THE RELATIONSHIP BETWEEN EUROCODE’S BEHAVIOUR FACTOR AND THE RISK-TARGETED SAFETY FACTOR

Jure ŽIŽMOND¹, Nuša LAZAR SINKOVIĆ², Matjaž DOLŠEK³

ABSTRACT

In this paper the recently proposed risk-targeted safety factor is presented. It is based on target probability of exceedance of a designated limit state and it can be used for direct risk-targeted check using nonlinear analysis. Alternatively, it can be used for the definition of a risk-targeted behaviour factor in order to design a building to a target risk with the conventional force-based approach. In the first part of the paper, the theoretical background of the risk-targeted safety factor and behaviour factor is presented. It is shown how the risk-targeted safety factor is related to the conventional behaviour factor. In the second part, the use of the risk-targeted safety factor is demonstrated by performing a safety check of two reinforced concrete frame buildings. The buildings were designed according to Eurocode 8 provisions by taking into account the conventional behaviour factor. In addition, it is also shown how the results of pushover analysis can be used for the approximate evaluation of the risk-targeted behaviour factor. It was found that the 4-storey building, which was designed for a behaviour factor of 5, was safe against collapse with some additional safety. It is also shown that the risk-targeted behaviour factor would amount to approximately 7.5 in order to achieve a collapse risk of \(10^{-4}\) per year. For the 6-storey building, on the other hand, the collapse safety was observed to be approximately equal to the target collapse risk. This fact was also reflected in the estimated risk-targeted behaviour factor, which was approximately equal to the conventional factor used for design of the 6-storey building.

Keywords: Eurocode 8; Risk-targeted design; Risk-targeted safety factor; Target risk; Risk-targeted behaviour factor

1. INTRODUCTION

The seismic demand in conventional earthquake-resistant design (often called force-based design) is obtained on the basis of linear elastic analysis by considering the elastic response spectrum, which is reduced with the behaviour factor \((q)\) (Fardis et al. 2015). For buildings of ordinary importance, the elastic response spectrum corresponds to a return period of 475 years. However, the acceptable return period of collapse is significantly greater, even to around 100 times greater, as discussed by some authors (e.g. Douglas et al., 2013, Fajfar et al. 2014). As a consequence, the acceptable performance of a structure corresponding to a seismic action with a return period of 475 years has to be defined with consideration of an adequate safety factor. In the case of force-based design this issue is approximately solved by so-called design factors (Žižmond and Dolšek, 2016).

However, the issue can be solved also in the case of performance-based design which involves nonlinear analysis (e.g. pushover-based methods, nonlinear time-history analysis). Performance-based design is to be used in the new generation of the Eurocode 8 standard (CEN, 2017), while a simplified reliability verification is foreseen in Annex F of final draft of Eurocode 8. The working material for Annex F, which included a procedure for the estimation of the risk-targeted safety factor and the risk-targeted behaviour factor, was prepared by Dolšek et al. (2017a).

A procedure for the calculation of the risk-targeted behaviour factor was originally proposed by Žižmond and Dolšek (2014). The manuscript was rejected in Earthquake Engineering and Structural Dynamics in 2016. A slightly different paper, which did not include the general solution of the problem, was published at the

¹Assistant, University of Ljubljana, Ljubljana, Slovenia, jure.zizmond@fgg.uni-lj.si
²Assistant, University of Ljubljana, Ljubljana, Slovenia, nusa.lazar-sinkovic@fgg.uni-lj.si
³Full Professor, University of Ljubljana, Ljubljana, Slovenia, matjaz.dolsek@fgg.uni-lj.si
The formulation and the solution of the risk-targeted behaviour factor from 2014 was for the development of Eurocode 8 proposed in June 2017 by Dolšek et al. (2017a). The risk-targeted behaviour factor is defined as the conventional behaviour factor, but by a clear definition of the ductility reduction factor, which is related to the near collapse limit state rather than to the limit-state of significant damage. The relation to target risk is achieved by incorporating a risk-targeted correction factor. In 2014, the correction factor was termed as $C_p$, while later on (e.g. Dolšek et al. 2017b) the correction factor was termed as risk-targeted safety factor $\gamma_{im} = 1/C_p$. The later terminology is used in Annex F of final draft of new Eurocode 8 (CEN, 2017). Recently a simple model for the risk-targeted safety factor was proposed (Dolšek et al., 2017b). The formulation of risk-targeted behaviour factor (Zižmond and Dolšek, 2015, 2017; Dolšek et al. 2017b) is useful for the calibration of the behaviour factors for conventional structural systems and new innovative structural systems, for which the values of the behaviour factors still need to be developed. However, it should be noted that in addition to the risk-targeted safety factor, the effect of epistemic uncertainty on the resistance model should be account by an adequate value of partial factors (Franchin, 2018) which are applied for estimating the resistance of the structural components. This approach then guarantee that the performance objectives are met for a great majority of realized constructions and not only for 50% them.

