16TH EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING THESSALONIKI 18 - 21 JUNE 2018

THE EVALUATION OF RISK-TARGETED SAFETY FACTOR AND BEHAVIOUR FACTOR FOR SELECTED STEEL STRUCTURES

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ABSTRACT

Steel structures can often be essential components of petrochemical plants. Their function is to support complex system of pipes. Although quite severe accidents can occur in petrochemical plants, they are almost always designed by conventional force-based design, which is not based on the target risk/resilience. Recently new philosophy for force-based design was introduced. It makes it possible to incorporate the design for a target risk by using risk-targeted safety factor in the evaluation of the behaviour factor. In this paper, the procedure is applied to a set of simple steel moment resisting frames, considering different modelling approaches, different locations and different target probabilities of failure. Special attention is given to the evaluation of the overstrength factor, which has to be calculated on the basis of the design base share which is decisive for the selection of cross-section of structural elements. The risk-targeted behaviour factors presented in the paper are meant to fulfill the near collapse limit state without applying the damage limitation requirements. It is shown that so-determined q-factors are lower than those prescribed in Eurocode 8. The results showed that with a 50% reduction of the behavior factor, the target risk can be reduced up to around four times. The advantage of the proposed approach in comparison to the conventional approach is that the risk-targeted behaviour and safety factors can be estimated from a target risk for occurrence of a designated consequences, which may be particularly important for the design of critical infrastructure such as complex petrochemical plants.

Keywords: risk-targeted behaviour factor; risk-targeted performance factor; steel frames; limit states; pushover analysis

1. INTRODUCTION

Steel structures are often used as a supporting system of essential components (e.g. piping system) of critical infrastructures such as petrochemical plants. Since they store large amount of hazardous materials, a major seismic event could cause severe accidents, which could involve all the population living in the close surrounding of the plant. Moreover, seismic events can cause damage of components of petrochemical plants, which can trigger significant loss of functionality and business interruption, also leading to permanent relocation of businesses.

Although the potential for losses is much greater in the case of petrochemical plants, the design procedure of such critical infrastructure is practically the same as that used for design for buildings of ordinary importance. In order to adequately account for potential losses in the design or assessment phase of critical infrastructures, it is important to develop a practice-oriented design procedure of petrochemical plants based on target risk and resilience.

The reference method of Eurocode 8 (CEN, 2004) for design of petrochemical plants and other type of buildings is based on linear elastic analysis. Due to inconsistency of the analysis method and objective of the code, which foresees that the structures can be damaged if subjected to the design seismic action,

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it is necessary to approximately account for the limit-state deformation capacity and the cumulative energy dissipation capacity in the prediction of design seismic action, i.e. when the structure is not yet full defined. This issue is solved by introducing the concept of reduction of seismic forces. The same concept is prescribed in Eurocode 8 by introducing the behavior factor (CEN, 2004), which is described in book by Fardis et al. (2015). The concept of reduction of seismic forces was used for decades. For example, Fischinger & Fajfar (1990) defined the reduction factor as the product of overstrenght and ductility reduction factor. A similar proposal was made by Uang (1991). Recently, an innovative procedure for the calculation of risk-targeted behaviour factor was proposed by Žižmond & Dolšek (2014). The paper was rejected in Earthquake Engineering & Structural Dynamics, but practice-oriented solution was eventually published at 16WCEE (Žižmond and Dolšek 2017). In addition, a simplified model for risk-targeted safety factor was recently introduced by Dolšek et al. (2017), where the IM-based decision model for the verification of collapse safety based on γ_{im} was proposed. Following this approach, a structure should be designed in such a way that the ratio between the risk-targeted seismic intensity causing a designated limit state and the γ_{im} is greater than the design value of seismic intensity used from seismic hazard map.

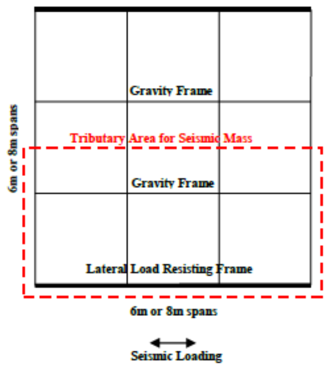
In this paper the concept of risk-targeted safety factor and the risk-targeted behaviour factor was implemented to selected steel structures in order to demonstrate its application also in the case when the design seismic action associated with significant damage state is not a decisive parameter for a structure. For simplicity, the application is made to regular moment resisting frames, while the procedure is general in terms of limit states and in terms of structures. Firstly, the investigated steel structures are described with an emphasize on the consideration of modelling uncertainties. Follows brief introduction to risk-targeted safety factor and behaviour factor according to notation used in Dolšek et al. (2017). In the second part of the paper the value of risk-targeted behaviour factor with consideration of different values of target risk is discussed based on the results of pushover analysis. The discussion also considers the impact of the consideration of the limit-state of damage limitation on the estimated value of risk-targeted behaviour factor.

2. DESIGN AND MODELLING OF INVESTIGATED STEEL STRUCTURES

2.1 Description of the investigated steel structures

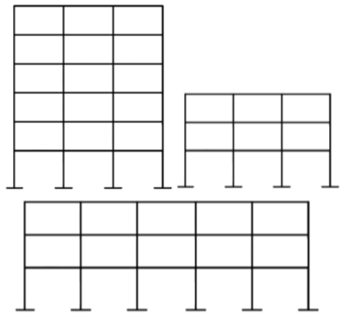
Three moment-resisting steel frames were analyzed. The frames were obtained from typical steel buildings designed by Tsitos et al. (2017) in accordance to Eurocode 3 (CEN, 2005) and Eurocode 8 (CEN, 2004). The investigated structures conform to current state-of-practice in Europe. They consist of interior gravity frames and lateral moment-resisting frames (MRF) located at the perimeter (Figure 1a). The structures were designed taking into account a reference value of peak ground acceleration equal to 0.25g, type 1 design spectrum, soil type C and behavior factor equal to 6.5.

