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Formulation of risk-targeted seismic action for the force-based seismic design of structures

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Summary

A risk-targeted design spectral acceleration and the corresponding seismic design action for the force-based design of structures is introduced by means of two formulations. The first one called direct formulation utilizes the seismic hazard function at the site of the structure. Because the seismic action defined in the codes is often associated with a designated return period, an indirect formulation is also introduced. It incorporates a risk-targeted safety factor that can be used to define a risk-targeted reduction factor. It is shown that the proposed formulations give analogical results and provide an insight into the concept of the reduction of seismic forces for the force-based seismic design of structures if the objective is defined by a target collapse risk. The introduced closed-form solution for the risk-targeted reduction factor can be used to investigate how the target collapse risk, the seismic hazard parameters, the randomness of the seismic action, and the conventional parameters (ie, the overstrength factor and the deformation and energy dissipation capacity) affect the seismic design forces in the case of force-based design. However, collaborative research is needed in order to develop appropriate models of these parameters. In the second part of the paper, the proposed formulations are demonstrated by estimating the risk-targeted seismic design action for a six-storey reinforced concrete building. By verifying the collapse risk of the designed structure, it is demonstrated that the risk-targeted seismic action, in conjunction with a conventional force-based design, provided structure with acceptable performance when measured in terms of collapse risk.

KEYWORDS

force-based design, performance-based earthquake engineering, reduction factor, risk-targeted seismic intensity, seismic hazard, seismic risk

1 | INTRODUCTION

Guidelines introducing minimum requirements for the construction of buildings in seismic regions were introduced in Slovenia in 1896, 1 year after the Easter Earthquake, which struck Ljubljana in 1895.¹ About 15 years later, after the Messina earthquake,² the first guidelines for the computational design of structures to resist horizontal loads were proposed in Italy. Until today, many different design philosophies and structural codes have been introduced.³ However,

the force-based seismic design procedure is still commonly used today's practice for the earthquake-resistant design of structures.^{4,5}

The quantitative risk-based design represents an alternative to the procedures prescribed in current building codes. Cornell et al⁶ proposed a probabilistic basis for the design of steel moment-resisting frames, which was developed in terms of a safety-checking format. This is followed by a detailed description of what is called demand and capacity factor design, performed by using a closed-form analytical expression for the mean annual frequency of exceeding a particular structural performance level.⁷ Decision support tools for the conceptual performance-based design were introduced as well (eg, Krawinkler et al⁸ and Zareian and Krawinkler^{9,10}). The challenge of the proposed approach is to use adequate relationships between the different components controlling the performance-based design. For example, a model of relationship between the selected intensity measure and the engineering demand parameters (eg, SPO2IDA,¹¹ WIDA¹²), which permits rapid evaluation of various structural systems, is needed.

In addition to the conceptual performance-based design of buildings, an automated performance-based design methodology has been proposed,¹³ which involves computation of the expected annual losses of given steel framing systems in conjunction with a genetic algorithm. However, it is not likely that such computationally very demanding procedures will be prescribed for the design of realistic structures in the newer versions of structural codes. For this reason, several simplified performance-based methodologies have been introduced, which, though still based on several assumptions, represent an improvement in comparison with the seismic design procedures that are currently implemented in building codes. For example, Vamvatsikos and Aschheim¹⁴ proposed performance-based design via Yield Frequency Spectra, which depend on the target mean frequency of exceedance and the assumed ductility of a structure. Franchin and Pinto¹⁵ and Franchin et al¹⁶ proposed a method for the probabilistic iterative design of reinforced concrete structures using linear elastic analysis. An iterative pushover-based procedure, which could be used when designing reinforced concrete frames for target collapse risk, has also been proposed.^{17,18} However, the initial design of a structure is of great importance when it is based on the iterative procedure and nonlinear analysis.

Current structural codes may represent a sound basis for initial design in the case of an iterative risk-targeted nonlinear design procedure, because some of the codes already take into account the concept of risk-targeted design (eg, ASCE 7-10,⁵ ASCE 43-05¹⁹). ASCE 43-05 provides seismic design criteria, which are more stringent than those prescribed in regular building codes. Therefore, it cannot be directly applied to ordinary buildings, for which the force reduction factor accounts for both the overstrength and the ductility reduction factor. On the other hand, in ASCE 7-10,⁵ the definition of seismic design action is based on the use of risk-targeted seismic design maps,²⁰ which were derived by the adoption of a constant target collapse risk (eg, 1% over a period of 50 years), and a single value of standard deviation of the intensity causing collapse. In order to attain the performance goal, the R factors still need to be calibrated for a different type of structures (FEMA P-695²¹). In Europe, a pilot study for the definition of risk-targeted seismic design maps was carried out in France.²² Some other authors²³ have proposed a methodology for the probabilistic assessment of behaviour factor q .

Regardless of the definition of the seismic design action, the force-based design approach involves the concept of the reduction of seismic forces, which has been used for decades (eg, ATC 3-06,²⁴ Fajfar³). Several authors have attempted to decompose the reduction factor in order to provide a better insight into its estimation. Fischinger and Fajfar²⁵ and Fardis et al²⁶ suggested that the reduction of seismic forces could be made equal to a ductility factor multiplied by an overstrength reduction factor. A similar proposal was made by Uang.²⁷ A comprehensive overview of models for the definition of reduction factors was presented by Miranda and Bertero.²⁸ At that time, the available models of reduction factors were calibrated for the purpose of deterministic analysis, whereas later on, when risk assessment had become a more acceptable concept, reduction factors were estimated by means of probabilistic frameworks (eg, FEMA P-695,²¹ Chryssanthopoulos et al²⁹).

The primary objective of this paper is to introduce a formulation of risk-targeted design spectral acceleration, which can be used for the definition of risk-targeted seismic action for force-based design. In the first part of the paper, the direct and indirect formulations for the calculation of the risk-targeted design spectral acceleration are introduced. This is followed by a discussion aimed at providing an insight into all the components of the proposed formulation, which, however, also incorporates the conventional parameters (ie, the overstrength factor and the ductility reduction factor). For ease of reading and completeness of the paper, part of the derivation is presented in an Appendix. In the second part of the paper, the proposed approach for the definition of the risk-targeted seismic action for the force-based design of structures is demonstrated and validated by means of design and seismic risk assessment of a six-storey reinforced concrete frame building.

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

The derivation starts from the assumption that the collapse prevention objective is fulfilled when the probability of collapse of a structure P_C is less than the target (acceptable) annual probability of collapse $P_{C,a}$:

$$P_C \leq P_{C,a}. \quad (1)$$

The target (acceptable) annual probability of collapse (hereinafter called target collapse risk) is an input parameter for the design. Thus, $P_{C,a}$ is considered as a known parameter, whereas the objective of the design is that the realized probability of collapse P_C , once the structure is designed, is as close as possible to $P_{C,a}$. Because in the design phase a structure is not yet defined, the probability of collapse P_C cannot be explicitly estimated according to the conventional risk equation (eg, Pinto et al,³⁰ Bradley and Dhakal,³¹ Vamvatsikos,³² Jalayer,³³ Eads et al,³⁴ Lazar and Dolšek³⁵):

$$P_C \approx \lambda_C = \int_0^\infty P(C|IM = im; im_C, \beta_{im,C}) \cdot \left| \frac{dH(im)}{dim} \right| \cdot dim \quad (2)$$

where, λ_C is the mean annual frequency of collapse exceedance, which is practically equal to the annual probability of collapse P_C (hereinafter called collapse risk) due to its low value, $H(im)$ is the seismic hazard function, ie, the mean annual frequency of exceedance of intensity measure im , $P(C|IM = im; im_C, \beta_{im,C})$ is the collapse fragility function, which is usually defined by the lognormal cumulative distribution function having two parameters, ie, by the median intensity causing collapse im_C , and the corresponding standard deviation $\beta_{im,C}$. However, seismic hazard function $H(im)$ from Equation 2 is considered to be known in the design phase and independent of the structural response.

In order to calculate collapse risk P_C , the parameters of the collapse fragility function, ie, im_C and $\beta_{im,C}$, have to be known, which is not the case in the design phase. Thus, Equation 2 can be used in the assessment phase, once the structure has already been defined. However, in the design phase, it is possible to estimate the acceptable (target) collapse fragility function based on a known value of target collapse risk $P_{C,a}$. In this case, Equation 2 can be rewritten as follows:

$$P_{C,a} \approx \lambda_{C,a} = \int_0^\infty P(C|IM = im; im_{C,a}, \beta_{im,C}) \cdot \left| \frac{dH(im)}{dim} \right| \cdot dim \quad (3)$$

where $P(C|IM = im; im_{C,a}, \beta_{im,C,a})$ is the target (acceptable) collapse fragility function, which has to be calculated based on known $P_{C,a}$ and seismic hazard function $H(im)$. However, the target collapse fragility function has two parameters: the target (acceptable) median intensity causing collapse $im_{C,a}$ and the corresponding standard deviation $\beta_{im,C,a}$. Because the standard deviation of the seismic intensity causing collapse is not very sensitive parameter with respect to P_C and because the objective in the design is that $P_C \approx P_{C,a}$, it can be assumed that $\beta_{im,C,a} \approx \beta_{im,C}$. Nevertheless, $\beta_{im,C}$ has to be assumed in the design phase, although this assumption may not be critical, because it was shown^{36,37} that the value of $\beta_{im,C}$ does not vary significantly for specific types of structures. Based on these assumptions, $im_{C,a}$ is the only unknown of Equation 3 and can thus be obtained by solving Equation 3. The result of such calculation is the target (acceptable) collapse fragility function (see Figure 1), which is thus known in the design phase.