In this paper, the theoretical background of the risk-targeted safety and behaviour factor is summarized. Then a risk-based decision model for the verification of collapse safety of structures (Dolšek et al., 2017b) is briefly presented. In the last part of the paper it is shown how the risk-based decision model, which involves a model of risk-targeted safety factor, can be applied if the performance of structure is checked by a pushover-based method. At the end it is also shown how it is possible to approximately estimate the risk-targeted behaviour factor on the basis of pushover analysis of a building designed according to Eurocode 8.

2. RISK-TARGETED SAFETY FACTOR

Methods of different accuracy and complexity have been proposed for seismic risk estimation. However, in the process of risk estimation, the analyst has to estimate the seismic intensity causing a designated limit state, which, in general, requires simulations based on non-linear time-history analysis, e.g. incremental dynamic analysis (Vamvatsikos and Cornell, 2002) or multiple stripe analysis (Jalayer and Cornell, 2009). Pushover-based methods can also be used and they have some advantages to rigorous simulation methods since the pushover analysis is an intuitive analysis method for the estimation of the capacity of the structure (Dolšek, 2016). An alternative to rigorous simulation methods is single-stripe analysis, e.g. a variant of 3R method (Dolšek and Brozovič, 2016). The aim of the 3R method is to check the capacity of the structure in terms of the risk-targeted seismic intensity causing a designated limit state. An analyst has to perform only single-stripe analysis (i.e. intensity-based assessment), while the decision-making about the acceptable performance of the structure is based on the proportion of ground motions which cause exceedance of a designated limit state and not on the observed median values of engineering demand parameters, which is done in conventional performance-based engineering.

Since engineering practitioners are not familiar with the risk estimation methods, a simple decision model for the verification of collapse safety was recently proposed by Dolšek et al. (2017b). The decision model involves the conventional concept of the safety check, which is based on a comparison between seismic demand and capacity, which are, however, expressed with seismic intensity. An engineering practitioner has to prove that the demand in terms of spectral acceleration at the fundamental vibration period, $S_{e,SD}$, is lower than the ratio between the spectral acceleration causing near collapse limit state, $S_{e,NC}$ (i.e. the median value of spectral acceleration causing near collapse limit state, which in the design has to determine with consideration of the effects of epistemic uncertainty on the resistance (capacity) side), and the risk-targeted safety factor $\gamma_{im}$:

$$S_{e,SD} \leq \frac{S_{e,NC}}{\gamma_{im}}$$

Seismic demand in equation 1 corresponds to the design seismic action associated with the significant damage (SD) limit state, $S_{e,SD}$, which for structures of ordinary importance corresponds to a return period of 475 years.
The median value of near-collapse (NC) limit-state intensities $S_{e,NC}$, which should be estimated by an adequate level of confidence, can be estimated by different method of analysis. In the simplest case pushover-based methods can be used, e.g. the N2 method (Fajfar, 2000), while the nonlinear behaviour of the components should account for an adequate confidence level (i.e. partial factors (Franchin 2018)). However, for engineering practitioners the estimation of the so-called risk-targeted safety factor $\gamma_{im}$ (Dolšek et al., 2017) is also challenging task:

$$\gamma_{im} = \frac{S_{e,t,NC}}{S_{e,SD}}$$  \hspace{1cm} (2)

where $S_{e,t,NC}$ is the risk (reliability) targeted limit-state spectral acceleration corresponding to the NC limit state, which represents the median value of risk-targeted intensities. Note that the seismic intensity corresponding to the NC limit state was intentionally selected instead of seismic intensity corresponding to the collapse (C), since the estimation of collapse for complex structures is very uncertain and often associated with numerical non-convergence and thus practically never estimated by engineering practitioners. However, the target reliability of a structure is usually associated with collapse, which means that there are several models for the estimation of probability of collapse ($P_{t,C}$). The $P_{t,C}$ is, in addition to the seismic hazard function $H(S_e)$ and the standard deviation of seismic collapse intensities ($\beta_{Se,C}$), an essential input parameter for the calculation of the risk (reliability) targeted limit-state spectral acceleration corresponding to collapse ($S_{e,t,C}$) which can be obtained iteratively by solving the so-called risk equation (Cornell et al., 2002):