It has to be noted that the criterion for the damage limitation (DL) limit state governed the design of the structures. The limit value of the inter-storey drift was considered equal to 0.75% of the storey height. The HEB and IPE profiles were selected respectively for the columns and beams and steel S355 was used. The geometry of frames used in this study is shown in Figure 1b. For all three frames (6S3B, 3S3B, 3S5B) the height of ground storey is 4.5 m whereas the height of other storeys is 3.5 m. The span length is 6 m for all the frames (Tsitos et al. 2017).



(a)

ID	N. of storeys	N. of spans
6S3B	6	3
3S3B	3	3
3S5B	3	5



(b)

Figure 1. (a) the plan view of the investigated steel moment resisting frames (MRFs) (Tsitos et al. 2017) and (b) the corresponding elevation view for three different configurations of the geometry of MRFs.

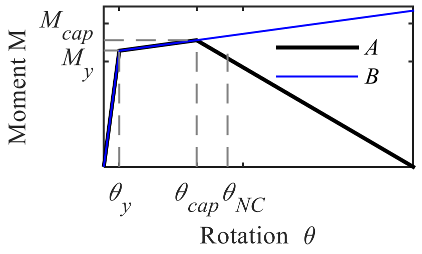
2.2 Description of mathematical modelling

Structural models used in the nonlinear analyses were consistent with the requirements of Eurocode 8. The mean material characteristics were therefore used. All the analyses were performed using OpenSees (McKenna & Fenves 2010). The columns were considered fixed at their base and all the beams were fully restrained at their ends. Diaphragm constraints were also used at each floor level. To simulate $P-\Delta$ effects, a leaning column with additional gravity loads was connected to the frame by axially rigid beams pinned at both ends. The leaning column was pinned at its base in order not to affect the behavior of the frame. Geometric imperfections were modelled by introducing off-plane midpoints with an offset of $L/1000$ in columns. In this way the buckling phenomenon is instigated and can be monitored. Distributed plasticity and lumped plasticity models were developed.

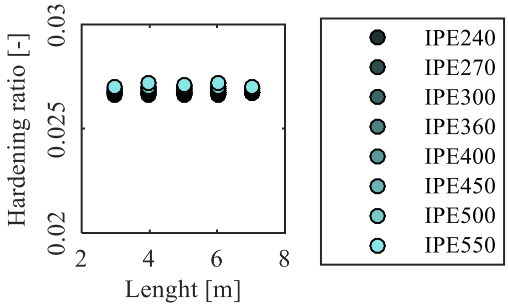
Distributed plasticity (DP) models, which are in this paper presented only for the purpose of comparison, were developed on the basis of models of eccentrically-braced frames provided by Tsitos et al. (2017). Beams and columns were modelled as ForceBeamColumn elements, each section was discretized into fibers and Steel02 material with isotropic strain hardening was used. In order to assess the element failure, a MinMax material was added in parallel for beams: the strain limit value was calibrated by Tsitos et al. (2017) and set equal to 0.021. For columns, instead, no ultimate point was defined.

The reference models in this study are the lumped plasticity models. In this case the flexural behaviour of the beams and columns was modelled by means of lumped plasticity elements, which consisted of an elastic element and two inelastic rotational hinges (defined by a moment-rotation relationship).

The inelastic hinges for beams were modeled with trilinear moment-rotation relationship (black line in Figure 2a). The yield moment M_y was obtained by the bilinearization of the moment-curvature diagram, which was calculated by a fiber-based analysis for each section, using the same material of DP models. The yield rotation was expressed as $\theta_y = M_y L_s / 3EI$, where L_s is the shear length, assumed as half the span L, E is the Young modulus of the material and I is the moment of inertia of the section. A value of hardening ratio $\alpha = 0.027$ (ratio between the post-yielding and the elastic slope) was assumed. This assumption was made on the basis of a sensitivity analysis, which was performed by considering a simple fiber model of one story and one bay, assigning the material Steel02 and by varying cross-section of the beam and the bay length. The hardening ratios obtained for different sections are presented in Figure 2b.



(a)



(b)

Figure 2. (a) Moment-rotation relationship for inelastic rotational hinges and (b) hardening ratios for different sections and lengths.

The capping rotation θ_{cap} for beams was determined by taking into account the ratio between the near collapse rotation θ_{NC} and the rotation at zero moment in the post capping range. This ratio was assumed equal to 1.5. The near collapse θ_{NC} was estimated by means of proposal of EC8-3, which defines that θ_{NC} is equal to 8 times yield chord rotation. The moment corresponding to θ_{NC} was related to a 20% drop in the member's peak strength (Mergos & Beyer 2014). Further studies will involve more complex capacity models. However, the so obtained moment-rotation relationship resulted to be quite similar to that obtained by fiber analysis with consideration of MinMax material.
The lumped plasticity elements of the columns were modeled using two different approaches:

- Approach A was based on a trilinear moment-rotation relationship in the rotational inelastic hinges. The axial force from gravity load was considered in the evaluation of yield moment. The hardening ratio was assumed to be equal to *0.027*. Capping rotation and near collapse rotation were defined using the same approach as for the beams (Figure 2a).
- Approach B was based on a bilinear moment-rotation relationship in the rotational inelastic hinges. The first branch of bilinear curve is the same as the first branch of trilinear. The second branch, instead, starts at yielding point and intersects the capping point of trilinear moment-rotation relationship (Figure 2a). In such way, softening behavior was not modeled. It was assumed that NC rotation of this backbone corresponds to NC rotation of trilinear backbone. Note that such approach follows the behavior of the distributed plasticity (DP) models.

