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Reliability-based partial factors for seismic design and assessment consistent with second-generation Eurocode 8

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Abstract

Second-generation Eurocode 8 introduces displacement-based design alongside the traditional force-based one with reduced equivalent forces; it provides resistance models needed to apply the approach to reinforced concrete, steel and composite frame or wall-frame structures: chord rotation for members, and shear strength for members and joints; finally, it declares a target reliability level for near collapse. This paper discusses the formulation of partial factors to achieve a target reliability when using the displacement-based approach. The factors are derived following a traditional, time-invariant way of performing code-calibration for other non-seismic design situations, starting from the probability distributions for the seismic action effect and the resistance. It is shown that the problem can be simplified to one with two Lognormal variables, a uniform reliability can be achieved over the calibration space when both a load-side and a resistance-side factor are used, and constant sensitivity coefficients can be used, with values different from the original König and Hosser ones and reflecting the larger weight of the action-related uncertainty in the seismic case. The problem with this otherwise good result is that a partial factor on the load side must be used and, further, that its value is site dependent. This approach is not in line with current European practice, thus a corrected site-independent resistance-side partial factor is proposed, which is compatible with the framework of Eurocode 8. As a further advantage, the proposed format coincides with that proposed in the second-generation EN1990/EN1992 for the assessment of existing structures under non-seismic design situations.

KEYWORDS
brittle failure, ductile failure, mean annual frequency, reliability index, seismic assessment

1 | INTRODUCTION

Modern codes adopt limit-state design with formats that, with some differences, share the partial-factors approach. For the chosen set of factors, or code-format, reliability-based code calibration has the goal of providing constant values that

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- A format for partial factors for seismic design and assessment is derived according to classical reliability theory results.
- It is shown how using two factors, one on resistance, as it is customary, and one on the action effect side, usually missing, the resulting reliability is close to the target one over all considered seismicity levels.
- The results are compared with a prexisting formulation, also using two partial factors.
- A single corrected resistance-side factor is proposed, to comply with the current code.
- Its use leads also to reliability close to the target over all seismicity levels, provided the return period of the seismic action is not independently chosen but determined as proportional to target reliability.
- This format has also the advantage of coinciding formally with that proposed for non-seismic assessment cases.

minimize the discrepancy between achieved and target safety across a range of design situations. The target safety is set out in probabilistic terms, as a target maximum probability that a given undesirable condition occurs over the reference period.

In principle, all actions, structural typologies, and materials should be calibrated in a consistent manner. This ideal situation, however, does not always coincide with reality. In 1980, the US National Bureau of Standards (NBS) published its report A58,[1] according to Melchers[2], "probably the most influential document in practical and illustrative terms," for reliability-based code calibration. This report represents a milestone exactly in virtue of its comprehensiveness. The considered range of actions included, notably, not just the usual dead and live loads plus climatic actions, such as wind and snow, but also the seismic action; further, it covered both "light" and "heavy" structures, that is, steel and timber as well as concrete and masonry (it only left out bridges and, consequently, traffic loads). What is most relevant for the discussion to follow is that the report used a demand-side partial factor also for the seismic action effect. On the contrary, no "centralized" calibration encompassing seismic and non-seismic design situations was carried out in Europe for the first-generation Eurocodes. While the other Eurocodes either focused on the safety format (EN1990[3]), the actions (all but the seismic one, EN1991[4]), or a specific material, like concrete (EN1992[5]) or steel (EN1993[6]), Eurocode 8 (EN1998[7]) spanned over safety, the seismic action and all materials, holistically dealing with the seismic design situation and even providing with its Part 3[8] the only portion of the entire set of Eurocodes devoted to assessment and retrofit. In terms of safety format, the *fib* Model Code[9] and the report by König and Hosser[10] were the main references for non-seismic design situations. Eurocode 8, instead, took another path, partly justified by the different expected performance of structures subjected to strong ground motion, and the associated design philosophy exploiting energy dissipation through cyclic inelastic deformation. While gravity affected equally all countries, a distinct separate seismic community existed made of experts from countries around the world, that evolved their concepts and protection strategies, advancing the use of nonlinear analysis[11,12] and identifying the need to move from forces to displacements.[13] Notably, Eurocode 8[7] did not declare a target reliability, like it is done for the other design situations.

This paper tries to reconcile the two points of view, by calibrating partial factors for the seismic design situation consistently with what is done for other actions. The procedure, inevitably, confirms the necessity of a site-dependent load-side partial factor to minimize the discrepancy between actual and target safety over the given calibration space. Since this is not compatible with the current format of Eurocode 8, a corrected resistance-side partial factor is proposed, shown to be sufficiently site-independent under certain conditions. As a further advantage, the partial factor format coincides with that proposed in the other Eurocodes[14] for the assessment of existing structures under gravity loads, thus providing the end user with a more consistent framework for the assessment of existing structures in both seismic and non-seismic design situations.

