 2 and 3, respectively. In general risk-based design is an iterative process, but iterations may not be required if the initial structural configuration is obtained by adequately calibrated force-based design.

The aim of this paper is to present the formulation of risk-targeted peak ground acceleration in closed form (Žižmond & Dolšek 2015), which accounts for all the key parameters that have an impact on force-based design (from the target collapse risk to forces and displacements). This formulation can be used to apply force-based design in the case of an explicitly selected target probability of collapse. A step-by-step approach is used in order to demonstrate how the design peak ground acceleration is calculated for an 8-storey reinforced concrete frame building. The structure is then designed. A simplified version of performance check is shown at the end of the paper. It involves evaluation of the reduction factor, which is assumed in the process of design.

2 FORMULATION OF STRUCTURE-SPECIFIC RISK-TARGETED SEISMIC INTENSITY FOR FORCE-BASED DESIGN

In this section 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 intensity measure was selected since Eurocode's hazard maps are based on peak ground acceleration. Note that a similar formulation can be defined for any intensity measure. It is assumed, firstly, 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_t

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

The probability of collapse can be estimated by applying 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 Equation 2 can be solved in closed form (e.g. Cornell 1996; McGuire 2004):

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

where a_{gC} is the median 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. Luco et al. 2007; FEMA P695 2009; Douglas et al. 2013). 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 Equation 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 a_{gC} is 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} .

2.1 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 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 Equation 5:

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

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

This formulation of a_{gD} is general. In this case the reduction factor has to be estimated according to Equation 5 by a trial and error procedure, which involves designing the structure using an assumed value of the factor r and a seismic risk assessment of the structure until the collapse risk P_C becomes equal to the target collapse risk P_t . Such an approach was recommended in FEMA P695 (2009), and its use has also been demonstrated in the case of reinforced concrete frames.

Although the above-mentioned procedure is general, it is sometimes useful to have some additional insight into the r factor. For this reason decomposition of the r factor could be useful. The following formulation of the r factor was recently proposed by the authors (Žižmond & Dolšek 2015):

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

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} (Equation 6) can be expressed by taking into account Equations 4 and 7, in the following form:

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

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" domain) of the structure. In order to define the design 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 . The reduction from a_{gC} to a_{gD} can be explained by introducing a median risk-targeted capacity spectrum and a risk-targeted design spectrum from the "demand" domain. The latter type of spectrum is used for force-based design. It is often represented by the Newmark-Hall type spectrum (Fig. 1), whereas the shape of the median risk-targeted capacity spectrum (see Fig. 1) is not exactly known during the design phase. However, in the case of a defined structure it can be obtained by calculating the median spectrum on the basis of the elastic spectra of those ground motions which would cause the collapse of the structure. From Figure 1 it can be seen that the median risk-targeted capacity spectrum is normalized to a_{gC} since this is an objective in the design, whereas the risk-targeted design spectrum is conditioned to a_{gD} .

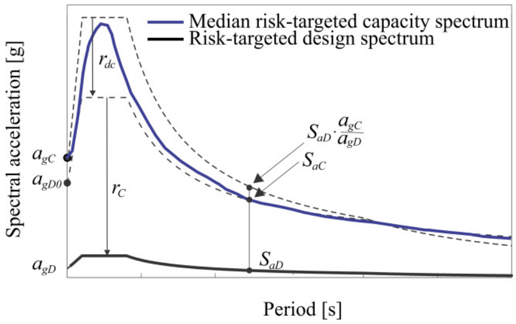

Figure 1: The median risk-targeted capacity spectrum (from the "capacity" domain), the risk-targeted design spectrum (from the "demand" domain), and a schematic representation of the gradual transition from the "capacity" domain to the "demand" domain.

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 normalized to a_{gC} (the dashed line shown in Figure 1). A comparison between the risk-targeted design spectrum and the median risk-targeted capacity spectrum when both are normalized to a_{gC} reveals the difference between the shapes of these spectra. This difference is accounted for by the reduction factor r_{dc}, which is defined as the ratio between the spectral acceleration obtained from the risk-targeted design spectrum when normalized to a_{gC} and the spectral acceleration 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 (Žižmond & Dolšek 2015).