A simplified modelling of the panel zones was used in the model, since it is beyond the scope of this study to analyze the specific behavior of beam-column connections. However, the impact of modelling of panel zones was investigated by defining two different models: Lumped plasticity model with Rigid links (LR), in which panel zones were considered as rigid and modelled by rigid links at beams and columns ends, and Lumped plasticity model (L) for which rigid links were neglected. It can be assumed that the model with consideration of rigid links is more realistic, but the complexity of the model and the computational effort was significantly increased. Such a model requires the definition of four nodes for each element and additional grid nodes, as illustrated in Figure 3. The set of investigated models is summarized in Table1.

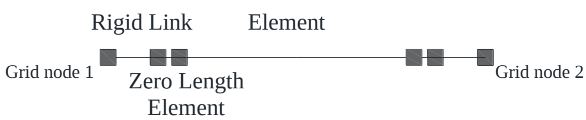


Figure 3. Schematic representation of one element of the LR model

Table 1. Model IDs and first fundamental period for lumped plasticity models of frames

Model ID	T_1 [s] 6S3B	3S3B	3S5B
LR-A **LR-B**	1.10	0.79	0.78
L-A **L-B**	1.25	0.88	0.87

2.3 Pushover analysis and near collapse limit state of structures

In order to assess the strength and the deformation capacity associated with the near collapse limit state of the structures, the sets of models presented in Table 1 were analyzed using conventional pushover analysis. All pushover analyses were performed by consideration of the invariant lateral forces which corresponded to the product of the storey masses and the first vibration mode ("modal" pattern). Pushover curves for frame 6S3B without consideration of rigid links are presented in Figure 4a for lumped plasticity model A and B. The pushover curve for distributed plasticity (DP) model is also added. It can be seen that the pushover curves for L models match well with the pushover curve for DP model. Pushover curve for the L-B model is practically equal to the pushover curve for DP model, since the strategy for modelling of columns was the same for both cases. On the other hand, the softening in the pushover curve of the model L-A is observed at top displacement which is about 20% smaller than the corresponding displacement of the model DP or L-B.

The twelve pushover curves (4 models for each of the 3 MRFs) are presented in Figure 4. It was assumed that the NC limit state of structure occurs when the NC rotation is observed in all the column of one storey (red dot on curves for LR-B models). The ratios between the NC displacement and the height of the structure is around 5% for the six storey frame (6S3B) and 8% for both variants of three storey frames (3S3B, 3S5B). Note that the NC collapse displacement corresponds to a drop in the peak strength between the 25% and 35%.

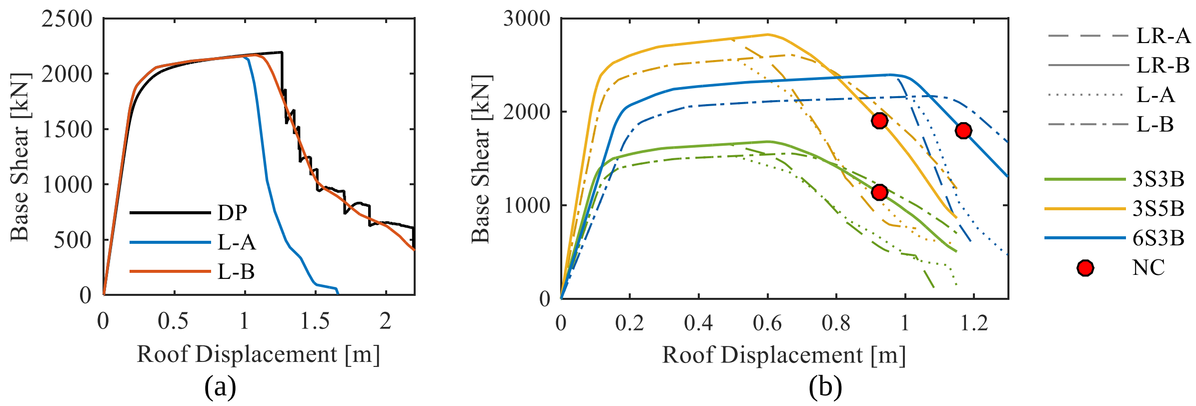


Figure 4. (a) Pushover curves for MRF 6S3B using distributed (DP) and lumped plasticity models (L-A, L-B) and (b) pushover curves with indication of NC point for LR-B models.

3. RISK-TARGETED APPROACH FOR DESIGN OF STRUCTURES

The protection of human lives with an adequate reliability is the fundamental objective when designing a structure of ordinary importance. In the Eurocode this objective is satisfied if a structure is designed taking into account all requirements of the code. For the design of structurs according to the Eurocode 8, the force-based design using linear-elastic analysis is the most used approach. One of the most important parameters in the design is the reduction factor (in Eurocode 8 termed as behaviour factor) which takes into account the ability of the inelastic behaviour of structure during strong earthquakes. According to the Eurocode 8 the behaviour factor (q factor) is used to reduce the seismic forces corresponding to design intensity which, in the case of ordinary buildings, corresponds to a return period of 475 years. However, the structures have to withstand a seismic intensity which is several times higher than the intensity corresponding to 475 years return period to be enough safety (Dolšek et al. 2017). In Eurocode 8 the additional safety is provided by prescribing values of q factor several times smaller than the product of overstrength and ductility reduction factor if the latter is related to NC limit state. Such an approach is necessary in order to achieve adequate collapse risk. However, in the case of petrochemical plants, the main objective for design of such complex infrastructure could be different than for the design of building of ordinary importance. Even so, the formulation of risk-targeted behaviour factor can still be applied to such problems, by realizing that the resilience of petrochemical plants can be controlled by an adequate definition of limit states with corresponding target risk. For the sake of simplicity, this general formulation is omitted in this paper, but, in the following, the theoretical background for the estimation of risk-targeted safety factor and risk-targeted behaviour factor related to target collapse risk is briefly presented.