$$P_{t,C} \approx \lambda_{t,C} = \int_0^\infty P(C \mid S = S_e ; S_{e,t,C}, \beta_{Se,C}) \cdot \frac{dH(S_e)}{dS_e} \cdot dS_e$$  \hspace{1cm} (3)

where $P(C \mid S = S_e ; S_{e,t,C}, \beta_{Se,C})$ is the so-called risk targeted collapse fragility function, which is usually defined by the lognormal cumulative distribution function, i.e. by median $S_{e,t,C}$ and the corresponding standard deviation $\beta_{Se,C}$. $S_{e,t,NC}$ can then be estimated from $S_{e,t,C}$ with consideration of the model of the so-called limit state reduction factor $\gamma_{ls}$ (Dolšek et al., 2017b, see Figure 1a):

$$S_{e,t,NC} = \frac{S_{e,t,C}}{\gamma_{ls}}$$  \hspace{1cm} (4)

The risk-targeted spectral acceleration causing the NC limit state can then be used in Equation 2 in order to determine $\gamma_{im}$. However, such calculation of $\gamma_{im}$ is currently too complicated for engineering practitioners. In addition, the seismic hazard function is very uncertain for low occurrence rates. Thus it makes sense to develop simple models for $\gamma_{im}$ which can be prescribed for different regions and selected levels of target risk. Such a model was recently developed by Dolšek et al. (2017). It corresponds to target collapse risk of $10^{-4}$ and depends on the period of the structure, as presented in Figure 1c. The model of $\gamma_{im}$ presented in Figure 1c is based on a period dependent model for limit state reduction factor $\gamma_{ls}$ (Figure 1a) and $\beta_{Se,C}$ (Figure 1b). The simplified formula for the proposed model of $\gamma_{im}$ for the region of Slovenia is as follows (Dolšek et al., 2017b):

$$\gamma_{im} = \begin{cases} 5 & T \geq 3T_c \\ 2.5 \left( \frac{T}{3T_c} + 1 \right) & T < 3T_c \end{cases}$$  \hspace{1cm} (5)

$\gamma_{im}$ is equal to 5 for buildings with long vibration periods, e.g. tall buildings, and between 2.5 and 5 for buildings with shorter vibration periods (see Figure 1c). Values of $\gamma_{im}$ appear to be very large, but their accuracy has been tested on buildings designed according to Eurocode 8 provisions, which satisfied the condition given by Equation 1 even with some additional safety. However, it is important to note that no additional safety factors should be used when $S_{e,NC}$ is estimated, but an adequate model for the confidence level should be taken into account.
account.

For regions with very different seismicity, values of $\gamma_{in}$ can be significantly different. The selected target risk also influences the value of $\gamma_{in}$. For example, by increasing the target risk by a factor of 2, i.e. to $2 \times 10^{-4}$ as used in some building codes (Luco et al. 2007), $\gamma_{in}$ decreases from 5 to 4 for buildings with long vibration periods. Even though the authors have chosen a target value of the collapse probability of $10^{-4}$, which is 10 times greater than the value given by both experts and non-experts in the field of structural engineering in a survey about tolerable probabilities of collapse (Fajfar et al., 2014), the values of $\gamma_{in}$ are considerably larger than 1. We can therefore assume that $S_{e,NC}$ has to be several times larger than $S_{e,SD}$ for buildings to be safe against collapse.

![Figure 1. The vibration period dependent models for a) the limit-state reduction factor $\gamma_{ls} = S_{e,C}/S_{e,NC}$, b) the standard deviation $\beta_{S,e,C}$ with indicated values for the investigated buildings and c) the risk-targeted safety factor $\gamma_{in}$ (Dolšek et al., 2017).](image)

### 3. THE CONVENTIONAL BEHAVIOUR FACTOR AND THE RISK-TARGETED BEHAVIOUR FACTOR

The conventional definition of the behaviour factor is not based on a target risk. According to the new draft of the Eurocode 8 (CEN, 2017) the reduced response spectrum $S_{hr}(T)$, which is used for the estimation of design seismic forces, is obtained by dividing the horizontal elastic response spectrum $S_e(T)$ with the reduction factor $R_q$:

$$S_{hr}(T) = \frac{S_e(T)}{R_q(T)}$$

(6)