The text is organized as follows. Section 2 briefly describes the evolution of seismic design and assessment in the Eurocodes, to give the necessary context. Section 3 recalls the fundamental steps of reliability-based code calibration as referred to non-seismic design situations, with a focus on the design value method (DVM)[15] and the approximate method by König and Hosser.[10] Section 4 discusses how to apply the summarized code-calibration procedure in the time-variant seismic design-situation, specifies the correspotarget safety, introduces the probability distribution of seismic action effect (demand, used interchangeably) and resistance (capacity, used interchangeably), presents the calibration of partial factors for displacement-based design[16,17] and compares them with an existing alternative formulation.[18] Section 4 also presents the corrected resistance-side partial factor and illustrates its application for assessment rather than design. Section 5 provides concluding remarks.

2 | EVOLUTION OF SEISMIC DESIGN AND ASSESSMENT IN THE EUROCODES

The first-generation Eurocode 8 deals with seismic design and assessment in its Parts 1[7] and 3,[8] respectively. Part 1 presents a traditional approach to seismic design, based on linear analysis[19] and force-reduction, through the "behaviour factor" q, accounting for ductility, redundancy, and overstrength. Capacity design is enforced to ensure that the structure possesses the necessary global ductility.[20] The life-safety ("significant damage" in the Eurocodes) seismic action for ordinary structures (Consequence Class, or CC,2) is specified as a uniform hazard spectrum with return period $T_R = 475$ years.

Part 3 is a rather different document. At the time of release, among its main novelties, it included nonlinear static analysis, in its N2 version,[12] and displacement-based assessment.[16,17,21] In this respect, Part 3 was regarded as an "experiment," a fact confirmed by the large amount of material included as informative annexes rather than normative text: an intermediate step (displacement-based assessment) towards the introduction of a deformation-controlled approach to design.[13] The second-generation Eurocode 8[22] completes this transition. According to it, designers can limit themselves to prove, through nonlinear analysis, that the code-supplied deformation limits for the "ductile" failure modes (flexure with or without axial force) and the strength limits for "brittle" ones (shear) are not exceeded.[1] One further novelty of Part 3 was a framework to deal with the specific uncertainty of existing structures. While compatibility with the general Eurocode partial factors format[23] was required, it was necessary to adapt the format to account for the differences between assessment and design. At the time of drafting, well before its issuance in 2005, a codified reliability-based partial factors format for existing structures was not yet available.[24] Nonetheless, some related documents had been recently issued in the USA.[25,26] FEMA-356[26] introduced discrete knowledge levels (KLs) to quantify the information gathered on the existing structure, and so-called knowledge factors, function of the KL, to amplify force demands. Part 3 of Eurocode 8 adopted a similar format, keeping the KLs but replacing the knowledge factors with a Confidence Factor (CF) used to reduce resistance, rather than to increase demands. The values of CF, however, were never subject to a reliability-based calibration. The latitude of assessment results that could be obtained following this framework was later shown to be quite large.[27,28] Nonetheless, Part 3 had a large impact, since it was adopted in national codes at least in Italy[29,30] and Greece[31] where it was used to assess and retrofit several thousands of buildings over the years. It even had an unintended consequence, informing recent attempts to address the problem of assessment of bridges approaching their service life under traffic loads.[32] During drafting of the second-generation Eurocode 8, Part 3 was thoroughly revised and confidence factors were replaced by new partial factors on resistance that better conformed to the general Eurocode safety format and had a stricter relation with the actual amount of information gathered at the time of assessment on aleatoric and epistemic uncertainties affecting each resistance model. The first iteration of this effort is described in.[33]

3 | FUNDAMENTALS OF RELIABILITY-BASED CODE-CALIBRATION

The goal of code-calibration is to provide an accepted minimum safety and economy over a desired set of structures and design situations, defined as scope of application of the code. This could be done explicitly, in a risk-based approach, by minimizing the total lifetime expected cost, even though attempts to derive partial factors in such a way are still scarce, see for example.[34] Mostly, code-calibration is still reliability-based, that is, with a target in terms of probability of failure that incorporates, mixing them, both the desired minimum safety and economy. The procedure for reliability-based code-calibration in non-seismic design situations is a well established one[1,2,35] and can be summarized in the following three steps.

The first step consists in setting the scope of the calibration: the range of actions, materials, and structural types for which the calibration results should hold. This consists in selecting a set of design cases, for example, flexure or shear for a set of reinforced concrete and steel beams and columns, under dead, live, and wind load.

The second step requires establishing a target reliability level β_t. The safety level implied by the previous generation code is usually taken as a reference. If β_t is not known explicitly, it is established by performing reliability analysis. Multiple target values can also be used, e.g., when actions and materials in the selected scope are different enough as to have design situations characterized by considerably different implied safety. This is exactly the case for the seismic design situation, generally found ([34] and several later international studies[36]) to be associated with values of $\beta \leq 2.0$, lower than those typical of gravity load combinations ($\beta \geq 3.0$).[2]

The third and final step involves choosing a code format[3] and finding optimal values of the corresponding partial factors that minimize the distance between actual and target safety, for all design cases in the calibration space:

$$\min_{\gamma} \sum_i w_i [\beta_t - \beta_i(\boldsymbol{\gamma})]^2 \tag{1}$$

where β_i is the reliability index of the i-th design case (step 1), a function of the partial factors collected in the vector $\boldsymbol{\gamma}$, as explained later, and w_i the weight reflecting its importance to the overall design practice. The above minimization can also be constrained, for example, with a minimum β_i, and there are even variants in terms of p_f or other derived quantities, rather than β.