In risk-targeted design it is assumed that the performance of a structure is adequate if:

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

where P_t is the target probability of collapse and P_C is the probability of collapse, which can be calculated by using the risk equation (Jalayer, 2003; Bradley and Dhakal, 2008; Lazar and Dolšek, 2014):

$$P_C \approx \lambda_C = \int_0^{\infty} P(C \mid S = S_a) \cdot \left| \frac{dH(S_a)}{dS_a} \right| \cdot dS_a \tag{2}$$

where S is a random variable representing the seismic intensity measure (i.e. the spectral acceleration at the fundamental vibration period), $P(C \mid S = S_a)$ is the collapse fragility function, and $H(S_a)$ is the hazard function which expresses the annual rate of exceedance of S_a.

Since engineering practitioners are not familiar with the reliability-based verification format, it is convenient to use equation 2 in order to calculate the risk-targeted seismic intensity causing a designated limit state (i.e. collapse) $S_{a,C,t}$, and to define the risk-targeted safety factor (Dolšek et al. 2017):

$$\gamma_{im} = \frac{S_{a,C,t}}{S_{a,TR}} \tag{3}$$

where $S_{a,TR}$ is the spectral acceleration corresponding to a designated return period (i.e. *TR*=475 years in the case of buildings of ordinary importance) (Dolšek et al., 2017). The basis for calculation of $S_{a,TR}$ are seismic design maps, while the $S_{a,C,t}$ can be evaluated from numerical integration of equation 2 by assuming dispersion of spectral acceleration causing a designated limit state, by considering the hazard curve at the site and by satisfying equation 2 in such a way that P_C is equal to the target risk. If the model for γ_{im} is defined, then equation 3 can be used for risk-targeted design checks using non-linear method of analysis.

In the force-based design, the spectral acceleration causing collapse cannot be estimated. Thus this issue is solved by introducing behaviour factor *q*. However, it has to be emphasized that the conventional deterministic approach for the definition of the behaviour factor (Fardis et al. 2015) cannot be used for the interpretation of the concept of the reduction of seismic forces in conjunction with the target collapse risk. This issue was solved by Žižmond and Dolšek (2017) by developing risk-targeted behaviour factor in closed-form:

$$q = r_s \cdot r_\mu \cdot C_p \tag{4}$$

where r_s is the overstrength factor, r_μ is the ductility factor and C_p defines the correction factor due to the risk-targeted definition of the behaviour factor, which is equal to the inverse of γ_{im}. In the original formulation of the risk-targeted behaviour factor, which was rejected in earthquake Engineering & Structural Dynamics, the simplified solution for C_p was defined also in closed-form (Žižmond & Dolšek, 2017). That solution was equal to the inverse of closed-form solution of γ_{im} (Dolšek et al. 2017) presented in the proposal for Annex F of latest draft of Eurocode 8.

The equation 4 for the calculation of behaviour factor for specified target reliability has been included in the new draft of Eurocode 8 using the following form (Dolšek et al. 2017):

$$q_t = \frac{q_R q_S q_{NC}}{\gamma_{im}} \tag{5}$$

where q_R and q_S account for overstrength, q_{NC} is the component of behavior factor accounting for the deformation capacity, energy dissipation capacity and the seismic response of the structure when measured to the NC limit state and γ_{im} is the risk-targeted safety factor as previously defined. Note that equation 5 is equivalent to equation 4 introduced and developed by Žižmond and Dolšek (2014).

In the equation 4, the overstrength factor r_s represents the ratio between the yield force F_y and the design base share corresponding to the first vibration mode F_{d1} ($r_s = F_y / F_{d1}$), while the ductility reduction factor r_μ (Žižmond and Dolšek 2014) is related to the near collapse limit state and given as the ratio between the available ductility μ_{NC} associated with the NC limit state and inelastic displacement ratio C_1 ($r_\mu = \mu_{NC} / C_1$). The near-collapse ductility of structure can be estimated from pushover analysis since its definition is as $\mu_{NC} = D_{NC} / D_y$, where D_{NC} is the displacement on the pushover curve corresponding to the near collapse limit state, and D_y is the yield displacement obtained from idealized force-displacement relationship of the pushover curve.
For reader, who may not be familiar with the conventional definition of the behaviour factor, it may be interested to explain that the inelastic displacement 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 $S_{a,NC}$ ($D^*_{e,NC}$) (see Žižmond and Dolšek 2014 or 2017). The median value of intensities causing collapse $\tilde{S}_{aNC}$ can be obtained from results of incremental dynamic analyses on equivalent SDOF model (Figure 6b) using the hazard-consistent set of ground motions. $D^*_{e,NC}$ is simply calculated as $D^*_{e,NC} = S_{a,NC} / \omega^2$ where ω^2 is the radial frequency of the SDOF model. D^*_{NC} is determined from the force-displacement relationship of the SDOF model.

In the study presented in this paper, the above-described theory was used to estimate risk-targeted behaviour factors for typical steel MRFs. Overstrength factor and ductility reduction factor were evaluated on the basis of the pushover curves for lumped plasticity models of the structures presented in Section 2. Moreover, three different locations (Bologna, Ljubljana and Skopje) were considered for the evaluation of risk-targeted safety factor. These three sites were chosen because, according to European Seismic Hazard Map 2013 (Woessner et al., 2015), they are characterized by the same reference peak ground acceleration considered for the design of the structures (PGA=0.25 g), while the corresponding acceleration spectra did not match well.