In order to fulfill the performance objective (Equation 1), the engineer has to design a structure in such a way that the actual (realized) collapse fragility function will be slightly on the right-hand side of the acceptable collapse fragility function (Figure 1). In this case, the structure will be slightly over-designed in terms of acceptable collapse risk, whereas in the opposite case, when the actual (realized) collapse fragility function is on the left-hand side of the target (acceptable) collapse fragility function (Figure 1), the structure will be considered under-designed and thus not safe against collapse.

By inserting Equations 2 and 3 into Equation 1, it can be realized that the risk-based objective for collapse prevention, as defined in Equation 1, can be transformed into an intensity-based objective:

$$im_C \geq im_{C,a}. \quad (4)$$

which is more familiar to engineers, because they can directly control the intensity measure causing collapse of structure im_C through the strength, ductility, and cumulative dissipative energy of the structure.

FIGURE 1 Target (acceptable) collapse fragility function, collapse fragility function of the under-designed structure, and collapse fragility function of the over-designed structure, which satisfies the design objective set by Equation 4

Equation 4 can also be interpreted schematically by observing Figure 1. If the collapse fragility function of the investigated structure is on the right-hand side with respect to the acceptable (target) fragility function, then the engineer has satisfied Equation 4 as well as Equation 1.

Because the primary objective of this paper is to estimate the risk-targeted seismic intensity for force-based design, the spectral acceleration at the first vibration period of a structure, S_e , is selected as intensity measure. Thus, in the derivation that follows, the general notation for the intensity measure, im , is replaced by S_e . Thus, Equation 4 can be rewritten as

$$S_{e,C} \geq S_{e,C,a}. \quad (5)$$

In general, different approaches can be used to design a structure that fulfills the requirement defined by Equation 5. Iterative design of a structure using nonlinear analysis (eg, Lazar Sinković et al¹⁸) can provide a fairly accurate solution. In this approach, the cross-sectional dimensions and reinforcement are increased until it is shown, by means of seismic performance assessment using nonlinear (static or dynamic) analysis, that $S_{e,C}$ is slightly greater than $S_{e,C,a}$. However, in reality, it is challenging to assess $S_{e,C}$ exactly, because the estimation of the collapse of complex structures is a very uncertain procedure and is often associated with numerical nonconvergence. It is therefore convenient to check the performance of the structure for a less severe limit state, ie, a near-collapse (NC) limit state. The transition between the collapse and NC limit state was introduced in a previous study³⁸ by means of a limit-state reduction factor γ_{ls} , which relates the median spectral acceleration causing collapse $S_{e,C}$ to the median spectral acceleration causing the NC limit state $S_{e,NC}$ of a structure³⁸:

$$\gamma_{ls} = \frac{S_{e,C}}{S_{e,NC}}. \quad (6)$$

By implementing the limit-state reduction factor γ_{ls} in Equation 5, the design condition is defined as follows:

$$S_{e,NC} \geq \frac{S_{e,C,a}}{\gamma_{ls}} = S_{e,NC,a} \quad (7)$$

where $S_{e,NC,a}$ is an acceptable median spectral acceleration corresponding to the first vibration period and causing the NC limit state of a structure (also called risk-targeted spectral acceleration causing NC limit state of a structure). Note that, in general, the limit-state reduction factor can be related to any other limit state.

Equation 5 or Equation 7 can only be verified on the basis of nonlinear analysis. Because force-based design is based on linear elastic analysis (eg, a conventional force-based design using response spectrum analysis) and it takes into account different design factors (eg, Žižmond and Dolšek³⁹), additional assumptions are needed to estimate the risk-targeted spectral acceleration for force-based design $S_{e,D,a}$. It has to be realized that, due to the design factors in force-based design and the inherent ductility of the structure,³⁹ the actual strength and deformation capacity of the structure are greater than the corresponding design values. As a consequence, the risk-targeted spectral acceleration causing the NC limit state of structure $S_{e,NC,a}$ has to be further reduced in order to estimate the risk-targeted spectral acceleration for the force-based design $S_{e,D,a}$. This reduction is performed by using the available reduction factor r_{NC} . The r_{NC} has to be assumed in the design because it depends on the available ductility, the overstrength reduction factor,

and the deformation and energy dissipation capacity, which are not precisely known in the design phase. Once the structure has been fully defined, the r_{NC} can be calculated as the ratio between the spectral acceleration causing the NC limit state of structure $S_{e,NC}$ and the risk-targeted design spectral acceleration for the force-based design $S_{e,D,a}$ (also called risk-targeted design spectral acceleration):

$$r_{NC} = \frac{S_{e,NC}}{S_{e,D,a}}. \quad (8)$$

One option to calculate the risk-targeted design spectral acceleration $S_{e,D,a}$ is to develop a model of the available reduction factor r_{NC} based on parametric studies for different types of structures. In such a case, the direct formulation of the risk-targeted design spectral acceleration $S_{e,D,a}$ can be introduced by inserting Equation 8 into Equation 7. $S_{e,D,a}$ can then be formulated as follows:

$$S_{e,D,a} = \frac{S_{e,NC,a}}{r_{NC}}. \quad (9)$$

Note that in Equation 9 the symbol greater or equal was replaced by the symbol equal, because the objective in the design is that $S_{e,NC}$ is as close as possible to $S_{e,NC,a}$.

The risk-targeted spectral acceleration causing NC of structure $S_{e,NC,a}$ (ie, the denominator in Equation 9), depends on the assumed value of the dispersion of the spectral acceleration causing collapse $\beta_{Se,C}$, the target collapse risk $P_{C,a}$, and the hazard function. $S_{e,NC,a}$ can be assessed as the ratio between $S_{e,C,a}$ and a limit-state reduction factor γ_{ls} , while $S_{e,C,a}$ can be calculated iteratively using Equation 3, or, under some assumptions, also by using a closed-form expression as discussed in the following section. Values of the other parameter of Equation 9 (ie, the available reduction factor r_{NC}) have to be assumed. There are some possibilities for further decomposition of the available reduction factor r_{NC} , as discussed in Section 3, which provide some guidance for engineers as how to estimate this factor in the design.

The formulation of the risk-targeted design spectral acceleration $S_{e,D,a}$ is based on the assumption that seismic hazard is defined by a seismic hazard function. If seismic hazard is defined by the traditional uniform seismic hazard maps related to a designated return period T_R (eg, Eurocode 8,⁴ Uniform Building Code⁴⁰), then the indirect formulation of $S_{e,D,a}$ has to be introduced. In this case, $S_{e,D,a}$ is not calculated directly from $S_{e,NC,a}$ but indirectly via spectral acceleration $S_{e,TR}$ corresponding to the return period of seismic intensity T_R . Thus, the risk-targeted approach can be incorporated in the design by introducing a risk-targeted reduction factor, which will be termed as q_a . By definition, the risk-targeted design spectral acceleration $S_{e,D,a}$ can be calculated as follows:

$$S_{e,D,a} = \frac{S_{e,TR}}{q_a}. \quad (10)$$

The risk-targeted reduction factor can then be easily derived by considering that $S_{e,D,a}$ from the indirect formulation (Equation 10) should be equal to that from the direct formulation (Equation 9):

$$q_a = \frac{S_{e,TR}}{S_{e,NC,a}} r_{NC}. \quad (11)$$

Note that the ratio between $S_{e,NC,a}$ and $S_{e,TR}$, was defined in a previous study as the risk-targeted safety factor³⁸:

$$\gamma_{im} = \frac{S_{e,NC,a}}{S_{e,TR}}. \quad (12)$$

γ_{im} can be interpreted as the multiplication factor for $S_{e,TR}$ (ie, seismic demand) in order to obtain $S_{e,NC,a}$ (ie, risk-targeted seismic capacity). However, if the risk-targeted safety factor for limit-state seismic intensity γ_{im} is introduced into Equation 11, then the risk-targeted reduction factor for a specific target collapse risk ($P_{C,a}$) can be expressed in the following form:

$$q_a = \frac{r_{NC}}{\gamma_{im}}. \quad (13)$$

The definition of $S_{e,D,a}$ according to Equation 10 is very useful in the case when the seismic hazard is defined by uniform seismic hazard map related to a designated return period. Such an approach requires a model of risk-targeted safety factor γ_{im} , which is associated with $S_{e,TR}$. A possible procedure for the estimation of γ_{im} is presented in Dolšek et al.³⁸ Alternatively, γ_{im} can be derived in closed form as presented in Section 3, if the slope of hazard function k at

the site of the structure is known. Note that γ_{im} can also be assessed on the basis of an entire hazard function (ie, by using Equation 12, where $S_{e,NC,a}$ is calculated iteratively by satisfying Equation 3).