For each design case in the calibration space the link between β_i and the partial factors $\boldsymbol{\gamma}$ in Equation (1) is offered by reliability analysis. The latter is performed on a limit state function (LSF), $g(\boldsymbol{x})$, of the random variables $\boldsymbol{x}$, whose negative values denote failure. In the simplest case of two random variables, resistance, R, and action effect, E, it can be put in the linear form:

$$g(\boldsymbol{x}) = R - E \tag{2}$$

where the subscript "i" has been dropped for ease of notation. Any probabilistic analysis method, capable of dealing with complex LSFs, can be used to evaluate the probability $p_f = p(g < 0)$, from which β follows with the well-known relation. In practice, however, owing to the computational effort, the First Order Reliability Method (FORM)[2,38] has been traditionally used. Since LSFs for all design cases will be like (2), it may not be obvious how each design case enters into (2). What changes, as a function of the design case and of the partial factors in $\boldsymbol{\gamma}$, is the probability distribution of the resistance. Assume both R and E to be described, without loss of generality, by two-parameter distributions. The parameters are related to the respective mean and variance, (μ_R, σ_R^2) and (μ_E, σ_E^2). Three of the latter, μ_E, σ_E^2, and σ_R^2, are given in any design problem. The designer only has (indirect[4]) control over the mean resistance μ_R. The mean μ_R is determined with the "design equation," obtained taking the equal sign in the design inequality, similar but distinct from the limit state function[15]:

$$R_d \geq E_d \tag{3}$$

The dependence on the partial factors can be made explicit by rewriting the design equation as:

$$\frac{R_k}{\gamma_R} = \gamma_E \, E_k \tag{4}$$

in a Eurocode-like format or, alternatively, according to the LRFD format, as:

$$\phi R_n = \gamma_E \, E_n \tag{5}$$

In (4) and (5), the subscripts "k" and "n" stand for characteristic and nominal and in both cases denote chosen representative values of resistance and action effect prescribed by each code to be used in design.[5] Once $R_k = \gamma_R \, \gamma_E E_k$ is found from the design equation, given σ_R^2 the mean resistance μ_R and the resistance distribution follow.

Equations (3) and (4) are the basis of the DVM for determining the partial factors.[37] Used together they yield:

$$\gamma_R = \frac{R_k}{R_d} \tag{6}$$

$$\gamma_E = \frac{E_d}{E_k} \tag{7}$$

which allow one computing the partial factors from the design and the representative values of the variables. This problem is the inverse of the previous one, where reliability analysis of (2) provides β for given factors $\boldsymbol{\gamma}$, E_k and σ_R. Now instead it is set $\beta = \beta_t$ and the design point is found. The design values E_d and R_d are then taken as the coordinates of the design point. For general probability distributions, the design point is found by FORM as the point closest to the origin

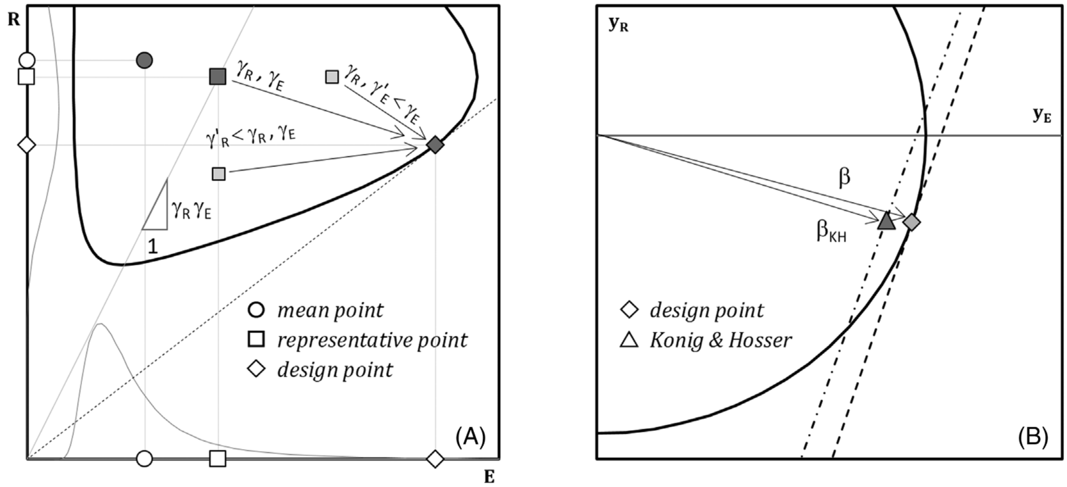

FIGURE 1 Relation between design point, characteristic point, and partial factors (note that by "mean point" in standard normal space it is meant the mapping of the mean point from the physical space, thus it does not coincide with the origin). For this example, the distribution of E is an Extreme value Type II (or Fréchet) and the distribution of R is Lognormal

in the transformed space of standard Gaussian variables $\boldsymbol{y}$. Its distance from the origin is the so-called Hasofer and Lind reliability index[39]:

$$\beta_{HL} = |\boldsymbol{y}_d| \tag{8}$$

Its direction cosines are the sensitivity factors α. These quantities provide the direct link between target reliability and the design values in the physical space, needed to evaluate partial factors by (6) and (7):

$$R_d = F_R^{-1}\left[\Phi\left(-\alpha_R\beta_t\right)\right] \tag{9}$$

$$E_d = F_E^{-1}\left[\Phi\left(-\alpha_E\beta_t\right)\right] \tag{10}$$

where F_R, F_E, and Φ are the cumulative distribution functions (CDF) of reistance, action effect and the standard Gaussian variable. Figure 1 illustrates the above in both the physical and the standard normal space. Figure 1A shows the marginal probability density functions (PDFs) of E and R plotted on the respective axes as grey lines, with the mean, representative and design values of each variable marked with a circle, square, and diamond white marker, respectively. The points of corresponding coordinates are shown with the same marker in grey color, for example, a grey circle for the mean point (μ_E, μ_R). The joint PDF is shown through its isoline (black solid) tangent at the design point to the limit state surface $R - E = 0$ (black dashed). The representative values R_k and E_k are, for illustration purposes, set equal to the median and the 90-th fractile, respectively (square markers). Their ratio $R_{50\%}/E_{90\%}$ is equal to the product of the partial factors $\gamma_E\gamma_R$, as implied by (6) and (7) since $E_d = R_d$, and is set for this example equal to 2.6. An arrow indicates how the partial factors $\boldsymbol{\gamma} = (\gamma_R, \gamma_E)$ transform the reference point (grey square) into the design one (grey diamond). Two alternative smaller square markers underline how the choice of the representative point is arbitrary and for a given problem what is fixed is the design point while the partial factors change as a function of the representative values of the two variables: in one case R_k is left unchanged and E_k is raised to the 97-th fractile, so that γ_R is the same and γ_E decreases to γ'_E, while in the other E_k is unchanged and R_k is decreased to the 16-th fractile, thus γ_E stays the same and γ_R decreases to γ'_R.

Figure 1B shows the design point in the standard normal space and the isoline of the joint PDF tangent to the limit state surface, which is a circle of radius β. Due to the nonlinearity in the transformation from $\boldsymbol{x}$ to $\boldsymbol{y}$ the transformed limit state function is not linear (the figure only shows the tangent to the design point, as a black dashed line).

For the developments to follow, it is noted that analytical solutions exist for the simple case of the linear LSF in (2) when R and E are either both Gaussian or Lognormal. In the latter case, Equation (8) becomes:

$$\beta = \frac{\mu_{\ln R} - \mu_{\ln E}}{\sqrt{\sigma^2_{\ln R} + \sigma^2_{\ln E}}} \tag{11}$$

where means and variances are the logarithmic ones. Equation (11), known as Cornell's reliability index,[40] depends only on second-moment information. Further, the sensitivity factors α_R and α_E, normally an outcome of the FORM analysis, also have an anlytical expression:

$$\alpha_R = \frac{\sigma_{\ln R}}{\sqrt{\sigma^2_{\ln R} + \sigma^2_{\ln E}}} = \frac{\sigma_{\ln R}}{\sigma_t} \tag{12}$$

and:

$$\alpha_E = \frac{-\sigma_{\ln E}}{\sigma_t} \tag{13}$$

Finally, Equations (6) and (7) become, respectively:

$$\gamma_R = \frac{\exp\left(\mu_{\ln R} + \kappa_R \sigma_{\ln R}\right)}{\exp\left(\mu_{\ln R} - \alpha_R \beta_t \sigma_{\ln R}\right)} = \exp\left(\left(\alpha_R \beta_t + \kappa_R\right)\sigma_{\ln R}\right) = \exp\left(\alpha_R^2 \beta_t \sigma_t\right)\exp\left(\kappa_R \sigma_{\ln R}\right) \tag{14}$$

$$\gamma_E = \frac{\exp\left(\mu_{\ln E} - \alpha_E \beta_t \sigma_{\ln E}\right)}{\exp\left(\mu_{\ln E} + \kappa_E \sigma_{\ln E}\right)} = \exp\left(-\left(\alpha_E \beta_t + \kappa_E\right)\sigma_{\ln E}\right) = \exp\left(\alpha_E^2 \beta_t \sigma_t\right)\exp\left(-\kappa_E \sigma_{\ln E}\right) \tag{15}$$

where κ_R and κ_E are the number of logarithmic standard deviations from the log-mean corresponding to R_k and E_k, respectively. The last four equations provide the partial factors in analytical form for any given choice of β_t, $\sigma_{\ln E}$, and $\sigma_{\ln R}$.