3.3 Calculation of risk-targeted safety factor

The value of risk-targeted safety factor (equation 3) depends on the seismic hazard at the site of interest, the seismic intensity measure, the assumed value of the dispersion of collapse intensities $S_a(T_1)$ and, of course, selected probability of collapse P_t. In order to investigate how these parameters affect the risk-targeted safety factor, the γ_{im} was evaluated for target probability of collapse P_t between $P_t = 5 \cdot 10^{-5}$ and $P_t = 2 \cdot 10^{-4}$ and for the three selected sites (Bologna, Ljubljana and Skopje). It has to be emphasized that a more accurate result would be obtained if national seismic hazard was used. However, SHARE seismotectonic model is considered, since it covers all the European territory. In this way a proper comparison between all the sites can be made.

In order to assess the safety factor γ_{im}, the values of the spectral acceleration corresponding to the return period of 475 years and the risk-target spectral acceleration causing collapse of the structure $S_{a,C,t}$ have to be calculated (equation 3). The $S_{a,C,t}$ were calculated from numerical integration of equation 2. A lognormal distribution of the collapse fragility function $P(C \mid S = S_a)$ was assumed. The logarithmic standard deviation was assumed equal to $\beta = 0.4$ for all analyzed structures, as in (Dolšek et al. 2017). It should be noted that different structures and models have a different first vibration period, therefore many hazard curves have to be used for a proper calculation. As a consequence, different γ_{im} were obtained for the different previously defined MRFs. Note, however, that the values of $S_{a,TR}$ were obtained directly from the type 1 spectrum (Eurocode 8), which was used for the design of the presented structures.

In Figure 5a the safety factor γ_{im} is plotted versus the target probability of failure for the different considered sites, structures and models. The higher is the vibration period, the greater is γ_{im}. For instance, γ_{im} from 4.5 to 4.9 for the site of Skopje and for a target probability $P_t = 5 \cdot 10^{-5}$. The γ_{im} always increases as the target probability of failure decreases: reducing P_t by a factor of 2, the increase of γ_{im} is between the 25% (for Ljubljana) and the 35% (for Bologna and Skopje). The γ_{im} vary between 5 and 2.5 for Bologna and Skopje and between 3.6 and 1.9 for Ljubljana. It can be noted that the values of γ_{im} are quite similar for Skopje and Bologna, while they are lower in the case of Ljubljana. This is firstly due to the slope of the hazard curve (Figure 5b for $T_1 = 1.10s$), which for Ljubljana is greater than for the other two locations. Moreover, the value of $S_{a,TR}$ used for the calculation is the same for the three sites, since the same spectrum from Eurocode 8 is considered, while the uniform hazard spectra show a slight difference in terms of $S_a(T_1)$ between the sites, as shown in figure 5c for T_R=10000 years. However, it should be emphasized that the difference in the evaluated γ_{im} is the consequence of the differences in the seismic hazard of the three sites.

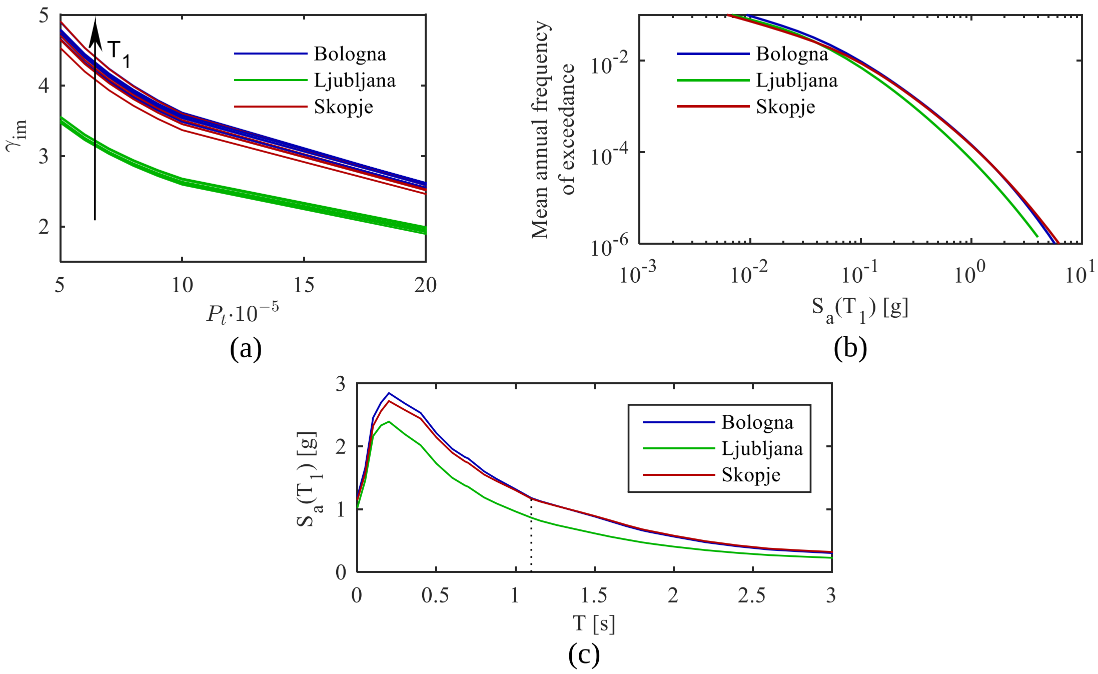
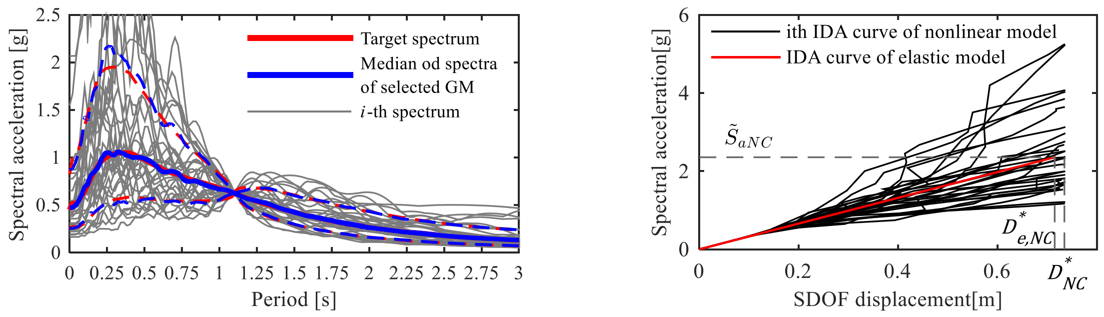