The horizontal elastic response spectrum $S_e(T)$ is defined by two values of spectral acceleration ($S_\alpha$, $S_\beta$) and prescribed spectral shape functions. For ordinary buildings (consequence class 2) and SD limit state, the two intensities ($S_\alpha$, $S_\beta$) correspond to return period $T_{ref} = 475$ years. By additionally taking into account the fact that the reduction factor $R_q$ is equal to the behaviour factor $q$ for periods greater than $T_B$, Equation 6 can be rewritten as follows:

$$S_{hr}(T) = \frac{S_{e,SD}(T)}{q} \quad T > T_B$$

(7)

where $S_{e,SD}(T)$ is the elastic response spectrum used for the verification of the SD limit state and $q$ is the conventional behaviour factor defined in the final draft of new Eurocode 8 (CEN, 2017) as:

$$q = q_R \cdot q_S \cdot q_{SD}$$

(8)
where \( q_R \) is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures, \( q_S \) is the behaviour factor component accounting for overstrength due to all other sources and \( q_{SD} \) is the behaviour factor component accounting for the deformation capacity and energy dissipation capacity associated to SD limit state.

The definition of the reduced spectrum in Equation 7 is based on the conventional deterministic approach (e.g. Fardis et al. 2015). Žižmond and Dolšek realized in the beginning of 2014 that such a definition has several shortcomings. The main problems of such a definition are that the behaviour factor component accounting for the deformation capacity and energy dissipation capacity was not associated with a clearly defined damage of the structure and that there was no relation to a target reliability associated with the fundamental performance objective of the code. In the conventional definition of the behaviour factor it is not clear whether the SD limit state is related to the protection of human lives or to the limitation of damage for rare seismic events. Furthermore, the current version of Eurocode 8 does not define what is an appropriate reliability for performance requirements although it should be clear that the acceptable return period for near collapse limit state or for collapse of a structure (or loss of life) is much greater than, for example, 475 years, which is the characterized return period associated with the consequence class 2 and the SD limit state. Thus the force-based design approach requires additional design factors (Žižmond and Dolšek, 2016) which approximately assure that the performance objective, such as the protection of human lives, is met with appropriate reliability.

In order to overcome shortcomings of the conventional definition of the behaviour factor, Žižmond and Dolšek proposed risk-targeted formulation of the behaviour factor. The procedure for the estimation of risk-targeted behavior factor was developed by the authors about 5 years ago, but it is still apparently very difficult to convince supervisors that the risk-targeted formulation of the behavior factor is more physics-based than the conventional formulation of the behavior factor. The debate is not yet over since many researchers still reject the fact that the design procedure has to accounts also for the effect of epistemic uncertainty, especially through the model of resistance of structural component (Franchin 2018).

Note that the derivation which follows was presented at international workshop on Eurocode 8 & Safety which took place in Ljubljana 26th May 2017, and it does not account for the effect of epistemic uncertainty, which should be accounted in the model of resistance of the structural component.

In the first step, the performance requirement is defined by the target probability of collapse \( P_{t,C} \). Starting from this requirement, the risk-targeted spectral acceleration causing the NC limit state \( S_{e,t,NC} \) can be calculated as defined in the previous Section. In the next step, it has to be realized that \( S_{e,t,NC} \) has to be reduced by reduction factor \( R_{NC} \) in order to obtain the risk-targeted spectral acceleration for force-based design \( S_{hr,t} \):

\[
S_{hr,t}(T_i) = \frac{S_{e,t,NC}(T_i)}{R_{NC}}
\]

Due to \( S_{e,t,NC} \) the reduction factor \( R_{NC} \) has to be related to the NC limit state. It can be shown that the reduction factor \( R_{NC} \) can be decomposed in the same manner as the behaviour factor \( q \) (Žižmond and Dolšek 2015, Žižmond, 2016):

\[
R_{NC} = q_h \cdot q_s \cdot q_{NC}
\]

where the first two components are the same as those defined to Equation 8, while the \( q_{NC} \) is the component accounting for the deformation capacity and energy dissipation capacity associated to NC limit state.

It can be seen from Equation 10 that factor \( R_{NC} \) can be explained also as the multiplication factor for \( S_{hr,t} \) in order to obtain the risk-targeted spectral acceleration causing the NC limit state \( S_{e,t,NC} \). However, the value of the \( R_{NC} \) factor depends on the design factors of the code and the objectives of design. For example, for reinforced concrete frames with long vibration periods it was observed that the \( R_{NC} \) factor is greater than 10
(or even 15) (Žižmond and Dolšek, 2016) if the frames were designed according to Eurocode 8. The large values are the consequence of material safety factors, minimum requirements, selection of reinforcement patterns and quite large deformation capacity and cumulative energy dissipation capacity associated to the near collapse limit state (Žižmond and Dolšek, 2016).