As a final remark, it should be observed that the sensitivity factors α_R and α_E depend on both $\sigma_{\ln E}$ and $\sigma_{\ln R}$. This represents a problem if, for example, separate calibration of resistance and load-side partial factors is sought (this has been a goal to decouple "material" codes from "actions" codes). König and Hosser,[10] however, have shown that approximate[6] a-priori constant values $\bar{\alpha}_R = 0.8$ and $\bar{\alpha}_E = -0.7$ can be used, within reasonably large bounds for the ratio σ_E/σ_R (or $\sigma_{\ln E}/\sigma_{\ln R}$), with a guaranteed deviations $|\Delta\beta| \leq 0.5$ from β_t. Figure 1B illustrates this fact, showing with a triangular marker the design point obtained solving a FORM with the same demand PDF and a resistance PDF consistent with a median resistance obtained as $\gamma_R \gamma_E E_k$, with γ_R and γ_E computed using $\bar{\alpha}_R = 0.8$ and $\bar{\alpha}_E = -0.7$ in expressions (14) and (15), respectively. In this specific case, $\beta_{KH} < \beta$. It should be noted that the variation of σ_E/σ_R (or $\sigma_{\ln E}/\sigma_{\ln R}$) is equivalent to spanning the range of design cases in the calibration space. In this respect, setting constant values $\bar{\alpha}_R$ and $\bar{\alpha}_E$ eliminates the need of solving the optimization problem in (1). This "simplified Level II method" was therefore very influential for all subsequent European normative developments. The method was also the source from which it was drawn when, much later, the need arose to derive a safety format for the assessment of existing structures within the International Federation of Structural Concrete (fib). This effort has led to the well-known Bulletin 80,[41] which has been a reference for the draft Eurocode on assessment of existing structures for non-seismic design situations.[40] For all these reasons, it is reported here for later comparison.

4 | CALIBRATION FOR THE SEISMIC DESIGN SITUATION

4.1 | Time-variant reliability problem

Section 3 summarized the fundamentals of reliability-based code-calibration for time-invariant reliability problems, while the seismic case is a time-variant one, where reliability is usually expressed in terms of first-passage probability or probability of time to first failure.[42] The occurrence of earthquakes is a stochastic process and also resistance varies with time, due to deterioration and damage-accumulation, for example.[43,44] The treatment of this reliability problem requires some care, see, for example.[45] The time-invariant approach of Section 3, however, can still be used under two assumptions. The first assumption, acceptable for mainshocks, is that earthquake occurrences follow a Poisson process, so that the distribution of demand in any unit time-interval is the same. The second is that deterioration of resistance is also neglected, which is acceptable for a well-maintained structure, not exposed to previous damaging earthquakes during its lifetime. Under these conditions, probability of failure in a reference period can then be found either as a function of the failure rate, product of the rate of earthquakes times the time-invariant probability of failure in a single event, or by comparing the maximum demand in the reference period, function of the rate of earthquakes, with the time-invariant resistance. The latter option is chosen in the following. The former option, in fact, implies, in the passage from the failure rate to the probability of failure in the reference period, that failure events in non-overlapping periods are independent, which is an approximation since the time-invariant resistance correlates them. This approximation, however, is inevitable when establishing the target reliability (following Section 4.2), which in the code is specified on an annual basis. In any case, it is a good approximation since the uncertainty in resistance is much lower than that in the seismic action effect.

4.2 | Target reliability

Unlike its predecessor,[22] the second-generation Eurocode 8[34] declares explicitly a target reliability. This is done with reference to the Near Collapse (NC) limit state. The corresponding target annual probability is denoted as P_{NCt}, and for the sake of brevity in the following as p_1. Its proposed default value for ordinary structures, 2×10^{-4}, is in line with the reliability assessed for traditional seismic designs worldwide, as summarized in.[36] Under the made assumptions, the failure process is Poisson like that of earthquake occurrences. Its rate or rate of limit state violation, λ_{LS}, is the product of the rate of all events on all seismic sources affecting the site, times the probability of failure in one event, and can be approximated, for values smaller than about 0.1 by the annual probability p_1. The 50-year reliability index, is thus given by[7]:

$$\beta_{50} = -\Phi^{-1}\left(1-(1-p_1)^{50}\right) \cong 2.33 \tag{16}$$

Finally, it is noted that the partial factors discussed herein are meant for use mainly[8] in local member-level verifications, which is how the NC limit state should be checked according to the Eurocode. Actual collapse is both a more severe condition and a global, system-level one, for which in general a further margin exists from the attainment of a member-level collapse condition,[33] depending on the components' arrangement and capacity correlation, see, for example.[46] As a result, the target reliability specified for the NC limit state implies a smaller acceptable probability for the actual collapse (not explicitly defined in the Eurocode).

4.3 | Probability distribution of demand

Demand E increases with the input seismic intensity measure (IM), S. This relationship is commonly put in the form of a power-law.[45] The IM, most commonly a scalar like, for example, the spectral acceleration at the fundamental period, $S = S_a(T)$, is always only an incomplete representation of any specific ground motion. Therefore, ground motions with the same IM value induce in general different demands E at any given structural location. These random deviations are called record-to-record variability.[47] This additional variability in E, conditional on S, is often described with a multiplicative variable η. As a result, demand is a random variable function of two random variables, S and η, according to:

$$E = aS^b\eta \tag{17}$$

and code-calibration according to (1) can be performed replacing (2) with the three-variable LSF:

$$g(\boldsymbol{x}) = R - aS^b\eta \tag{18}$$

whose corresponding design equation is:

$$\frac{R_k}{\gamma_R} - aS_k^b \cdot \eta_k \tag{19}$$

Variable η can be modelled as a unit-median Lognormal with logarithmic standard deviation $\sigma_{\ln E|S}$. The distribution of S is thus the only further element needed to on the demand side. Under the assumptions made, the interest is in the maximum demand over L years, denoted in the following as S_L, whose distribution is obtained from the seismic hazard curve (SHC), $\lambda_S(s)$, assuming a Poisson distribution for the occurrences:

$$F_{S_L}(s) = p\ (n = 0, L; \lambda_S) = \frac{(\lambda_S(s) L)^0 \exp(-\lambda_S(s) L)}{0!} = \exp(-\lambda_S(s) L) \tag{20}$$