Figure 5. (a) risk-targeted safety factors γ_{im} for all three sites and for different values of target collapse risk, (b) the hazard curves (T_1=1.10s) and (c) hazard uniform spectra for Bologna, Ljubljana and Skopje

3.4 Calculation of risk-targeted q-factor

The risk-targeted behavior factor (equation 4) was calculated for all twelve analyzed models and three sites (Bologna, Ljubljana and Skopje). The yield forces F_y and ductilities μ_{NC} were obtained from pushover curves, whereas the design base share corresponding to first vibration mode F_{d1} was calculated based on the period of the model of frame. The inelastic displacement ratio C_1 used for the calculation of r_μ was estimated from IDA analyses which was performed by equivalent SDOF model of frames and a hazard consistent set of ground motions. In the case of this study, the SDOF model was defined based on idealized pushover curve. It was assumed that the period of the SDOF model was equal to the fundamental vibration period of structure. A hazard consistent set of 30 ground motions was selected for each model of frame and location. The conditional spectrum (CS) (Baker, 2010) was selected as the target spectrum. The CS was estimated based on the seismotectonic model which was used for the seismic hazard assessment of the European region carried out during the SHARE project (Woessner et al., 2015). The target spectrum was defined on the basis of a conditional period corresponding to the first vibration mode of the models of frames. The results of the seismic hazard analysis for locations of frames and a return period of 2475 years were considered. An example of the target spectrum for location of Bologna and spectra of the selected ground motions for $T_1 = 1.10$ s are presented in Figure 6a. The set of ground motions were selected by the slightly modified algorithm proposed by Jayaram et al. (2011), taking into account the Strong ground motion database which was recently established (Šebenik & Dolšek 2016)by combining the NGA (Chiou et al. 2008)and the RESORCE (Akkar et al. 2014) ground motion databases. The selected ground motions correspond to events with magnitudes between 4.5 and 7, and source-to-site distances between 5 and 50 km which were recorded on soil having a shear-wave velocity in the upper 30 m between than 360 and 800 m/s. For ground motion selection the largest considered scale factor was set to 4.



(a) (b)

Figure 6. (a) acceleration spectra of the selected ground motions, the target spectrum for ground motion selection and the median value of the spectral accelerations of selected ground motions and (b) IDA curves using the nonlinear and elastic SDOF model

The intermediate results for the calculation of risk-targeted *q*-factor based on the pushover curves using model LR-A are presented in Table 2, for the three MRFs located in the site of Bologna and with γ_{im} related to target collapse risk $P_t = 5 \cdot 10^{-5}$. In Figure 7 the results for $P_t = 5 \cdot 10^{-5}$ are plotted. .

Table 2. Calculation of q-factor for all MRFs, LR-A model, site Bologna

MRF	r_s	r_μ	γ_{im}	q
6S3B	7.16	5.46	4.77	8.19
3S3B	6.65	5.54	4.78	7.71
3S5B	6.45	5.66	4.66	7.80

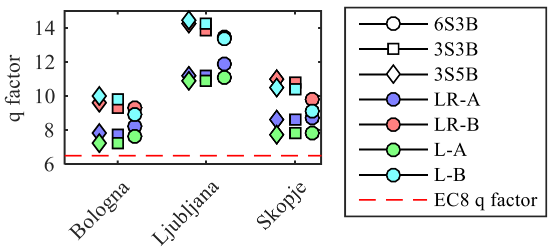


Figure7. Risk-targeted q-factors for MRFs, all structures, all models, all sites

In the conventional force-based design (e.g. Eurocode 8) all the structures for all the sites would be designed with the same q-factor. In a risk-targeted approach, instead, the q-factor values can change significantly depending on structure and site. Different structural models for seismic assessment of the same frame also lead to different results. In particular, the q-factors values vary from 7.2 to 10.0 for Bologna, from 10.9 to 14.5 for Ljubljana and from 7.7 to 11.0 for Skopje. The impact given by the location is the most significant due to the large difference of spectral values for a given return period. The *q* factors for the site of Ljubljana are the highest for the same reason why the γ_{im} are the lowest. Therefore, the uncertainty which affect the q-factor cannot be simply neglected. However, all the values for the risk-targeted *q*-factor are higher than the value of q factor used in design ($q = 6.5$), even if the target probability of collapse is very low. This outcome is investigated in details in the following.