By the analogy to Eurocode 8 (Equation 7), the risk-targeted spectral acceleration for force-based $S_{hr,t}(T)$ can be defined by reducing the elastic response spectrum $S_{e,SD}(T)$ with a so-called risk-targeted behaviour factor $q_t$:

$$S_{hr,t}(T) = \frac{S_{e,SD}(T)}{q_t}$$

(11)

The risk-targeted behaviour factor $q_t$ can then be expressed from Equation 11 as follows:

$$q_t = \frac{S_{e,SD}(T)}{S_{hr,t}(T)}$$

(12)

By substituting $S_{hr,t}$ from Equation 9 into Equation 12 and by considering Equation 10 and Equation 2, the risk-targeted behaviour factor can be expressed as

$$q_t = \frac{q_R \cdot q_S \cdot q_{NC}}{\gamma_{im}}$$

(13)

From Equation 13 it can be realized that the risk-targeted behaviour factor does not depend just on the behaviour factor components $q_R$, $q_S$ and $q_{NC}$, but it is also affected by the risk-targeted safety factor $\gamma_{im}$, which depends on the return period, the target collapse risk $P_{t,C}$, the seismic hazard function and the standard deviation of the logarithm of the collapse intensities $\beta_{Se,C}$.

4. EXAMPLE

The collapse safety of a 4-storey and a 6-storey frame building (Figure 2) was verified according to Equation 1, by using the model for $\gamma_{im}$ defined with Equation 5. Additionally, the risk-targeted behaviour factor $q_t$ was estimated with Equation 13 and compared to the conventional behaviour factor $q$.

![Figure 2. Plan view and elevation of the a) 4-storey and b) 6-storey building (Dolšek et al., 2017)](image)

Both buildings were assumed to be located in Ljubljana, Slovenia, on soil type B. The 6-storey building was designed according to Eurocode 8 provisions (CEN, 2004). On the other hand, the 4-storey building was designed using Eurocode 8 prestandard (CEN, 1994). The design peak ground acceleration at the building’s site is 0.30 g. The fundamental vibration periods in the X direction amounted to 0.8 s and 1.0 s, respectively, for the 4- and 6-storey building. The 4-storey building was designed for ductility class high, whereas ductility...
class medium was taken into account for the 6-storey building. The behaviour factor $q$ used for design was equal to 5 and 3.9, respectively, for the 4- and 6-storey building. The total mass $m$, the mass of the equivalent SDOF system $m^*$, and the design base shear corresponding to the first vibration mode $F_{D,1}$ are shown in Table 1.

The structural model used for nonlinear analyses in general follows the Eurocode 8 requirements for the modelling of structures as discussed in (Dolšek, 2010). The beam and column flexural behaviour was therefore modelled by one-component lumped plasticity elements, composed of an elastic beam and two inelastic rotational hinges (defined by a moment-rotation relationship). The mean values of the compressive strength of concrete and the yield strength of the reinforcement were used for the calculation of the strength of structural elements. The ultimate rotations for the primary seismic elements were determined according to Eurocode 8-3 (CEN, 2005) by estimating median values, i.e. by omitting additional safety factors. For this reason, the safety factor $\gamma_E$ was set to 1.0 and not to 1.5, as foreseen in Eurocode 8-3 (CEN, 2005). All analyses were carried out with OpenSees (2007) in conjunction with the PBEE toolbox (Dolšek, 2010). The horizontal forces for pushover analysis were determined based on the first period of vibration, which provides the most likely plastic mechanism observed from dynamic analysis. This statement is based on the results of IDA analysis. In the case of 4-storey building it was observed that almost all failure modes caused by ground motions were similar to that obtained from pushover analysis. The most likely failure mode in the case of 6-storey building was also similar to the failure mode from the pushover analysis. However, several failure modes caused by ground motions were also different to that observed in pushover analysis.

The pushover curves and the idealized force-displacement relationship used to determine equivalent single degree of freedom models are shown in Figure 3a, whereas the yield strength $F_Y$ and displacement $D_Y$ together with the near-collapse displacement $D_{NC}$ are shown in Table 1. For brevity, only the results corresponding to the X direction of loading are presented. The highlighted points on Figure 3 indicate the design seismic action associated with the significant damage limit state $S_{e,SD}$ and the near-collapse (NC) limit state. The global near-collapse limit state was considered to occur when the near-collapse limit state was exceeded in the first column of the frame buildings. The latter was defined at a 20 % drop of strength. The spectral acceleration causing the near-collapse limit state $S_{e,NC}$ was estimated with the N2 method (Fajfar, 2000) and with incremental dynamic analysis of the SDOF system (Dolšek, 2012). In such a way the bias of the N2 method, which is caused by prescribed model for $R-\mu-T$ relationship, was checked. Note that the model for the $R-\mu-T$ relationship assumed in the N2 method (Fajfar, 2000) was developed based on sets of ground motions which provided biased seismic response since the effect of the conditional spectrum (Baker, 2011) was neglected.