The SHC can be written as:

$$\lambda_S\ (s) = \sum_i \nu_i \int_M \int_R \left[1 - \Phi\left(\frac{\ln s - \mu_{\ln S|m,r,i}}{\sigma_{\ln S|m,r,i}}\right)\right] f_{M,R}(m, r|i)\, dmdr \tag{21}$$

where ν_i is the mean annual rate of all events on source i above a minimum magnitude threshold, $f_{M,R}(m, r|i)dmdr$ is the probability of occurrence of an event on source i with magnitude M and source-to-site distance R in the neighborhood of (m, r), and the term within square brackets is the Lognormal complementary CDF of S conditional on magnitude and distance. The terms $\mu_{\ln S|m,r,i}$ and $\sigma_{\ln S|m,r,i}$ depend on the source i through source-dependent parameters other than magnitude and distance such as, e.g., soil site conditions or faulting style.[9]

Since (21) has in general no closed form solution, the CDF in (20) is not any known analytical probability distribution. In the following, two approximations will thus be considered. The first one is sought for its accuracy, while the second one as a convenient alternative to revert back to a two-variables limit state and exploit the results in (14) and (15). The possibility of using the second is established comparing the results of reliability analyses carried out with both.

The first model considered to approximate F_{S_L} is the extreme value (EV), Type II model, or Fréchet. EV models were found to describe well the distribution of maximum intensity already by Cornell.[48] Iervolino also recently looked in detail to analytical probability distributions to describe seismic intensity,[49] finding that, among several candidate models, including also EV-I or Gumbel, Lognormal and Gamma, the Fréchet distribution offers the best fit. One way to model S_L as Fréchet, for example, is through a power-law fit to the SHC. Equation (20) then becomes (the tilde denoting this is an approximation):

$$\tilde{F}_{S_L}\ (s) = \exp\left[-\left(k_0 s^{-k}\right) L\right] = \exp\left[-\left(\frac{s}{\sqrt[k]{k_0 L}}\right)^{-k}\right] = \exp\left[-\left(\frac{s}{u}\right)^{-k}\right] \tag{22}$$

which is Fréchet with parameters $u = \sqrt[k]{k_0 L}$ and k. Note, for later use in the comparison among the two approximations, that Equation (22) can also be written, in terms of a normalized variable $\ln w$, as:

$$\tilde{F}_{S_L}\ (s) = \exp\left[-\exp\left(-\frac{\ln x - \mu_F}{\sigma_F}\right)\right] = \exp\left[-\exp(-\ln w)\right] \tag{23}$$

Equating (22) and (23) one easily finds the new parameters (μ_F, σ_F) as a function of (u, k):

$$\sigma_F = \frac{1}{k} \tag{24}$$

$$\mu_F = \ln u \tag{25}$$

The second approximation $\check{F}_{S_L}$ is the Lognormal distribution. Despite not being the optimal one according to,[49] this is an attractive choice, because it also describes well the resistance side of the problem, ultimately allowing reduction to the reliability problem with two Lognormal variables, and use of (14) and (15) to evaluate partial factors. The Lognormal approximation can be written as:

$$\check{F}_{S_L}(s) = \Phi\left(\frac{\ln s - \mu_{\ln S_L}}{\sigma_{\ln S_L}}\right) = \Phi(\ln z) \tag{26}$$

with parameters $(\mu_{\ln S_L}, \sigma_{\ln S_L})$, easily found equating $\check{F}_{S_L}$ to F_{S_L} and performing a least square fit.

Figure 2 illustrates both approximations for two sites in Italy, representative of low (Milan) and high seismicity (L'Aquila), respectively, and four IMs, the spectral ordinates at periods: $T = 0s,\ 0.25s,\ 1.0s,\ 2.0s$, where the first three correspond to the PGA, the plateau acceleration, S_α, and the long-period spectral ordinate, S_β, respectively (symbols are