The two formulations (Equations 9 and 10) of the risk-targeted design spectral acceleration are equivalent, but several assumptions are needed in order to estimate $S_{e,D,a}$. This means that some additional insight is needed as how to apply the risk-targeted design spectral acceleration to the seismic design action for the force-based design. This is provided in the next section.

3 | DISCUSSION

The proposed formulations of risk-targeted design spectral acceleration $S_{e,D,a}$ can be implemented in several different ways, depending on the objective (eg, conceptual design, the development of codes) and on the available information about seismic hazard and other parameters that have to be assumed in the estimation of $S_{e,D,a}$. However, $S_{e,D,a}$ cannot be precisely estimated, regardless of the formulation used. The accuracy of the estimated $S_{e,D,a}$ depends on the accuracy of the estimated $\beta_{Se,C}$, γ_{ls} , and r_{NC} , the values of which need to be prescribed in the process of the estimation of $S_{e,D,a}$. This is because these parameters are related to the seismic performance of the structure, which is not yet defined at the beginning of the design phase. However, the proposed formulation of $S_{e,D,a}$ can be used to set up a framework of parametric studies aimed at developing models of $\beta_{Se,C}$, γ_{ls} , and r_{NC} for different types of structures and different levels of seismic hazard. Additionally, $S_{e,D,a}$ is affected by the value of target collapse risk $P_{C,a}$ and by return period T_R of acceleration spectrum $S_{e,TR}$, if the seismic hazard is defined on the basis of the uniform hazard map corresponding to return period T_R .

In order to provide some insight into the parameters needed for the estimation of $S_{e,D,a}$, a discussion on each of these parameters is given below. Some guidelines are also provided as to how the different parameters affecting the risk-targeted design spectral acceleration can be estimated in the design.

3.1 | Available reduction factor r_{NC}

The available reduction factor r_{NC} is the most important parameter for an adequate estimation of $S_{e,D,a}$. It depends on the seismic response of the structure and on the design factors in the force-based design, including $S_{e,D,a}$. Parameter r_{NC} is directly implemented in Equation 9 or indirectly in Equation 10 through risk-targeted reduction factor q_a (Equation 13). Thus, it is useful to have some additional insight into its estimation. The available reduction factor r_{NC} depends on the overstrength, deformation capacity, and cumulative energy dissipation capacity of the structure. For this reason, it makes sense that the r_{NC} factor is decomposed. Different formulations are possible. In the case of pushover analysis, the available reduction factor r_{NC} can be decomposed into an overstrength reduction factor r_s and a ductility reduction factor $r_{\mu NC}$.^{25,26} Note that in the sequel, the ductility reduction factor is related to the NC limit state. Starting from Equation 8, r_{NC} can be decomposed as a product of two spectral acceleration ratios, as follows:

$$r_{NC} = \frac{S_{e,NC}}{S_{e,Y}} \cdot \frac{S_{e,Y}}{S_{e,D,a}} = r_{\mu NC} \cdot r_s \quad (14)$$

where $S_{e,Y}$ is the spectral acceleration at yielding, $r_{\mu NC} = S_{e,NC}/S_{e,Y}$ is a ductility reduction factor, and $r_s = S_{e,Y}/S_{e,D,a}$ is the overstrength reduction factor. The definition of the available reduction factor in Equation 14 is based on seismic intensity measure, ie, the spectral acceleration at the first vibration period of a structure. In general, $r_{\mu NC}$ and r_s can be estimated from the results of parametric studies, taking into account different building classes or even building subclasses.

However, r_{NC} can also be expressed directly by the risk-targeted design base shear $F_{D,a}$, yield strength F_Y , the available system ductility μ_{NC} (ie, the ratio between the NC displacement of structure D_{nNC} and the corresponding yield displacement D_Y), and the inelastic deformation ratio C_I ⁴¹⁻⁴³:

$$r_{NC} = r_{\mu NC} \cdot r_s = \frac{\mu_{NC}}{C_I} \cdot \frac{F_Y}{F_{D,a}} \quad (15)$$

According to Equation 15, the overstrength reduction factor r_s can be interpreted as the ratio between yield strength (F_Y) and the risk-targeted design base shear ($F_{D,a}$), whereas $r_{\mu NC}$ is defined as the available system ductility divided by the inelastic deformation ratio C_I . Equation 15 is an application of Equation 14. Equation 15 can be precisely applied to a single-degree-of-freedom structure, as it is shown in the Appendix, where step-by-step transition from Equations 14 to

15 is presented. However, the example of a possible approach to the estimation of the inelastic displacement ratio C_I , the overstrength reduction factor r_s , and the available system ductility $r_{\mu NC}$, which are needed in the design phase for the calculation of the available reduction factor r_{NC} , are presented in Section 5.

When estimating the available reduction factor r_{NC} for structures with very short first vibration periods, it can be considered that $r_{\mu NC}$ (ie, μ_{NC}/C_I in Equation 15) has a value, which is approximately equal to 1, because the inelastic displacement demand of such systems is significantly greater than the displacement demand corresponding to a linear elastic system (ie, the inelastic deformation ratio C_I , which is defined as a ratio between near collapse displacements of inelastic and linear elastic structures, is very large in comparison with μ_{NC}). In this case, it becomes essential to adequately assess the overstrength reduction factor r_s . For structures having longer first vibration periods it can be assumed that the displacement of the inelastic and linear elastic structure can be considered equal (called equal displacement rule [eg, Veletsos and Newmark,⁴⁴ Fajfar⁴⁵], $C_I = 1$). In this case, it becomes more important to focus on the estimation of the available system ductility of the structure μ_{NC} , because it can be significantly greater than the overstrength reduction factor r_s . However, this is true only if the seismic design action is decisive for the design of a structure. In areas with low seismicity, where minimum requirements for the dimensions of the available elements control the design, the overstrength reduction factor can be very large, which means that this fact should be adequately taken into account in the estimation of r_s .

The inelastic deformation ratio C_I can be estimated from models that were developed in past studies (eg, Vidic et al,⁴⁶ Miranda⁴²). In a more general case, the inelastic deformation ratio C_I can be estimated by nonlinear dynamic analyses (eg, incremental dynamic analysis,⁴⁷ multistribe analysis⁴⁸). In the example (Section 5), C_I is estimated by incremental dynamic analysis. Note that the value of C_I depends also on the intensity measure and on the shape of the spectrum for the selection of ground motions for seismic performance assessment. Different types of the spectrum can be used for the selection of ground motions for seismic performance assessment of structures (eg, a code-based elastic spectrum, a uniform hazard spectrum, or a conditional spectrum). However, according to Baker,⁴⁹ the most appropriate spectrum for the selection of ground motions for hazard consistent seismic performance assessment is the conditional spectrum, which provides an unbiased assessment in comparison to other types of response spectra. The conditional spectrum can also be defined by using the average spectral acceleration as an intensity measure.⁵⁰ The value of C_I calculated on the basis of the conditional spectrum can be smaller than that based on the uniform hazard spectrum or code-based response spectrum, which means that the available reduction factor r_{NC} assessed on the basis of the conditional spectrum can be greater than that based on the uniform hazard spectrum or code-based response spectrum.

3.2 | Limit-state reduction factor γ_{ls}

The limit-state reduction factor was introduced in Dolšek et al,³⁸ because simplified nonlinear models prescribed by the codes cannot predict the collapse of complex structures. It relates the spectral acceleration causing collapse to the spectral acceleration causing a NC limit state (Equation 6).

In general, it is quite difficult to estimate values for the limit-state reduction factor γ_{ls} , because the introduction of this factor is not justified if no tools are available to create adequate simulations of the collapse of a structure. However, boundary values of γ_{ls} can be defined from the observations of the seismic response of structures in the nonlinear range. In the case of low-rise buildings, which have very short vibration periods, γ_{ls} takes on values close to 1, because the inelastic displacement of structures with short vibration periods is very sensitive to increments in the seismic intensity. However, the value of γ_{ls} increases as the first vibration period of a structure gets longer. Finally, it becomes more or less independent of the first vibration period if the first vibration period is greater than T_C , ie, beyond the point of intersection between the constant-acceleration and the constant-velocity portions of the idealized design spectrum defined in building codes (eg, Eurocode 8⁴). Recently, Dolšek et al³⁸ proposed a model that takes into account the abovementioned facts. According to their model for a structure with a fundamental period greater than T_C , the seismic intensity causing collapse is approximately 15% greater than that causing the NC limit state ($\gamma_{ls} = 1.15$), but a linear function with a value between 1 and 1.15 is assumed for the range of periods between 0 and T_C . However, the value of γ_{ls} can significantly depend on the definition of the NC limit state and also on the type of structure.