The equivalent SDOF models were defined based on the results of pushover analysis of the MDOF models. The Rayleigh damping model was considered in the nonlinear time-history analysis of the equivalent SDOF. The coefficients of Rayleigh model were calculated based of 5% damping and first two fundamental periods of structure in analysed direction. The nonlinear behaviour was modelled with the peak-oriented hysteretic model, i.e. uniaxial »Hysteretic« material implemented in OpenSees (2007). For each investigated building a set of 30 ground motions was selected based on the conditional spectrum approach (Baker, 2011). The conditional spectrum was estimated based on the seismotectonic model used for the calculation of the official probabilistic seismic hazard maps for the region of Slovenia (Lapajne et. al. 2003). The conditional period corresponded to the first vibration mode of the model of the structure and a return period of 2475 years. The selected ground motions corresponded to events with magnitudes between 4.5 and 7, and source-to-site distances between 5 and 50 km. The largest considered scale factor was 4.

Table 1: The period of vibration of the equivalent model $T^*$, the total mass $m$ of the building, the mass of the equivalent SDOF system $m^*$, the transformation factor $\Gamma$, the design base shear corresponding to first vibration mode $F_{D,1}$, the yield strength $F_Y$, the yield displacement $D_Y$ and the near-collapse displacement $D_{NC}$.

<table>
<thead>
<tr>
<th>Model</th>
<th>$T^*$ (s)</th>
<th>$m$ (t)</th>
<th>$m^*$ (t)</th>
<th>$\Gamma$</th>
<th>$F_{D,1}$ (kN)</th>
<th>$F_Y$ (kN)</th>
<th>$D_Y$ (cm)</th>
<th>$D_{NC}$ (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-storey</td>
<td>0.84</td>
<td>342</td>
<td>245</td>
<td>1.25</td>
<td>268</td>
<td>1129</td>
<td>8.3</td>
<td>56.3</td>
</tr>
<tr>
<td>6-storey</td>
<td>1.02</td>
<td>5193</td>
<td>3628</td>
<td>1.26</td>
<td>4127</td>
<td>9852</td>
<td>7.2</td>
<td>47.7</td>
</tr>
</tbody>
</table>
Figure 3. a) Pushover curves with idealized force-displacement relationships and points indicating the design seismic action associated with SD and NC limit states and b) the incremental N2 curves and the median IDA curves for both buildings.

The collapse safety of the investigated buildings was then checked by the decision model proposed in Equation 1. Additionally, the collapse safety was checked with the direct decision model, which is also incorporated in Annex F of the new draft of Eurocode 8 (CEN, 2017):

$$ P_C \leq P_{t,C} $$

(14)

where $P_{t,C}$ is the target probability of collapse and $P_C$ is probability of collapse estimated with Equation 3 by taking into account the seismic hazard curve at the building’s location, standard deviation $\beta_{Sa,C}$ according to Figure 1b and the median acceleration causing collapse $S_{e,C}$, which was determined by multiplying $S_{e,NC}$ with the limit-state reduction factor $\gamma_{ls}$ (Figure 1a). It can be seen from Figure 3b that $S_{e,NC}$ estimated with SDOF IDA is slightly larger than $S_{e,NC}$ estimated with the incremental N2 method, i.e. by a factor of 1.06 ($S_{e,NC} = 2.71$ g) for the 4-storey building and 1.07 ($S_{e,NC} = 1.57$ g) for the 6-storey building.

The risk-targeted safety factors $\gamma_{lm}$, which are needed for collapse safety check using Equation 1, were obtained from the proposed model (Equation 5 (Dolšek et al., 2017)). The outcome of the collapse safety checks according to Equation 1 and 5 are presented in Table 2. Based on both safety checks it can be concluded that design according to Eurocode 8 provision provided an adequate collapse safety for the 4-storey building, which is even slightly overdesigned. For the 6-storey building, however, the collapse safety check according to Equation 14, if the assessment was based on the N2 method, was satisfied even in the case of the 6-storey building. It should be noted that the proposed decision model offers a considerable level of safety, since the selected model for $\gamma_{lm}$ is slightly conservative in most cases (Dolšek et al., 2017b). The actual values of $P_{t,C}$ taken into account for the observed buildings are therefore likely to be slightly smaller than $10^{-4}$ per year. However, if $S_{e,NC}$ obtained with the SDOF IDA is taken into account, the risk check according to both Equation 1 and Equation 14 would be satisfied, since the results of SDOF IDA are slightly less conservative.