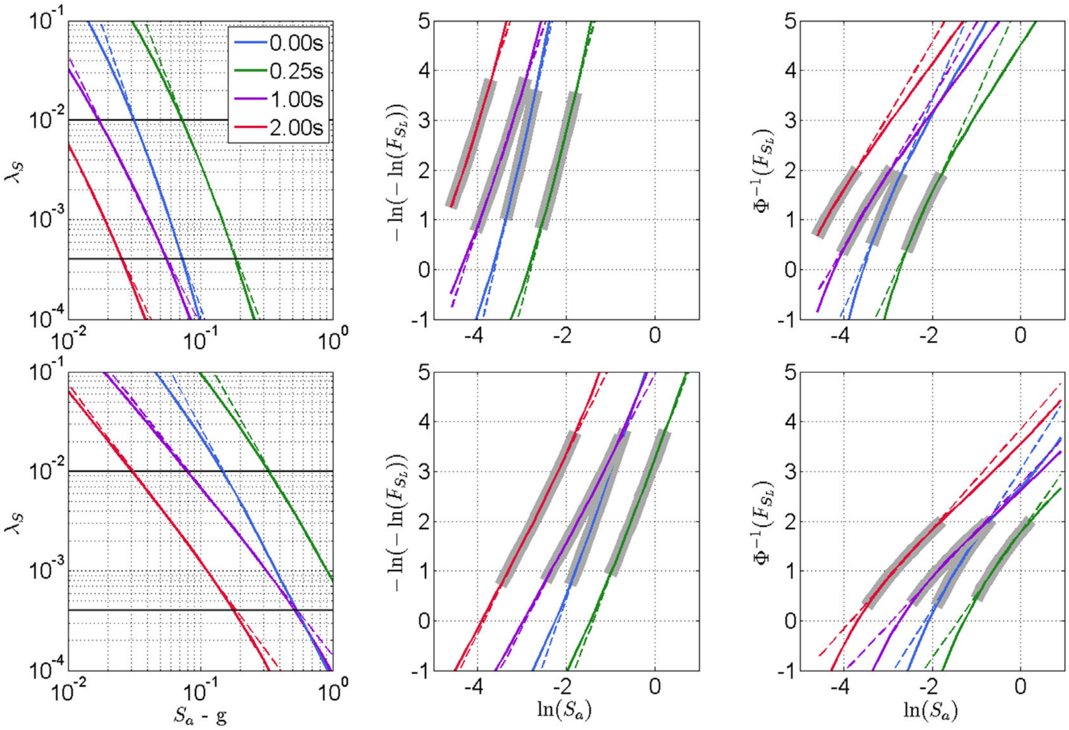

FIGURE 2 Seismic hazard curves (left) for four different spectral ordinates, for the sites of Milan (top) and L'Aquila (bottom), together with the Fréchet (center) and Lognormal approximation (right) of the CDF of maximum intensity over a 50 years period. Continuous line are exact numerical values (of hazard curves on the left, and of the CDF in the center and right plots), while dashed lines denote the approximations (power-law for the hazard, with parameters k_0 and k obtained from the Fréchet fit)

those used in the second-generation Eurocode 8). The left column of the figure shows the SHC's, computed using the ZS9 area source model for Italy,[50] using all zones within a 200 km radius from the site, and the ground motion prediction equation for horizontal components by Ambraseys et al.[51] The central and right columns of the figure show the distributions F_{S_L} (solid), together with the Fréchet $\tilde{F}_{S_L}$ (center) and Lognormal $\check{F}_{S_L}$ (right) approximations (dashed), respectively.

Note that the Fréchet approximation can be performed in two ways. The first fits the SHC and yields k_0 and k, which are then transformed in the Fréchet parameters. The second aims at the best approximation of F_{S_L}, fitting the Fréchet distribution directly to it, finding u and k first to then retrieve $k_0 = u^k/L$, if needed (k coincides with the slope of the hazard approximation). The latter was the procedure used in Figure 2. A least square regression was carried out on the range of intensities with return periods between 100 and 2500 years[10], marked by horizontal black lines in Figure 2 (left column). These intensity ranges are also highlighted with a grey marker on the center and right plots. Note that the approximating functions are linear in the plots since they have both been fitted in the transformed planes: $-\ln(-\ln F_{S_L})$ versus $\ln s$ or $\Phi^{-1}(F_{S_L})$ versus $\ln s$. All fit parameters are reported in Table 1.

The relationship between $(\mu_{\ln S_L}, \sigma_{\ln S_L})$ and (k, u) is established by fitting $\check{F}_{S_L}$ to $\tilde{F}_{S_L}$, rather than directly to F_{S_L}, which in turn can be accomplished by finding the optimal values of the parameters of a mapping of $\ln z$ to $\ln w$, defined in (26) and (23), respectively:

$$\ln z = c_1 + c_2 \ln w \tag{27}$$

that minimize the difference between the two distributions in the $-\ln(-\ln F_{S_L})$ versus $\ln w$ plane:

$$\begin{aligned} &\min_{c_1,c_2} \int_{\ln w_{lb}}^{\ln w_{ub}} \left\{\left[-\ln\left(-\ln \tilde{F}_{S_N}\right)\right] - \left[-\ln\left(-\ln \Phi\left(\ln z\right)\right)\right]\right\}^2 d\ln w \\ &= \min_{c_1,c_2} \int_{\ln w_{lb}}^{\ln w_{ub}} \left\{\ln w - \left[-\ln\left(-\ln \Phi\left(c_1 + c_2 \ln w\right)\right)\right]\right\}^2 d\ln w \end{aligned} \tag{28}$$

TABLE 1 Parameters of the Fréchet and Lognormal approximations of S_L, in terms of spectral acceleration at four vibration periods, for the two Italian sites of Milan and L'Aquila