Value of a_{gD0} is obtained if a_{gC} is divided by r_{dc}. However, a_{gD0} can be further reduced since every structure has some ductility and overstrength. This reduction is performed using r_C. From Figure 1 it can be seen that r_C is defined as the ratio between S_{aC} and S_{aD}. If the equivalent SDOF model is used to estimate the response of a structure, then the reduction factor r_C can also be formulated as a product of an overstrength reduction factor r_s and a ductility reduction factor r_μ (Fischinger & Fajfar 1990). Furthermore it can be shown (Žižmond & Dolšek 2015) that r_μ can be expressed as the ratio between the available system ductility μ_C (i.e. the ratio between the collapse displacement of the structure and the corresponding yield displacement), which has to be ensured in the design, and the inelastic deformation ratio C_1 (Miranda 2001; Dolšek & Fajfar 2004), which is defined as the ratio between the collapse displacement 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. The overstrength factor 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 ($F_{d,1}$). Thus the reduction factor r_C can be expressed as follows:

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

Taking into account Equation 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 EXAMPLE: CALCULATION OF RISK-TARGETED PEAK GROUND ACCELERATION AND DESIGN CHECK FOR AN 8-STOREY RC FRAME BUILDING

3.1 Description of the investigated building

The geometry of the 8-storey reinforced concrete frame building is presented in Figure 2a. The building will be located in Ljubljana (Slovenia) on soil type A, where the peak ground acceleration corresponding to a 475 year return period is 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 C5-8) have a cross-section of 55/55 cm, whereas the others (columns C1-4, 9-12) 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 X and Y directions, respectively, were 1.26 s and 1.28 s.

3.2 The target probability of collapse and the risk-targeted peak ground accelerations

The calculation of a_{gD} involves five steps: (1) Define the target collapse risk P_t, (2) Define the seismic hazard at the site of the building, (3) Calculate a_{gC}, (4) Assume values for r_C and calculate r_{dc}, and (5) Calculate a_{gD}.

Step 1: The target collapse risk was set to $P_t = 5 \cdot 10^{-5}$ (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 (Fajfar et al. 2014) (i.e. $1.1 \cdot 10^{-5}$).

Step 2: The hazard curve (Fig. 2b) was calculated by using a simplified seismotectonic model (Baker 2011a; EZ-FRISK 2012) and fitted by means of a linear function in the log domain using appropriate acceleration intervals [0.20 g 2.00 g]. The obtained parameters of the hazard function correspond to k=2.80 and $k_0 = 4.3 \cdot 10^{-5}$. There is no need to use linear hazard function as discussed in the following.

Step 3: The peak ground acceleration a_{gC} was assessed iteratively by solving Equation 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 $P\left(C \middle| A_g = a_g\right)$ and by using the closed form solution of the risk equation (Equation 3). The standard deviation of the logarithm of the peak ground accelerations causing collapse was assumed to be equal to β_C =0.60 (Dolšek 2012; Lazar & Dolšek 2014). For this particular example a_{gC} was estimated to amount to 1.56 g. The same value was obtained when a_{gC} was estimated on the basis of an approximate closed-form solution (Equation 4), taking into account a linear hazard function (see Fig. 2b).

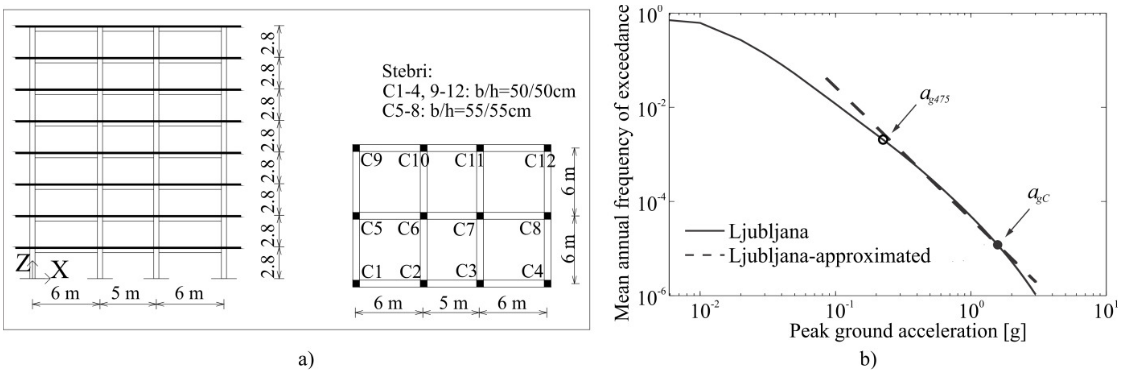

Figure 2. (a) Elevation and plan views of the investigated 8-storey building, and (b) the hazard function for Ljubljana (Slovenia) and the approximated linear hazard function in the log domain.

Step 4: A reduction factor equal to $r_C = 15.5$ was assumed. Such a value was 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 $r_C = r_\mu \cdot r_s$ vary from around 13 to 16 (Žižmond et al. 2014).