3.3 | Acceptable median spectral acceleration causing collapse of a structure $S_{e,C,a}$

$S_{e,C,a}$ is an input parameter for the estimation of $S_{e,D,a}$ (Equation 9 or Equation 10). It mainly depends on the seismic hazard at the site of a structure and on the target collapse risk $P_{C,a}$. It is partly dependent on the performance of the

FIGURE 2 Seismic hazard function for Ljubljana and the approximated linear hazard function in the logarithmic domain, which is constrained to return periods of 475 and 10 000 years [Colour figure can be viewed at wileyonlinelibrary.com]

structure as it is necessary to assume a value of the standard deviation $\beta_{Se,C}$ in order to estimate $S_{e,C,a}$. However, several studies have already addressed $\beta_{Se,C}$, as is discussed in the following subsection. Thus, this assumption is not critical in the estimation of $S_{e,D,a}$.

Additional insight into the estimation of $S_{e,C,a}$ can be obtained by solving Equation 3 analytically. Such an approach can be convenient if the seismic hazard function is not known in detail. There are several available solutions of Equation 3 in closed form (eg, Cornell,⁵¹ Vamvatsikos,³² Kumar and Gardoni,⁵² Lazar and Dolšek⁵³). In the simplest case, when it is assumed that the hazard function $H(S_e)$ is linear in the log-log domain:

$$H(S_e) = k_0 S_e^{-k} \quad (16)$$

where k is the slope of the hazard function in the log-log domain, and k_0 is the annual rate of exceedance of $S_e = 1$ g (see Figure 2), and the collapse intensity is log-normally distributed, Equation 3 can be expressed as follows (eg, Cornell,⁵¹ McGuire⁵⁴):

$$P_{C,a} = k_0 \cdot S_{e,C,a}^{-k} \cdot e^{\frac{k^2 \cdot \beta_{Se,C}^2}{2}} \quad (17)$$

whereas $S_{e,C,a}$ can then be expressed from Equation 17:

$$S_{e,C,a} = \left(\frac{k_0}{P_{C,a}} \right)^{\frac{1}{k}} e^{\frac{k \cdot \beta_{Se,C}^2}{2}}. \quad (18)$$

It should be emphasized that $S_{e,C,a}$ is the risk-targeted median value of the spectral accelerations causing collapse of a structure and actually represents a measure of the target capacity of a structure if the capacity is expressed in terms of seismic intensity, ie, the spectral acceleration at the first vibration period of a structure. From Equation 18, it can be seen that $S_{e,C,a}$ increases when $P_{C,a}$ decreases, or when $\beta_{Se,C}$ or k_0 increases. However, the impact of k on $S_{e,C,a}$ cannot be uniformly described, as it depends on the values of the other parameters of Equation 18.

3.4 | Standard deviation $\beta_{Se,C}$

The value of the standard deviation of the collapse intensity $\beta_{Se,C}$ has to be assumed in order to calculate the acceptable median spectral acceleration causing collapse $S_{e,C,a}$. The record-to-record and modelling uncertainties are the two sources of uncertainty that mainly affect the value of $\beta_{Se,C}$. In general, other sources of uncertainty should also be considered in the evaluation of $\beta_{Se,C}$.²¹ Several studies (eg, Haselton and Deierlein,⁵⁵ Lazar and Dolšek,⁵³ Kosič et al,³⁷ Cattari et al⁵⁶) have shown that $\beta_{Se,C}$ depends on the type of intensity measure. However, because the formulation introduced in Section 2 is based on spectral acceleration at the first vibration period, β_{imC} has to be obtained on the basis of this intensity measure. In the previous study,³⁸ period-dependent models were proposed for the estimation of standard deviations when the intensity measures correspond to the peak ground acceleration and the spectral acceleration at the first vibration period. In the case of the spectral acceleration at the first vibration period, the value of standard deviation $\beta_{Se,C}$ for periods greater than T_C was considered equal to 0.40. However, a linear function between 0.20 and 0.40 was assumed for the range of periods between 0 and T_C .

3.5 | Risk-targeted safety factor γ_{im}

The risk-targeted safety factor γ_{im} (Equation 12), which was introduced in Dolšek et al.,³⁸ is used for the modification of the available reduction factor r_{NC} in the case when the seismic hazard is defined by uniform hazard maps corresponding to a designated return period T_R . Recently, a simplified model for γ_{im} has been developed for Slovenia³⁸ based on the numerical implementation of Equation 12. It was shown that when setting the return period of seismic design action to $T_R = 475$ years and $P_{C,a}$ to $1 \cdot 10^{-4}$ per year (0.5%/50 years), γ_{im} is around 2.5 and 5, respectively, for short-period and long-period structures. γ_{im} is used not only for the determination of risk-targeted reduction factors for specific target reliabilities q_a , but it can also be used, for example, as a decision model for the verification of the collapse of buildings.³⁸

γ_{im} can also be expressed in closed form. From Equation 16, spectral acceleration $S_{e,TR}$ that corresponds to return period T_R can be derived as follows:

$$S_{e,TR} = (k_0 T_R)^{\frac{1}{k}}. \quad (19)$$

The risk-targeted safety factor for limit-state seismic intensity γ_{im} can then be calculated according to Equation 12 by considering Equations 18 and 19 and that $S_{e,NC,a} = S_{e,C,a}/\gamma_{ls}$:

$$\gamma_{im} = \frac{1}{\gamma_{ls}} (T_R \cdot P_{C,a})^{-\frac{1}{k}} \cdot e^{\frac{k\beta_{Se,C}^2}{2}}. \quad (20)$$

Equation 20 was introduced by Žižmond.⁵⁷ In the derivation, it was considered that the seismic design action in the case of conventional earthquake-resistant design is usually associated with the designated return period T_R , while the acceptable performance in the risk studies is usually measured by the acceptable annual probability of collapse $P_{C,a}$. Obviously, T_R and/or $P_{C,a}$ from Equation 20 can be transformed into other appropriate measures, depending on the interest of the user. For example, Fajfar³ preferred to express $P_{C,a}$ with the acceptable return period of collapse $T_{C,a}$ ($T_{C,a} = 1/P_{C,a}$), which results in the following expression:

$$\gamma_{im} = \frac{1}{\gamma_{ls}} \left(\frac{T_{C,a}}{T_R} \right)^{\frac{1}{k}} \cdot e^{\frac{k\beta_{Se,C}^2}{2}}. \quad (21)$$

The closed-form solution for the determination of γ_{im} (Equation 20) can be used to analyze the influence of different parameters (ie, the return period of the design elastic spectral acceleration T_R , target collapse risk $P_{C,a}$, the slope of hazard function k , standard deviation $\beta_{Se,C}$, and limit-state reduction factor γ_{ls}) on the value of γ_{im} . For example, from Equation 20, it can be clearly seen that in the case when the return period of the design elastic spectral acceleration T_R increases, γ_{im} decreases, or that it increases in the case when the target collapse risk $P_{C,a}$ decreases. However, the use of a closed-form expression for the determination of γ_{im} is always subject to uncertainty due to the linearization of the hazard function. Therefore, the slope of the hazard function should be assessed carefully.

4 | DETERMINATION OF RISK-TARGETED DESIGN SEISMIC ACTION

For a conventional force-based design using response spectrum analysis, the determination of the value of the risk-targeted design spectral acceleration $S_{e,D,a}$ is not sufficient, because it represents only one point of the design spectrum. Thus, for force-based design, the design spectral accelerations for a wide range of periods are needed in order to capture the higher mode effects. There are different possibilities for the definition of design spectra based on $S_{e,D,a}$. However, in the simplest case, the design spectrum can be determined by normalizing the assumed shape of the spectrum (eg, Eurocode 8 elastic spectrum,⁴ design spectrum proposed by Calvi et al.⁵⁸) to the risk-targeted design spectral acceleration $S_{e,D,a}$. In this way, the entire risk-targeted design spectrum is defined, which can be used for force-based design in exactly the same way as, for example, the design spectrum in Eurocode 8.⁴

The step-by-step procedure for the determination of the risk-targeted design seismic action using the direct or indirect formulation is presented in Figure 3. In both cases, the target collapse risk, ie, the acceptable annual probability of collapse (Step 1) and the shape of the acceleration response spectrum (Step 6) have to be defined. Other steps are taken depending on the selected formulation of the risk-targeted design spectral acceleration. The aim of the second step is to identify the seismic hazard at the site in terms of the spectral acceleration at the first period of the structure. In the third

FIGURE 3 The step-by-step procedure for the determination of the risk-targeted design seismic action using direct and indirect formulations

step, the risk-targeted fragility function is obtained by assuming $\beta_{Se,C}$ and calculating the median spectral acceleration causing the NC limit state $S_{e,NC,a}$, which is done in the case of direct formulation, whereas in the case of indirect formulation the risk-targeted safety factor γ_{im} has to be assessed. Several different implementations, which depend on the scope of the available data, are possible for the calculation of γ_{im} . In the fourth step, the available reduction factor r_{NC} has to be determined in the case of the direct or indirect formulation. In the case of indirect formulation, r_{NC} is used for the determination of the risk-targeted reduction factor q_a . In the fifth step, the risk-targeted design spectral acceleration $S_{e,D,a}$ is calculated. Finally, the seismic design action is defined by defining the shape of the design spectrum and assigning it to $S_{e,D,a}$ (Step 6).