4. RISK-TARGETED Q-FACTOR WITH CONSIDERATION OF LIMIT STATE OF DAMAGE LIMITATION

In the previous section it has been found that the investigated frames are characterized by a large value of overstrength (see Table 2). However, in the process of calculation of overstrength factor, it has not been considered that the design was governed by the damage limitation requirement, which was prescribed by limiting the maximum interstorey drift to 0.075% of the storey height. This means that the design base shear, which was used for the calculation of risk-targeted q-factors, actually did not affect the resistance of structural elements. Therefore the large values of q-factors presented in Figure 7b would not control the design of the new structure, if the DL requirement would be fulfilled.
Thus it can be realized that the design base shear F_d for the evaluation of the overstrength factor is the base shear used for checking the DL limit state, which is, according to Eurocode 8 (CEN 2004), defined as:

$$F_d = \nu F_e \tag{6}$$

where ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement and F_e is the base shear associated with the elastic spectrum of Eurocode 8 and the return period of 475 years. In the Eurocode 8, $\nu = 0.5$ for buildings of ordinary importance. Therefore, if the DL limit state controls the design, the overstrength of the structure can be calculated as:

$$r_{s,DL} = \frac{F_y}{\nu F_e} \tag{7}$$

The damage limitation requirement can be fulfilled by an adequate stiffness. When DL requirement is not fulfilled, dimensions of cross-sections have to be increased, which consequently increases the strength. If it is not necessary to account for the DL requirement, the resulting structures can have very low strength, which may not necessary be safe against collapse ($P_C > P_t$).

The trend of the new draft of Eurocodes is to neglect the damage limitation requirement in the design phase. In such a case, a more accurate evaluation of behaviour factors would be needed in order to ensure that $P_C \le P_t$ and, more important, to ensure that the expected annual loss due to earthquakes would be reasonably low. Without consideration of DL requirement, the expected performance of MRFs may not be economical on longer period of time. The simplest way to solve this issue is to increase the strength of the structure. It makes sense that the strength is increased in such a way that the overstrength factor used for the calculation of risk-targeted behaviour factor is estimated according to equation 7.

The values of DL-based overstrength factors, presented in Table 3 for LR model, are much lower than those presented in Table 3. It is quite interesting that for all three structural configurations the overstregth factor with consideration of DL requirement is around 2. The $r_{s,DL}$ is mainly affected by the difference between the mean and the design values of the material properties, by the selection of sections from catalogues, by structural system and by some other design factors (e.g. capacity design requirement). It can also be shown from theory that based on few assumptions, the $r_{s,DL}$ is equal to around 2.

Table 3. $r_{s,DL}$ values for LR model

Frame	**6S3B**	**3S3B**	**3S5B**
$\boldsymbol{r_s}$	2.20	2.00	1.98

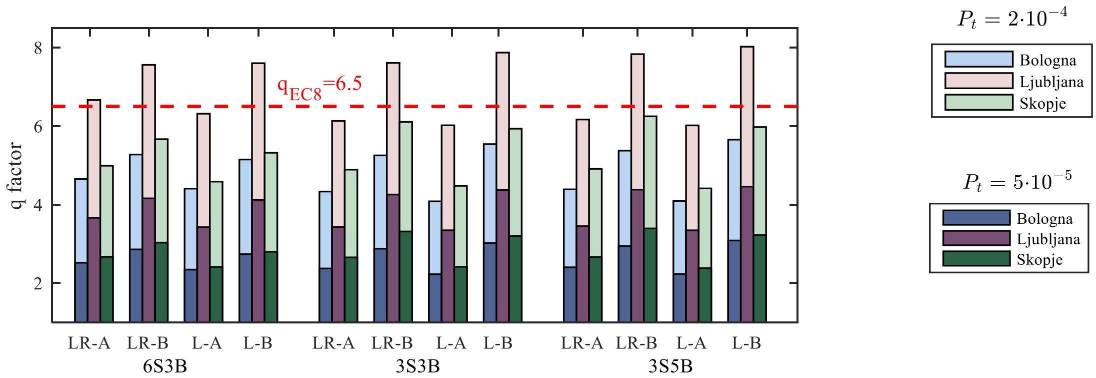


Figure 8. The risk-targeted q-factors with consideration of DL requirement for all investigated MRFs, model types and sites and for two values of target risk $P_t = 5 \cdot 10^{-5}$ and $P_t = 2 \cdot 10^{-4}$

The risk-targeted behaviour factors obtained by assuming $r_s = r_{s,DC}$ are presented in Figure 8 for $P_t = 2 \cdot 10^{-4}$ and $P_t = 5 \cdot 10^{-5}$. These values are up to 77% smaller than those presented in Figure 7. If $P_t = 2 \cdot 10^{-4}$, the q-factors varies from 4.1 to 5.7, from 6.0 to 8.0 and from 4.4 to 6.2, respectively, for Bologna, for Ljubljana and for Skopje. Reducing P_t by a factor of 4 (from $P_t = 2 \cdot 10^{-4}$ to $P_t = 5 \cdot 10^{-5}$), the q-factors decrease up to the 47%. Moreover, different modeling approaches lead to different q factors, which varied up to the 30% (for the case of Skopje, MRF 3S5B). The large difference between the sites is due to the difference in terms of γ_{im}, as explained in the previous section.

The resulting q-factors, which are meant to fulfill the NC limit state without applying the DL requirements, are lower than the value proposed by EC8 ($q = 6.5$). Therefore, if the designer doesn't want to fulfill DL limitations (like in the new draft of Eurocode 8), the risk-targeted q-factors (Figure 8) should be used in order to ensure $P_C \le P_t$. However, it should be noted that all the values presented for q factors were evaluated considering overstrength and ductility factors based on pushover analysis, without checking the design with iterative process in order to prove their accuracy.

5. CONCLUSIONS

The behaviour factor based on risk formulation is site-specific, structure-specific and depends on a target risk. Thus the formulation can be very useful for the calculation of adequate values of the behaviour factor for complex infrastructures, for which the acceptable target risk should be lower than that of ordinary buildings, and for which very specific limit states can be defined. In addition, the formulation of the risk-targeted behaviour factor can be used for different studies which can help to understand the dependence of the behaviour factor to all input parameters.