The demand-to-capacity spectral acceleration ratio $r_{dc} = 0.90$ was calculated by using the median spectrum corresponding to the ground motions that were selected on the basis of the conditional mean spectrum, which was determined according to the conditional spectrum approach (Baker 2011b). The earthquake scenario was obtained from the deaggregation of the seismic hazard for a_{gC}. It is important to mention that r_{dc} was assessed using the median spectral intensity from the spectrum of the selected ground motions. This is the only possible approach since the risk-targeted median capacity spectrum is not known during the design process. However such an approach is correct, since the shape of the median spectrum corresponding to the hazard-consistent ground motions is the same as that of the median risk-targeted capacity spectrum when scaled to the same value of the peak ground acceleration (Žižmond & Dolšek 2015). Such an outcome can also be demonstrated by a simple derivation if it is assumed that the median value of a sample of spectral accelerations is calculated by the maximum likelihood method. However, the 16th and 84th percentile spectra are different.

Step 5: The risk-targeted peak ground acceleration for the design a_{gD} =0.114 g was obtained by reducing a_{gC} =1.56 g by means of the reduction factors r_C =15.5 and r_{dc} =0.9 (Equation 8).

3.3 **Design of the structure**

The structure was designed using the design spectrum which was defined by the design peak ground acceleration a_{gD} =0.114 g and the shape of the Eurocode 8 elastic spectrum. The ratio between the design base shear and the weight corresponded to 7.6% and 7.3% for the X and Y directions, respectively. However, calculations of the amount and detailing of the reinforcement were based on Eurocode 8 (CEN 2004) by taking into account the minimum requirement corresponding to ductility class medium (DCM). The damage limitation requirement was also taken into account.

3.4 **Assessment of the structure**

3.4.1 *Pushover analysis*

The nonlinear structural model of the building was developed with consideration of the Eurocode 8-3 (CEN 2005) requirements. The model consisted of a linear elastic beam and column and two inelastic rotational hinges (defined by a moment-rotation relationship) which simulate the nonlinear behaviour of the structure. More details about the simplified nonlinear models of frame buildings can be found elsewhere (Dolsek 2010). All the analyses were performed by means of the PBEE toolbox (Dolsek 2010) which provide user-friendly interface with OpenSees (McKenna & Fenves 2010).

Conventional pushover analyses were performed. The results are presented for the X direction only. The invariant force vectors corresponded to the product of the storey masses and the fundamental vibration modes. The pushover curves and the corresponding idealized pushover curve are shown in Figure 3a. The yield strength F_y (which corresponds to the maximum strength) of the idealized system was 3326 kN. The yield displacement D_y and collapse displacement D_C were equal to 8.8 cm and 64.6 cm, respectively. Note that the collapse (C) limit state was assumed to occur at a base shear corresponding to 25% of the maximum strength, if measured in the post-capping range of the pushover curve (the end of the idealized base shear – top displacement relationship).

3.4.2 *Conventional reduction factor r_C*

In order to prove that the structure is safe against collapse, the most uncertain assumption in the design, has to be checked. It was therefore decided to estimate the value of the actual reduction factor r_C (Equation 10), which depends on the actual system ductility μ_C, the design value of the base shear corresponding to the first vibration period $F_{D,1}$ and the inelastic deformation ratio C_1. All these factors can be estimated by pushover based methods once the structure has been designed.

The available system ductility associated with collapse of the structure μ_C was obtained as the ratio between the displacement corresponding to the collapse limit state and that causing yielding of the idealized system $\mu_C = D_C / D_y = 64.6/8.8 = 7.31$.

The design value of the base shear corresponding to the first vibration period $F_{D,1} = 1704kN$ was calculated as product of the effective mass corresponding to the first mode shape ($m_{eff,1} = 1920t$) and the spectral acceleration ($S_a(T_1) = a_{gD} \cdot S \cdot 2.5 \cdot T_C / T_1 = 0.114 \cdot 1 \cdot 2.5 \cdot 0.4/1.26 = 0.090g$) which corresponded to first vibration period.

The inelastic deformation ratio C_1 is defined as the ratio between the collapse displacement of the equivalent nonlinear SDOF model (D^*_{nC}) and the displacement of the equivalent linear elastic SDOF model when subjected to S_{aC} (D^*_{eC}). In order to estimate C_1, incremental dynamic analysis has to be performed on a nonlinear SDOF system for the hazard-consistent set of ground motions used for the calculation of r_{dc}. S_{aC} is then calculated as the median value of the sample of collapse intensities which cause a collapse displacement in the case of the nonlinear SDOF model (D^*_{nC}). The displacement of the elastic model is finally obtained by using the formula which connects together the spectral acceleration and the displacement of the elastic system ($S_{aC} = \omega^2 \cdot D^*_{eC}$) or by performing a dynamic analysis for the ground motion which is scaled to the value of spectral the acceleration S_{aC} .