At this point, it makes sense to emphasize that the proposed formulations of the risk-targeted design intensity were derived by using a spectral acceleration corresponding to the first vibration period $S_e(T_1)$ as an intensity measure (ie, Equations 9 and 10). Such an approach simplifies the derivation because the spectral acceleration corresponding to the first vibration period is proportional to the base shear. Thus, the shape of the design response spectra which is adopted in the force-based design does not bias the estimated base shear, at least not for the most important component that corresponds to the first vibration mode. If any other intensity measure is used to formulate the risk-targeted design intensity, an additional transformation factor needs to be introduced in order to account for the difference between the

risk-targeted spectral acceleration at the first vibration period and the corresponding spectral acceleration from the design response spectrum, which would be constrained to any other type of risk-targeted intensity measure. A detailed explanation of this is outside of the scope of this paper, but is available elsewhere (eg, Žižmond⁵⁷).

5 | EXAMPLE: ESTIMATION OF A RISK-TARGETED SEISMIC ACTION, FORCE-BASED DESIGN, AND RISK ASSESSMENT OF A SIX-STOREY RC BUILDING

The direct and indirect formulations for the estimation of the risk-targeted design spectral acceleration $S_{e,D,a}$ were applied to an example of a six-storey RC building. The acceleration response spectrum according to Eurocode 8 was then normalized to $S_{e,D,a}$, and the structure was designed according to the force-based design procedure prescribed in Eurocode 8. Next, the design was validated by means of nonlinear static and dynamic analyses.

5.1 | Description of the investigated building and the seismic hazard

The investigated building is a six-storey reinforced concrete frame (Figure 4A, see also Lazar et al⁵⁹). The building consists of four bays in the X direction and six bays in the Y direction. The cross section of the columns is 55/60 cm, whereas the slab thickness is 22 cm. Reinforcing steel B500B and concrete C35/45 were adopted in the design. The total mass of the structure amounts to 5193 t. The first vibration periods in the X and Y directions are practically equal and amount to 1.00 second. The building is located in Ljubljana, Slovenia, on soil type B.

The seismic hazard function for the spectral acceleration corresponding to the first vibration period of the structure $S_e(T_1)$, the location, and the soil type of the structure (Figure 2) was calculated based on the official probabilistic seismic hazard analysis for Slovenia.⁶⁰ The spectral acceleration corresponding to the return periods $T_R = 475$ and $T_R = 10\,000$ years, which were then used to calculate the risk-targeted design seismic action according to the indirect formulation, were computed directly from the hazard function. Their values are equal to $S_{e,475} = 0.35$ g and $S_{e,10000} = 1.41$ g, respectively. In addition, the spectral acceleration corresponding to the return period $T_R = 2475$ years ($S_{e,2475} = 0.79$ g) was calculated because the ground motions for the collapse safety check were selected based on this spectral acceleration.

As has already been mentioned, the indirect formulation of $S_{e,D,a}$ is intended to be used if the code prescribes that the seismic action is defined on the basis of a designated return period of occurrence (eg, 475 years). This formulation is also convenient if the entire hazard function is not available and the hazard at the site is given only on the basis of seismic hazard maps for certain return periods. However, in order to compare the results obtained by using both approaches for the calculation of the risk-targeted design seismic action, it was decided to calculate the spectral accelerations directly from the hazard function eliminating in this way the possible error due to the difference in the seismic hazard input.

FIGURE 4 A, Elevation and plan views of the investigated six-storey building and B, the force-displacement relationship of the SDOF model [Colour figure can be viewed at wileyonlinelibrary.com]

5.2 | Estimation of the risk-targeted design spectral acceleration according to the direct formulation

The risk-targeted design action according to direct formulation was estimated by performing the six steps presented in Figure 3. Each step is briefly explained below.

Step 1: The target collapse risk was set to $P_{C,a} = 10^{-4}$ (0.5% in 50 years). This risk is two times smaller than that prescribed in the USA ($2 \cdot 10^{-4}$) (eg, FEMA P-695²¹), but around 10 times greater than that estimated on the basis of a survey about the tolerable probabilities of collapse for ordinary structures in Slovenia.⁶¹

Step 2: The seismic hazard function for Ljubljana, ie, the spectral acceleration corresponding to the first vibration period of the structure $S_e(T_1 = 1 \text{ second})$, soil type B (Figure 2) was obtained from the official probabilistic seismic hazard analysis for Slovenia.⁶⁰

Step 3: $S_{e,NC,a}$ was calculated as the ratio between $S_{e,C,a}$ and the limit-state reduction factor $\gamma_{ls} = 1.15$, which was adopted from the previous study by Dolšek et al.³⁸ $S_{e,C,a}$ was assessed iteratively by solving Equation 3, taking into account the target collapse risk, the hazard function defined in the previous step, and by assuming a log-normal distribution function for the collapse fragility function. The standard deviation was assumed to be equal to $\beta_{Se,C} = 0.40$.³⁸ For this particular example, $S_{e,C,a}$ was estimated to amount to 1.69 g. $S_{e,NC,a}$ was then calculated by using (Equation 7):

$$S_{e,NC,a} = \frac{S_{e,C,a}}{\gamma_{ls}} = \frac{1.69 \text{ g}}{1.15} = 1.47 \text{ g}. \quad (22)$$

Step 4: The results of the assessments of structures in previous studies show that typical values of the overstrength reduction factor r_s of multistorey reinforced concrete frame buildings designed according to Eurocodes 2 and 8 vary from around 2 to 3, whereas the values of ductility of the structure associated with the NC limit state μ_{NC} vary from around 5 to 8.^{39,59} Based on these boundary values, the overstrength reduction factor and ductility were estimated as $r_s = 2$ and $\mu_{NC} = 6$, respectively. Note, that the ductility of the structure can also be estimated by the approach considered in the direct displacement-based design.⁶² In addition, it should be noted that the assumed values of the overstrength and ductility factors should be based on the best estimates of their values without consideration of any safety factors.

Ductility μ_{NC} affects the ductility reduction factor associated with the NC limit state $r_{\mu_{NC}}$. Different approaches can be used to calculate $r_{\mu_{NC}}$ on the basis of known ductility μ_{NC} . As the period of the investigated building is greater than T_C , the value of the ductility reduction factor $r_{\mu_{NC}}$ can be assumed to be equal to μ_{NC} (ie, equal displacement rule). However, in this example, the ductility reduction factor $r_{\mu_{NC}}$ was determined as the ratio between ductility μ_{NC} and the inelastic displacement ratio C_I , which was estimated on the basis of the seismic response of the SDOF model (see the Appendix). The period and mass of the SDOF model were considered equal to the first period of the structure ($T_1 = 1 \text{ second}$) and the equivalent mass of the SDOF system, respectively. The ductility of the SDOF model corresponded to the assumed ductility μ_{NC} , whereas the yield strength of the SDOF model was defined in such a way that the intensity causing the NC limit state (displacement) of the SDOF model was in the order of magnitude of $S_{e,NC,a}$ (Figure 4B).

Nonlinear dynamic analyses have to be performed in order to calculate C_I . A hazard-consistent set of ground motions (Figure 5A) was selected using the conditional spectrum approach.⁶³ This provides an unbiased estimation of the seismic demand. The conditional spectrum for the selection of ground motions was defined based on the mean magnitude ($M = 6.2$) and the mean distance ($R = 7.3 \text{ km}$), which were obtained from the results of seismic hazard disaggregation, the spectral acceleration corresponding to the return period of 2475 years and the first vibration period of the structure ($S_{e,2475} = 0.79 \text{ g}$), and a Sabetta & Pugliese attenuation relationship.⁶⁴ All of the 30 ground motions (Figure 5A) corresponded to events with magnitudes between 4.5 and 7, and source-to-site distances between 5 and 50 km, and were recorded on soil having a shear-wave velocity in the upper 30 m between 380 and 800 m/s.