<table>
<thead>
<tr>
<th>Model</th>
<th>$S_{e,SD}$ (g)</th>
<th>$S_{e,NC}$ (g)</th>
<th>$S_{e,NC}/\gamma_{lm}$ (g)</th>
<th>$\gamma_{lm}$</th>
<th>$\gamma_{lm}/\gamma_{lm}$</th>
<th>$P_C$ ($10^{-4}$)</th>
<th>$P_{t,C}$ ($10^{-4}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-storey</td>
<td>0.45</td>
<td>2.56</td>
<td>2.71</td>
<td>3.9</td>
<td>0.66</td>
<td>0.69</td>
<td>2.94</td>
</tr>
<tr>
<td>6-storey</td>
<td>0.37</td>
<td>1.46</td>
<td>1.57</td>
<td>4.2</td>
<td>0.35</td>
<td>0.37</td>
<td>1.68</td>
</tr>
</tbody>
</table>

Table 2: The period of vibration of the equivalent model $T^*$, the intensity based collapse safety check according to Equation (1) and the direct collapse safety check according to Equation (14). Results of the N2 method and IDA are considered.
The risk-targeted behaviour factors were estimated according to Equation 13. The product of the two components of overstrength factors $q_R q_S$ was calculated as the ratio between yield strength $F_Y$ and the base shear corresponding to the first vibration mode $F_{S,1}$ (Table 3) (Žižmond and Dolšek 2015, 2017). Note that the overstrength factor for the 4-storey building was equal to 4.2, which is almost two times greater value than that for the 6-storey building (2.3).

The behaviour factor component accounting for the deformation capacity, energy dissipation capacity and the seismic response of the structure associated to the NC limit state was estimated according to two approaches. Firstly, $q_{NC}$ was assumed to be equal to the near-collapse ductility of the structure $\mu_{NC}$, which was calculated by dividing the near-collapse displacement $D_{NC}$ with the yield displacement $D_Y$ (see previous Table 1). Such an approach for the estimation of $q_{NC}$ is based on the assumption of equal displacement rule (EDR). The rule assumes that for a structure with a long vibration period the displacement of the inelastic system is equal to the displacement of the elastic system. However, it has to be noted that for the calculation of the risk-targeted behaviour factor such an assumption may be biased. Therefore, $q_{NC}$ was also calculated by dividing the near collapse ductility of structure $\mu_{NC}$ with the inelastic displacement ratio $C_1$ which was obtained on the basis of nonlinear dynamic analysis by taking into account hazard-consistent ground motions (Baker, 2011), which were selected for safety collapse check (see previous paragraph). The inelastic displacement ratio $C_1$ can be calculated as a ratio between the NC displacement and the displacement of the elastic system corresponding to the spectral acceleration causing the NC limit state ($S_{NC}$). However, it can be proved that for structures with long periods the inelastic displacement ratio $C_1$ can also be calculated as ratio between the $S_{NC}$ obtained on the basis of N2 method and SDOF IDA if both spectral acceleration are known. Using later described approach for determination the $C_1$ it was found that the inelastic displacement ratio $C_1$ amounted to 2.56/2.71 = 0.94 and 1.46/1.57 = 0.93, respectively, for 4- and 6-storey buildings. Consequently, the values of $q_{NC,IDA}$ obtained on the basis of SDOF IDA are for a factor of 1.06 and 1.07 greater than $q_{NC,EDR}$ obtained on the basis of the equal displacement rule, respectively, for the 4- and 6-storey building. Note that for these particular examples the assumption of equal displacement rule does not have large impact on the estimation of the behaviour factor component accounting for the deformation capacity, energy dissipation capacity and the seismic response of the structure associated to the NC limit state, which may not be the case for some other examples.

The risk-targeted safety factor $\gamma_m$, which is needed for the calculation of risk-targeted behaviour factor (Equation 13), was estimated by Equation 5 (Dolšek et al. (2017b)). The values are the same as those used in the case of the previous check.

Table 3: The product of two overstrength factors $q_R q_S$, the ductility of the structure $\mu_{NC}$, the behaviour factor component accounting for the deformation capacity $q_{NC}$ based on the equal displacement rule (EDR) and incremental dynamic analysis (IDA), the risk-targeted safety factor $\gamma_m$, the risk-targeted behaviour factor $q_t$ based on EDR and IDA and the conventional behaviour factor $q$.