The SDOF model was defined according to the N2 method (Fajfar 2000). For this particular SDOF model and the set of ground motions described in Section 3.2, the observed displacement of the elastic model (D^*_{eNC} = 54.5 cm) was slightly greater than the inelastic displacement (D^*_{nNC} = 50.6 cm) (Fig. 3b). Consequently the inelastic deformation ratio C_1 was smaller than 1 (C_1 =0.93). This phenomenon is a consequence of the use of the conditional spectrum approach for the selection of hazard-consistent ground motions. Note that the collapse displacement of the nonlinear SDOF (D^*_{nC}) model (Fig. 3b) is estimated by dividing the near-collapse displacement of the structure from the pushover curve (D_C =64.6 cm) by the transformation factor associated with the first vibration model (Γ_1 =1.27)).

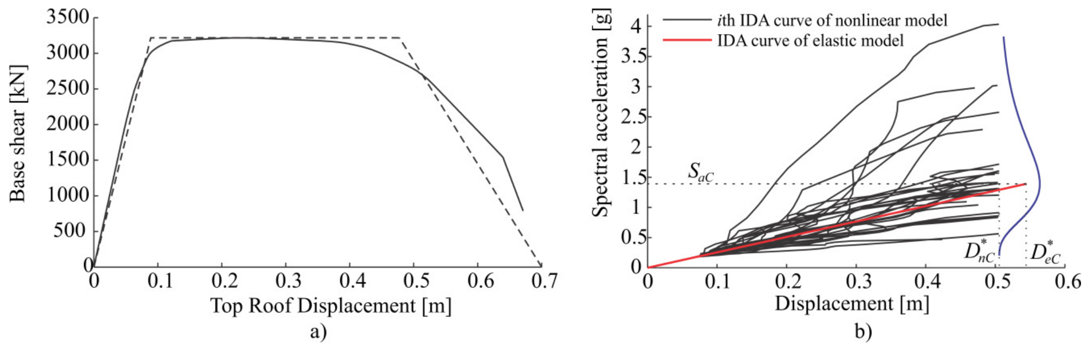

Figure 3: (a) Pushover curve and idealized base shear – top displacement relationship for the X direction of the investigated 8-storey building, and (b) IDA curves for the nonlinear and elastic SDOF models

The reduction factor can then be calculated according to Equation 10:

$$r_C = r_\mu \cdot r_s = \frac{\mu_C}{C_1} \cdot \frac{F_y}{F_{d,1}} = \frac{7.31}{0.93} \cdot \frac{3326}{1704} = 7.88 \cdot 1.95 = 15.39$$

It has to be noted that the estimated value of the reduction factor using pushover analysis and the response of the SDOF model is only slightly smaller than the value assumed for the definition of the risk-targeted design spectrum for force-based design (in the design phase r_C was assumed to be equal to 15.5). It can therefore be concluded that the collapse risk of the investigated structure is very similar to the target collapse risk. However, such a conclusion is based on the results of a simplified seismic performance assessment. For a more accurate approach, the so-called 3R method (Dolšek & Brozovič 2015) could be used. The method requires only a few nonlinear dynamic analyses for risk-based decision-making. However, it is necessary to adequately select so-called characteristic ground motions, which can be done by using CGMapp (www.smartengineering.si).

4 CONCLUSIONS

The application of risk-targeted force-based design was demonstrated for an 8-storey reinforced concrete frame building. The proposed design is a simple extension of the current state of practice, but it allows design for an explicitly defined target collapse risk. It is shown that the proposed definition of the design peak ground acceleration in closed form clearly accounts for 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. Thus the designer can obtain a better insight into the importance of the various parameters which influence the design.

A simple approach for checking the seismic performance of structures by estimating the value of the reduction factor was also demonstrated on the basis of conventional pushover-based methods. It was shown that the assumed value of the reduction factor was only slightly overestimated, so that the actual seismic collapse risk is very close to the target collapse risk. This example also shows that, although force-based design is approximate, it can provide sufficiently accurate results. In the more general case, performance checks can be estimated based on nonlinear dynamic analysis (e.g. the 3R method, Dolšek

& Brozovič 2015). Alternatively, a risk-targeted force-based design procedure can be used only to define the initial structural configuration, which is required within a risk-based design procedure (Lazar Sinković et al. 2015, www.smartengineering.si).

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