The inelastic deformation ratio C_I is defined as the ratio between the displacement at the NC limit state of the inelastic SDOF model (D_{nNC}^*) and the displacement of the linear elastic SDOF model when subjected to $S_{e,NC}$ (D_{eNC}^*), where $S_{e,NC}$ is the median spectral acceleration causing the NC limit state of the inelastic SDOF model. Thus, $S_{e,NC}$ was first calculated as the median value of the ground-motion intensities causing the NC limit state, which were

FIGURE 5 A, The target median and 16th and 84th percentile spectra for ground-motion selection obtained by the conditional spectrum approach for an earthquake scenario based on $S_{e,2475}$ and B, IDA curves using the inelastic and elastic SDOF models

obtained from incremental dynamic analysis⁴⁷ using an inelastic SDOF model and the hazard-consistent set of ground motions. The displacement of the elastic model was then calculated based on one of the fundamental equations of structural dynamics (ie, $S_{e,NC} = \omega^2 \cdot D_{eNC}^*$, where ω is the circular frequency). It was observed that the displacement of the elastic model ($D_{eNC}^* = 44.7$ cm) was slightly higher than the inelastic displacement ($D_{mNC}^* = 35.0$ cm) (Figure 5B). Consequently, the inelastic deformation ratio C_I was smaller than 1 ($C_I = 0.78$). This phenomenon is a consequence of the use of the conditional spectrum approach for the selection of hazard-consistent ground motions. Once the inelastic deformation ratio C_I was assessed, the ductility reduction factor was simply obtained as $r_{\mu NC} = \mu_{NC}/C_I = 6.0/0.78 = 7.66$.

Step 5: The risk-targeted design spectral acceleration $S_{e,D,a}$ was calculated as the ratio between $S_{e,NC,a}$ and the product of the overstrength factor and ductility reduction factor (Equation 9):

$$S_{e,D,a} = \frac{S_{e,NC,a}}{r_{NC}} = \frac{S_{e,NC,a}}{r_s \cdot r_{\mu NC}} = \frac{1.47g}{2 \cdot 7.66} = 0.10g. \quad (23)$$

Step 6: The design spectrum was then obtained by normalizing the Eurocode 8 elastic spectrum to $S_{e,D,a}$ (Figure 6).

5.3 | Estimation of the risk-targeted design spectral acceleration according to the indirect formulation

$S_{e,D,a}$ was estimated according to the indirect formulation by performing the six steps presented in Figure 3. Each step is briefly explained below.

FIGURE 6 The Eurocode 8 elastic spectrum, the risk-target design spectrum, and the risk-targeted spectral acceleration causing the near-collapse limit state of a structure $S_{e,NC,a}$

Step 1: The target collapse risk was set to be equal to that used in the calculation of the risk-targeted design seismic action according to the direct formulation (ie, $P_{C,a} = 10^{-4}$).

Step 2: A return period of 475 years was assumed as the basis for the seismic design action. The corresponding value of the spectral acceleration at the first vibration period ($T_1 = 1$ seconds), which was obtained from the hazard function, amounted to $S_{e,475} = 0.35$ g. The slope of the approximate hazard function ($k = 2.19$) (ie, a linear function in the log domain) was calculated utilizing the spectral acceleration at the first vibration periods, which corresponded to return periods of $T_R = 475$ and $T_R = 10\,000$ years. Note that the two return periods were selected based on the results of a small-scale parametric study, which was done in order to investigate how to calculate the slope of the hazard function on the basis of the uniform hazard maps. The parametric study considered the seismic hazard for Slovenia based on the official⁶⁰ and SHARE⁶⁵ seismotectonic models, the target collapse risk $P_{C,a} = 10^{-4}$, and a return period of $T_R = 475$ years. The results indicated that the values of γ_{im} , when assessed using the iterative numerical procedure and the closed-form expression (ie, Equation 20), were almost equal if k was assessed based on spectral accelerations corresponding to return periods of 475 and 10 000 years.

Step 3: The risk-targeted safety factor for limit-state seismic intensity γ_{im} was estimated using two different approaches: (a) the simplified model proposed in a previous study³⁸ and (b) on the basis of an approximate closed-form solution (Equation 20). The simplified model was developed by taking into account the design elastic spectral acceleration for a return period of 475 years and prescribes a period-dependent model of γ_{im} . According to the model proposed in the previous study,³⁸ the value of γ_{im} is equal to 5.0 for periods greater than $3T_C$, where T_C is the period at the intersection of the constant branch of acceleration and velocity of the Eurocode 8 elastic spectrum.⁴ However, a linear function between 2.5 and 5.0 was assumed for the range of periods between 0 and $3T_C$. According to this model, the value of γ_{im} is equal to 4.17, because in this particular example $T_1 = 1$ second and $T_C = 0.5$ second (see Figure 7).

For the estimation of γ_{im} according to the closed-form solution (Equation 20), the slope of hazard function k , logarithmic standard deviation $\beta_{Se,C}$, and limit-state reduction factor γ_{ls} have to be assumed. The slope of hazard function $k = 2.19$ was obtained based on the spectral acceleration for return periods of 475 and 10 000 years, whereas the assumed standard deviation ($\beta_{Se,C} = 0.40$) and the limit-state reduction factor ($\gamma_{ls} = 1.15$) are equal to that used in the previous subsection. The risk-targeted safety factor γ_{im} , which was calculated according to Equation 20, was practically the same (see Equation 24) as that obtained from the simplified model for γ_{im} .³⁸

$$\gamma_{im} = \frac{1}{\gamma_{ls}} (T_R \cdot P_{C,a})^{-\frac{1}{k}} \cdot e^{\frac{k\beta_{Se,C}^2}{2}} = \frac{1}{1.15} (475 \cdot 10^{-4})^{-\frac{1}{2.19}} \cdot e^{\frac{2.19 \cdot 0.4^2}{2}} = 4.17. \quad (24)$$

Step 4: In the first part of Step 4, the available reduction factor r_{NC} has to be assessed. Because the value of r_{NC} does not depend on the approach used for the assessment of the risk-targeted design action, the same value was used as in the previous subsection (ie, $r_{NC} = r_s \cdot r_{\mu NC} = 2.7.66 = 15.3$). For details about the assessment of r_{NC} , the reader is referred to the previous subsection. The risk-targeted reduction factor for specific target reliability (see Equation 13) was calculated by taking into account $\gamma_{im} = 4.17$, $r_s = 2$ and $r_{\mu NC} = 7.66$:

FIGURE 7 Period-dependent model for γ_{im} according to Dolšek et al.³⁸

$$q_a = \frac{r_{NC}}{\gamma_{im}} = \frac{r_s \cdot r_{\mu NC}}{\gamma_{im}} = \frac{2 \cdot 7.66}{4.17} = 3.67. \quad (25)$$

Step 5: The risk-targeted design spectral acceleration $S_{e,D,a}$ was calculated as the ratio between the spectral acceleration corresponding to a return period of 475 years, $S_{e,475}$, and the risk-targeted reduction factor q_a (Equation 10):

$$S_{e,D,a} = \frac{S_{e,475}}{q_a} = \frac{0.35g}{3.67} = 0.10g. \quad (26)$$

Step 6: The design spectrum was obtained by normalizing the Eurocode 8 elastic spectrum to $S_{e,D,a}$ (Figure 6). The resulting spectrum, which was used in the force-based design, is practically the same as that obtained by utilizing the direct formulation of $S_{e,D,a}$.

It is interesting that for this particular example the value of q_a is practically equal to the behaviour factor ($q = 3.9$), which is prescribed in Eurocode 8 for RC frames and ductility class medium. However, if the owner preferred to have a structure for which the collapse risk would be less than target collapse risk, the conventional force-based design approach cannot be applied, because the engineer has no tool to define seismic design action related to target collapse risk. In such cases, the direct or indirect formulation of the seismic design action can be used. For example, if seismic collapse risk is set to 0.25% in 50 years (ie, $P_{C,a} = 0.5 \cdot 10^{-4}$), it can be shown that the resulting q_a is equal to 2.7. This is about 30% less than q_a observed in the primary example. If the value of the target collapse risk would be set to 1% in 50 years ($P_{C,a} = 2 \cdot 10^{-4}$), then q_a would be equal to 5.0, which is about 40% higher value than q_a obtained for the primary example ($P_{C,a} = 10^{-4}$). However, this evaluation is simplistic, because it was assumed that the values of r_s , $r_{\mu NC}$, $\beta_{Se,C}$, and γ_{ls} are equal to those from the primary example where $P_{C,a} = 10^{-4}$, although they are, to some extent, dependent on the seismic design action. However, the accuracy of the risk-targeted force-based design can be improved by further research aimed at developing models for those factors that are related to the performance of a structure (eg, $\beta_{Se,C}$, γ_{ls} and $r_{NC} = r_{\mu NC} \cdot r_s$).