<table>
<thead>
<tr>
<th>Model</th>
<th>$q_R q_S$</th>
<th>$\mu_{NC}$</th>
<th>$q_{NC}$</th>
<th>$\gamma_m$</th>
<th>$q_t$</th>
<th>$q$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>EDR</td>
<td>IDA</td>
<td>EDR</td>
<td>IDA</td>
</tr>
<tr>
<td>4-storey</td>
<td>4.2</td>
<td>6.8</td>
<td>6.8</td>
<td>7.2</td>
<td>3.9</td>
<td>7.3</td>
</tr>
<tr>
<td>6-storey</td>
<td>2.3</td>
<td>6.6</td>
<td>6.6</td>
<td>7.1</td>
<td>4.2</td>
<td>3.7</td>
</tr>
</tbody>
</table>

The risk-targeted behaviour factor was then calculated according to Equation 13. For the 6-storey building, the risk-targeted behaviour factor was estimated to $q_{t,EDR} = 3.7$ and $q_{t,IDA} = 4.0$, which is indeed close to $q=5.0$ as proposed in Eurocode 8. This outcome was also expected from results of the collapse check in the first part of the Section. However, the estimated values of $q_t$ for the 4-storey building ($q_{t,EDR} = 7.3$ and $q_{t,IDA} = 7.7$) (Table 3) are significantly higher than the value $q=5.0$ which was proposed by Eurocode 8 prestandard (CEN 1994) for such reinforced concrete frames and ductility class high. Such a result was expected since the probability of collapse of the building was smaller than the target probability of collapse (Table 2). Thus it makes sense that prescribed value of the behaviour factor for the design of reinforced concrete frames (DCH) was increased in the current code to 5.85 or even 6.75 (CEN, 2004). There is still small safety margin which, however, is probably needed in order to achieve an adequate confidence level, which is not discussed in this paper. In addition, if the assumed value of behaviour factor and the estimated value are significantly different, as it was...
observed in this particular example, it is recommended to redesign the structure using new value of behaviour factor and repeat the procedure of the estimation of risk-targeted behaviour factor.

5. CONCLUSIONS

In this paper it was shown how the risk-targeted safety factor $\gamma_{im}$ is incorporated in conventional earthquake-resistant design which involves design seismic action associated with low return period (e.g. 475 years). Since engineering practitioners are not familiar with probabilistic approaches it is quite useful to introduce a model of $\gamma_{im}$, which can then be used in performance-based design or for the estimation of the behaviour factor. It was shown before that for structures with long vibration period which are located in a region with relatively high seismic hazard, the value of risk-targeted safety factor should be around 5, if the target collapse risk is set to 0.5% in 50 years ($10^{-4}$ per year). However, for structures with very short vibration period (e.g. single-storey masonry buildings), the value $\gamma_{im}$ can be assumed equal to 2.5.

In the paper it was also summarized how the risk-targeted safety factor $\gamma_{im}$ is related to the behaviour factor and how the behaviour factor can be estimated on the basis of pushover analysis. However, several issues associated with the implementation of $\gamma_{im}$ to the behaviour factor were not addressed in this paper. The risk-targeted behaviour factor does not take into account only the overstrength and the ability of structures to deform in the nonlinear range but it also depends on the target collapse risk, the seismic hazard and the uncertainty in the seismic response of structures. The risk-targeted behaviour factor can thus be used for risk-based design of structures, similarly as the conventional behaviour factor in Eurocode 8. The procedure for the estimation of the risk-targeted behaviour factor was also included in the informative Annex F of the final draft of new Eurocode 8. This was not an easy task since quite a significant number of engineering practitioners and scholars believe that additional design factors in force-based design cause excessive use of resources which leads to uneconomic construction.

The risk-targeted behaviour factor was estimated for two reinforced concrete frame buildings by taking into account the proposed model for the risk-targeted safety factor. The risk-targeted behaviour factor was approximately 1.5 times larger than the conventional behaviour factor for the 4-storey building, whereas it was approximately equal for the 6-storey building. This indicates that the 4-storey building is fairly on the safe side regarding the seismic collapse safety, since it was designed to withstand a larger seismic action than the target seismic action causing the collapse of the building. The 6-storey building, however, was designed using seismic loads which approximately correspond to actions obtained based on the risk-targeted behaviour factor. These conclusions were also confirmed by the simple collapse check based on the risk-targeted safety factor and the direct collapse check. The safety conditions were satisfied with some additional safety for the 4-storey building and were approximately satisfied for the 6-story building.

6. ACKNOWLEDGMENTS

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7. REFERENCES