In the study it was shown that the values of the behaviour factors quite significantly depend on the seismic hazard function although only the sites with the same level of seismic intensity were taken into account. However, these results can be carefully interpreted, since the seismic hazard function associated with long return periods are highly uncertain. It was also found that the risk-targeted safety factor increases with decreasing target probability of failure. For example, reducing P_t by a factor of 2, the value of γ_{im} increased up to the 35%. The behaviour factor can be also quite sensitive to the modelling uncertainties. However, further studies are needed in order to better consider the capacity of steel columns and beams.
This study also showed that, if the DL requirements are not considered in the design phase, the behaviour factors should decrease up to the 77% in order to ensure $P_C \le P_t$. This outcome, however, is affected by several approximations, which should be further investigated

It is hoped that the calibration of risk-targeted behaviour factors and safety factors for steel structures is the first step to more comprehensive procedure for resilience-targeted design of petrochemical plant, which will overcome the current code provisions. This can be of great interest for industry as well as for community, which is so dependent on energy resources and not yet resilient in reaction to natural-hazard triggered by technological accidents.

6. ACKNOWLEDGMENTS

The authors gratefully acknowledge Professor Aurel Stratan, who kindly shared with the authors the models of steel structures used as basis of this work. The work presented herein was carried out with a financial grant from H2020 Marie Skłodowska-Curie Innovative Training Network, within the research project XP-RESILIENCE (G.A. 721816). This support is gratefully acknowledged

7. REFERENCES

Akkar, S., Sandikkaya, M. a., Şenyurt, M., Azari Sisi, A., Ay, B.O., Traversa, P., Douglas, J., Cotton, F., Luzi, L., Hernandez, B., Godey, S. 2014. Reference database for seismic ground-motion in Europe (RESORCE). Bulletin Of Earthquake Engineering 12, 1: 311–339.

Baker, J. 2010. Conditional Mean Spectrum: Tool for Ground-Motion Selection. Journal Of Structural Engineering 137, 3: 322–331.

Bradley, B.A., Dhakal, R.P. 2008. Error estimation of closed-form solution for annual rate of structural collapse. Earthquake Engineering & Structural Dynamics 37, 15: 1721–1737.

CEN. 2005. Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildingsEuropean Committee for Standardisation2005Brussels1993–1–1:2003

CEN. 2004. Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildingsEuropean Committee for Standardisation2004 Brussels 1998–1:2004

Chiou, B., Darragh, R., Gregor, N., Silva, W. 2008. NGA project strong-motion database. Earthquake Spectra 24, 1: 23–44.

Dolšek, M., Lazar Sinković, N., Žižmond, J. 2017. IM-based and EDP-based decision models for the verification of the seismic collapse safety of buildings. Earthquake Engineering And Structural Dynamics: 1–18.

Dolšek, M., Kosič, M., Žižmond, J., Lazar Sinković, N. 2017. Development of Eurocode 8, Proposal for Annex F (Informative) Simplifed reliability-based verification format, Rev. 3, University of Ljubljana, 16.6.2017, Ljubljana.

Fardis, M.N., Carvalho, E.C., Fajfar, P., Pecker, A. 2015. Seismic Design of Concrete Buildings to Eurocode 8. 1st. izd. Boca Raton, CRC Press

Fischinger, M., Fajfar, P. 1990. On the response modification factors for reinforced concrete buildings. V: Proccedings of Fourth U.S. National Conference on Earthquake Engineering. Palm Springs, California, USA, 20-24 May 1990: 1–10 pp.

Jalayer, F. 2003. Direct probabilistic seismic analysis: Implementing non-linear dynamic assessment. Doctoral dissertation. Stanford, Stanford University, Department of Civil and Environmental Engineering

Lazar, N., Dolšek, M. 2014. Incorporating intensity bounds for assessing the seismic safety of structures: Does it matter? Earthquake Engineering & Structural Dynamics 43, 5: 717–738.

Lazar Sinković, N., Brozovič, M., Dolšek, M. 2016. Risk-based seismic design for collapse safety. Earthquake Engineering & Structural Dynamics 45, 9: 1451–1471.

McKenna, F., Fenves, G.L. 2010. Open System for Earthquake Engineering Simulation (OpenSees). Pacific Earthquake Engineering Research Centerhttp://opensees.berkeley.edu

Mergos, P.E., Beyer, K. 2014. Loading protocols for European regions of low to moderate seismicity. Bulletin Of Earthquake Engineering 12, 6: 2507–2530.

Šebenik, Ž., Dolšek, M. 2016. Strong ground motion database. Ljubljana, University of Ljubljana, Faculty of Civil and Geodetic Engineering, IKPIR

Tsitos, A., Elghayouli, A., Stratan, A., Sabău, G., Dubina, D., Gentili, F., Rebelo, C., da Silva, L.S. 2017. Seismic performance assessment of frames. Delivarable D-WP1-2.

Uang, C.M. 1991. Establishing R (or Rw) and Cd factors for building seismic provisions. Journal Of Structural Engineering 117, 1: 19–28.

Woessner, J., Laurentiu, D., Giardini, D., Crowley, H., Cotton, F., Grünthal, G., Valensise, G., Arvidsson, R., Basili, R., Demircioglu, M.B., Hiemer, S., Meletti, C., Musson, R.W., Rovida, A.N., Sesetyan, K., Stucchi, M. 2015. The 2013 European Seismic Hazard Model: key components and results. Bulletin Of Earthquake Engineering 13, 12: 3553–3596.

Žižmond, J., Dolšek, M. 2017. The formulation of risk-targeted behaviour factor and its application to reinforced concrete buildings. V: 16th World Conference on Earthquake. Santiago, Chile, 9-13 January: 1–11 pp.