5.4 | Design of the structure

The structure was designed according to the force-based design approach described in Eurocode 8.⁴ The only difference was the seismic design action, which was in this case prescribed by the Eurocode elastic spectrum normalized to the risk-targeted design spectral acceleration $S_{e,D,a}$. A response spectrum analysis was performed, and the reinforcement of the structural elements was determined by the rules of Eurocode 8⁴ by taking into account the minimum requirement corresponding to ductility class medium (DCM). The damage limitation requirement was also verified according to Eurocode 8. The elements of the structure were not heavily reinforced, because the longitudinal reinforcing ratio of the columns in first two storeys is equal to 1.65%, whereas in all other columns the longitudinal reinforcing ratio is 1.14%.

5.5 | Risk assessment of the structure using the pushover-based method

In order to verify the collapse safety, the spectral acceleration causing the NC limit state of structure $S_{e,NC}$ and the collapse risk of the building were estimated by the pushover-based method. A nonlinear structural model of the building was developed taking into account the requirements of Eurocode 8-3.⁶⁶ Each structural element of the building was modelled by a linear elastic beam element with two inelastic rotational hinges (defined by a moment-rotation relationship). More details about simplified nonlinear models of frame buildings can be found elsewhere.⁶⁷ The equivalent SDOF model was obtained from the results of the pushover analysis of the building and was used to estimate the seismic demand on the basis of incremental dynamic analysis (SDOF IDA). All the analyses were performed by means of the PBEE toolbox,⁶⁷ which provides a user-friendly interface with OpenSees.⁶⁸

The pushover analyses were performed for both directions. For simplicity reasons, hereinafter only the results for the X direction are presented. The invariant force vectors corresponded to the product of the storey masses and the

FIGURE 8 A, Pushover curve, idealized pushover curve of the investigated six-storey building, and the force-displacement relationship of the SDOF model, and B, IDA curves for the inelastic and elastic SDOF models, the force-displacement relationship of the SDOF model [Colour figure can be viewed at wileyonlinelibrary.com]

fundamental vibration modes. The pushover curve and the corresponding idealized pushover curve are shown in Figure 8A. Yield strength F_Y (which corresponds to the maximum strength) of the idealized system was 9852 kN. Yield displacement D_Y and the NC displacement D_{NC} were equal to 6.9 and 47.8 cm, respectively. Consequently, the available system ductility associated with the NC limit state of structure μ_{NC} , which was obtained as the ratio between the displacement corresponding to the NC limit state and the one causing yielding, was equal to $\mu_{NC} = D_{NC}/D_Y = 47.8/6.9 = 7.0$. Note that the NC limit state was defined at the displacement corresponding to the occurrence of the NC limit state of the first column. Overstrength reduction factor r_s , which is defined as a ratio between yield base shear F_Y and risk-targeted design base shear $F_{D,a}$, was also assessed. Its value is equal to $r_s = 2.3$.

The force-displacement relationship of the SDOF model was obtained by idealizing the pushover curve (Figure 8A) and by dividing the idealized pushover curve by means of a transformation factor Γ_I , which amounted to 1.26. The 5% mass proportional Rayleigh damping model was considered in the nonlinear time-history analysis of the equivalent SDOF. The nonlinear behaviour was modelled with the peak-oriented hysteretic model, ie, uniaxial material »Hysteretic«, which is implemented in OpenSees. The SDOF IDA was then performed by utilizing the set of ground motions used for the determination of $r_{\mu NC}$ (see Section 5.2).

The IDA curves for each considered ground motion and median IDA curve are presented in Figure 8B. It can be observed that the median value of spectral acceleration causing NC limit state $S_{e,NC}$ is equal to 2.04 g, which is higher than the acceptable value of spectral acceleration causing NC limit state (ie, $S_{e,NC,a} = 1.46$ g). As a consequence, the collapse risk of structure ($P_C = 0.4 \cdot 10^{-4}$), which was calculated on the basis of Equation 2 by utilizing the hazard curve, the value of $\beta_{S_{e,C}}$ and γ_{ls} as defined in Section 5.2, is also smaller than $P_{C,a}$. It can, therefore, be concluded that the collapse risk of the investigated structure is smaller than the target collapse risk.

5.6 | Risk assessment of the structure using nonlinear dynamic analysis

The spectral acceleration causing the NC limit state $S_{e,NC}$ and the collapse risk were also estimated by performing incremental dynamic analysis for the nonlinear model of the entire structure. In this case, the nonlinear structural model used in the pushover analysis was extended. Damping was considered proportional to the mass matrix (5% critical damping). Thus, the estimated median spectral acceleration causing collapse may be overestimated to some extent, because the mass proportional damping probably overestimates the effect of damping in a nonlinear range, even if the damping is modelled only proportional to masses concentrated at the storey levels. However, the damping in the nonlinear range is not yet fully understood.⁶⁹ Peak-oriented hysteretic rules were adopted for nonlinear plastic hinges (ie, the uniaxial material »Hysteretic«, in OpenSees⁶⁸). Parameter β that controls the unloading stiffness was assumed equal to 0.8, as this value was calibrated in the previous study⁶⁷ based on the results of pseudo-dynamic tests. The IDA was then performed by utilizing the set of ground motions, which were used for the determination of $r_{\mu NC}$ (see Section 5.2). For simplicity reasons, the effect of modelling uncertainties was neglected.

FIGURE 9 The IDA curves of the investigated six-storey building on the basis of the nonlinear model of the entire structure

The results of the IDA analysis are presented in Figure 9. It can be seen that the median value of the spectral acceleration causing NC limit state ($S_{e,NC} = 2.42$ g) is significantly higher than the value of $S_{e,NC}$ assessed on the basis of the SDOF IDA (ie, $S_{e,NC} = 2.06$ g) and the acceptable value of the spectral acceleration causing the NC limit state (ie, $S_{e,NC,a} = 1.46$ g). As a consequence, the collapse risk of the structure ($P_C = 0.3 \cdot 10^{-4}$) is even smaller than that estimated by using the pushover-based method.

The evaluated collapse risk is about 3 times (or 2.5 times in the case of the pushover-based method) smaller than the target collapse risk. Although the accuracy of the evaluated collapse risk can be discussed further (eg, to some extent, it is overestimated due to the use of the mass proportional damping), it can be concluded that the design is most probably conservative. For this particular example, the result is a consequence of conservatism considered in the assessment of the risk-targeted design spectral acceleration, because the assumed values of both the ductility and the overstrength reduction factor were by about 15% smaller than the actual assessed values. However, the structure would be practically the same if designed according to Eurocode 8. This should be the outcome of force-based design for a vast majority of structures, because such the design, which is based on several assumptions, should provide results with a certain level of confidence. Recently, it has been realized that this might not be the case for the most hazardous sites because seismic structural reliability implied by code-conforming design generally decreases as the seismic hazard of the construction site increases (Iervolino and Dolce⁷⁰). As a consequence, the use of nonlinear analysis followed by the conventional design should be promoted, especially for the most hazardous sites. However, in general, two strategies can be developed. In the first case, as presented in this example, the nonlinear methods can be used to verify that the seismic performance of the structure is better than the minimum performance required by the code. This information should have some impact on the market price as well as on the regulator, who should establish an environment where the owners that invest in safety and enhancing the community resilience are given some benefits in comparison to those that fulfill only minimum requirements of the codes. In the second case, the nonlinear analysis can be used to optimize the design. In this process, the force-based design using the proposed approach can provide a very good initial structural configuration. However, in these cases, it is reasonable to assume that the unknown parameters are assumed without any conservatism, as it is easier to define guidelines on how to increase the strength and deformation capacity of the structure (eg, Lazar Sinković et al¹⁸) rather than how to decrease those parameters of the structure. It has been shown before¹⁸ that by using simple guidelines for adjusting the structure, the risk-based performance objective could be fulfilled by a reasonable utilization rate when performing only a few iterations. The motivation for such an optimization is to reduce construction costs. On the other hand, if only the force-based approach is used in the design, then the unknown parameters should be assumed with a certain level of confidence, which means that the resulting collapse risk will probably be lower than the target collapse risk.

6 | CONCLUSIONS

The proposed formulation of the risk-targeted design seismic action provides a physics-based insight into the relationship between the acceptable collapse risk, the seismic hazard at the site of a structure, the ability of a structure to deform in the nonlinear range and dissipate energy (ie, the ductility reduction factor), the overstrength reduction factor, and the uncertainty in the seismic response of structures. As such, it can represent a straightforward extension of conventional seismic design to risk-targeted design. It can also be used as a tool for engineering practitioners to communicate the minimal target risk set by a regulator (eg, a target collapse risk of 1% in 50 years) to stakeholders, and to design a structure for a stricter criterion of target risk if required to do so by the stakeholders for different reasons.

It is shown how different formulations can be used for the estimation of the risk-targeted seismic action. The direct formulation can be used if full insight into the seismic hazard at the site is available. In the case when the seismic hazard is defined by the uniform hazard maps associated with a designated return period, it is more convenient to use the indirect formulation of the risk-targeted seismic action, which involves a risk-targeted safety factor.³⁸ It is shown how such a factor can be applied to the reduction factor in order to account for the target risk. Several methods can be applied in order to evaluate the risk-targeted reduction factor. In a specific case, the risk-targeted safety factor and consequently the risk-targeted reduction factor can be expressed in closed form, which provides an insight into all the key parameters that affect the risk-targeted design spectral acceleration.

The primary source of uncertainty in the definition of a risk-targeted seismic action for the force-based design of structures originates from the incompatibility between the performance objective (eg, collapse prevention), which is related to extremely nonlinear structural behaviour, and the engineering demand parameters, which are in the case of force-based design determined by linear elastic analysis. As a consequence, several assumptions have to be made in order to estimate the risk-targeted seismic action for the force-based design. The paper shows that it is necessary to assume dispersion of the intensities causing collapse of a structure, limit-state reduction factor, and a conventional reduction factor, which can be decomposed to an overstrength reduction factor and a ductility reduction factor. Thus, the accuracy of the force-based design mainly depends on these assumptions. It is essential for engineering practitioners to have a good insight into overstrength and ductility reduction factors, because, as shown in the example, inaccurate assumptions of reduction factors can cause significant deviations from the target performance of the structure.

The proposed formulations can also be used to set up a framework for the parametric studies aimed at developing the models of the parameters that are primarily structure dependent and to some extent also site dependent ($\beta_{Se,C}$, γ_{ls} and $r_{NC} = r_{\mu NC} \times r_s$). Insight into these parameters can significantly improve the accuracy of the force-based design in the future, while the effects of site-specific seismic hazard can still be independently incorporated into the evaluation of the seismic design action. However, from a more general point of view, the proposed procedure can always be used to improve the determination of initial structural configuration, which is needed when the design is based on the use of nonlinear analysis. Thus, the results of the paper are complementary to those presented in previous papers that addressed the risk-based design (eg, Lazar Sinković et al¹⁸) and risk-targeted design,^{38,71} either by using pushover or nonlinear response history analysis. All these papers together form a more general framework for the risk-based or risk-targeted design of structures using linear elastic, pushover or nonlinear response history analysis. The framework represents a palette of methods for the seismic design of structures that account for the different levels of detail and the different levels of reliability of results. With additional collaborative research, these methods can be in the future developed to such an extent that they can be introduced in the codes, which will allow differentiating the level of detail and the level of reliability of the results with respect to the hazard level and the consequence class of the structure.

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APPENDIX A

Relationship between risk-targeted spectral accelerations, ductility reduction factor, and overstrength reduction factor

The aim of this Appendix is to show that the available reduction factor r_{NC} is equal to the product of overstrength reduction factor r_s , and ductility reduction factor $r_{\mu NC}$ and that the relationship with spectral acceleration can be formulated with displacement and forces as presented in Equation 15. However, the derivation is limited to the SDOF model. It is also demonstrated that ductility reduction factor $r_{\mu NC}$ and overstrength reduction factor r_s can be formulated in the same manner as in the case of a deterministic approach.²⁵

In order to decompose structural reduction factor r_{NC} into the product of a ductility reduction factor and an overstrength reduction factor, the spectral accelerations corresponding to the first vibration period of the equivalent SDOF model ($S_{e,NC}$, $S_{e,D,a}$) have to be related to the forces and displacements. This can be done at the level of an SDOF model. In the main body of the paper, it has already been established that the available reduction factor can be defined with spectral accelerations (Equation 8):

$$r_{NC} = \frac{S_{e,NC}}{S_{e,D,a}}. \quad (A1)$$

For further derivations, two types of SDOF models are introduced: a (linear) elastic (e) and an inelastic (n) SDOF model, which have the same initial stiffness and period (Figure A1). Furthermore, let us define spectral acceleration $S_{e,NC}$ as the median spectral acceleration of ground motions that cause the NC displacement D_{nNC}^* of the inelastic SDOF model (Figure A1a). System ductility μ_{NC} of the inelastic SDOF model is defined as follows:

$$\mu_{NC} = \frac{D_{nNC}^*}{D_Y^*} \quad (A2)$$

where D_Y^* is the displacement associated with the yielding of the model (Figure A1).

Equation A1 is now decomposed into two factors by using the median spectral acceleration of the ground motions that cause yielding of inelastic system $S_{e,Y}$:

$$r_{NC} = \frac{S_{e,NC}}{S_{e,Y}} \cdot \frac{S_{e,Y}}{S_{e,D,a}} = r_{\mu NC} \cdot r_s. \quad (A3)$$

From Equation A3, it can be seen that the available reduction factor has been decomposed into a ductility reduction factor $r_{\mu NC}$ and an overstrength reduction factor r_s , which are defined by three characteristic values of the spectral accelerations ($S_{e,NC}$, $S_{e,Y}$, and $S_{e,D,a}$). In Equation A3, factors $r_{\mu NC}$ and r_s are expressed in terms of seismic intensities. In order to express both reduction factors in terms of the system's strength and ductility, as in the traditional formulation of the reduction factor (eg, Fischinger and Fajfar,²⁵ Uang²⁷), the relationship between the spectral displacement and the spectral acceleration of the elastic model needs to be taken into account. From theory, it follows that spectral acceleration $S_{e,NC}$ is equal to the product of the square of the circular frequency times the spectral displacement corresponding to the NC limit state D_{eNC}^* of the elastic SDOF model:

FIGURE A1 A, Acceleration spectrum in acceleration-displacement format and acceleration-displacement relationship of the elastic and inelastic SDOF models, and B, force-displacement relationship for the elastic and inelastic SDOF models [Colour figure can be viewed at wileyonlinelibrary.com]

$$S_{e,NC} = \omega^2 \cdot D_{eNC}^* \quad (A4)$$

The spectral acceleration at the yielding of the elastic or inelastic model can be expressed by analogy to Equation A4, because it is assumed that the response of the inelastic model is elastic until displacement D_Y^* is reached:

$$S_{e,Y} = \omega^2 \cdot D_Y^* \quad (A5)$$

Ductility reduction factor $r_{\mu NC}$ from Equation A3 can now be expressed by displacements of the elastic SDOF model:

$$r_{\mu NC} = \frac{D_{eNC}^*}{D_Y^*} \quad (A6)$$

Because the structure is inelastic, it is necessary to express D_{eNC}^* from Equation A6 with the displacements of the inelastic SDOF model. This can be achieved by introducing the models of inelastic deformation ratio C_1 (eg, Dolšek and Fajfar,⁴¹ Miranda,⁴² FEMA 273⁴³), which is defined as follows:

$$C_1 = \frac{D_{nNC}^*}{D_{eNC}^*} \quad (A7)$$

where D_{nNC}^* and D_{eNC}^* are the displacements of inelastic and elastic SDOF model, respectively, corresponding to $S_{e,NC}$. However, D_{nNC}^* can be expressed as a product of the available system ductility μ_{NC} , which has to be ensured in the design, and the yield displacement of the inelastic SDOF model:

$$D_{nNC}^* = \mu_{NC} \cdot D_Y^* \quad (A8)$$

By inserting Equation A8 into Equation A7, displacement D_{eNC}^* can be expressed as follows:

$$D_{eNC}^* = \frac{\mu_{NC}}{C_1} \cdot D_Y^* \quad (A9)$$

By inserting Equation A9 into Equation A6, ductility reduction factor $r_{\mu NC}$ can now be expressed by available system ductility μ_{NC} divided by inelastic deformation ratio C_1 (eg, Fajfar⁴⁵):

$$r_{\mu NC} = \frac{\mu_{NC}}{C_1} \quad (A10)$$

Overstrength reduction factor r_s is the other factor occurring in Equation A3. It can be expressed by forces rather than by the accelerations:

$$r_s = \frac{F_Y^*}{F_{D,a}^*} \quad (\text{A11})$$

where it is considered that the yield spectral acceleration $S_{e,Y}$, which causes a yield force of structure F_Y^* , and spectral acceleration $S_{e,D,a}$ associated with design force $F_{D,a}^*$ of the SDOF model are defined by the expressions:

$$S_{e,Y} = \frac{F_Y^*}{m^*} \quad (\text{A12})$$

$$S_{e,D,a} = \frac{F_{D,a}^*}{m^*} \quad (\text{A13})$$

where m^* is the mass of the SDOF model. Equation 15, when it is applied to the SDOF model, can then be obtained by inserting Equations A10 and A11 into Equation A3:

$$r_{NC} = \frac{\mu_{NC}}{C_1} \cdot \frac{F_Y^*}{F_{D,a}^*}. \quad (\text{A14})$$