Site	IM	k	u	k_0	$\mu_{\ln S_L}$	$\sigma_{\ln S_L}$	$\sigma_{\ln E}$	V_E
Milan	PGA = (0s)	3.820	0.027	1.97E-08	-3.626	0.479	0.565	0.613
	$S_\alpha = S_a\,(0.25s)$	3.458	0.061	1.24E-06	-2.789	0.514	0.595	0.652
	$S_\beta = S_a\,(1.0s)$	2.713	0.014	1.69E-07	-4.296	0.668	0.732	0.842
	$S_a(2.0s)$	2.830	0.007	1.31E-08	-5.081	0.676	0.740	0.853
L'Aquila	PGA =	2.524	0.114	8.24E-05	-2.160	0.714	0.774	0.906
	$S_\alpha = S_a\,(0.25s)$	2.328	0.249	7.89E-04	-1.402	0.792	0.847	1.024
	$S_\beta = S_a\,(1.0s)$	1.698	0.054	1.41E-04	-2.904	1.059	1.101	1.536
	$S_a(2.0s)$	1.794	0.021	1.97E-05	-3.836	0.999	1.043	1.402

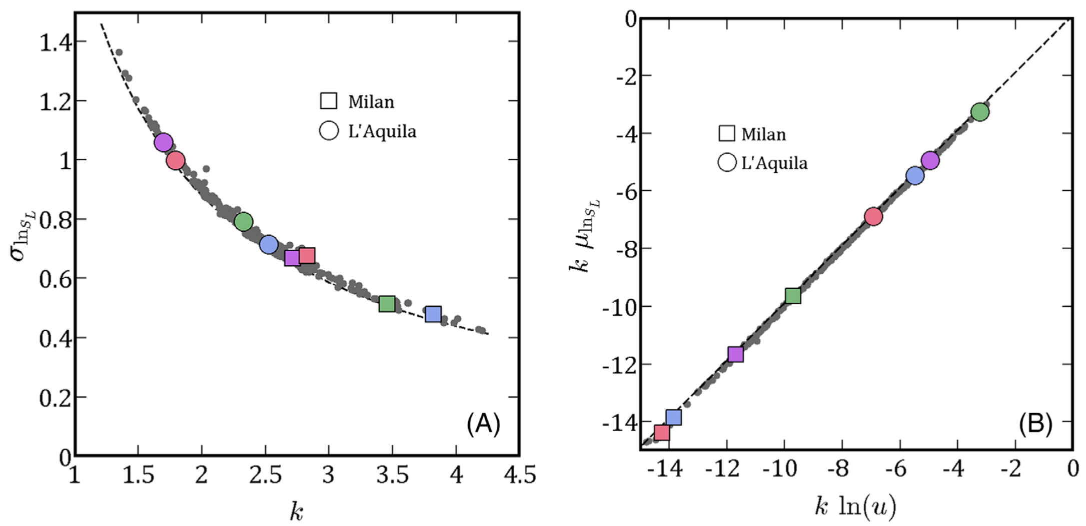

FIGURE 3 Relationship between the parameters of the Lognormal and Fréchet fit to the CDF of maximum intensity over N years

In (28) w_{lb} and w_{ub} are the lower and upper integration bounds corresponding to the values of w associated to the return periods of 100 and 2500 years. Once the optimal pair ($c_1 = -0.069,\ c_2 = 0.569$) is known, replacing the expressions of $\ln z$ and $\ln w$ in Equation (27) from (23) and (26), together with Equations (24) and (25) yields:

$$\sigma_{\ln S_L} = \frac{\sigma_F}{c_2} = \frac{1}{c_2 k} \tag{29}$$

$$\mu_{\ln S_L} = \mu_F - \frac{c_1}{c_2}\sigma_F = \ln u - \frac{c_1}{c_2 k} \tag{30}$$

These relationships show how $\sigma_{\ln S_L}$ depends only on the hazard slope k. They are plotted in Figure 3A and B, respectively, as black lines. Figure 3 also shows (grey circles) the actual values of $\sigma_{\ln S_L}$ and $\mu_{\ln S_L}$ computed by fitting $\check{F}_{S_L}$ directly to F_{S_L} (as in Figure 2, left), for all the provincial capitals in Italy and the four spectral ordinates considered in Figure 2. The square and circle markers identify the sites of Milan and L'Aquila, respectively (the colors, denoting spectral ordinates, are the same as in Figure 2). These points follow closely the trends described by (29) and (30). These equations can thus be used to establish parameters for both $\tilde{F}_{S_L}$ and $\check{F}_{S_L}$ in a consistent manner when comparing the reliability results in Section 4.4.

Let us finally come back to the demand E. If S_L is approximated as Fréchet, E will be the product of a Lognormal η and a Fréchet variable aS_L^b (the power of a Fréchet is still Fréchet), with parameters:

$$u' = au^b \text{ and } k' = \frac{k}{b} \tag{31}$$

This product does not result in any known analytical distribution, thus the three-variables LSF will be used. If S_L is instead approximated as Longormal, then the product $aS_L^b\eta$ is also lognormal, with parameters:

$$\mu_{\ln E} = \ln a + b\mu_{\ln S_L} + \ln\hat{\eta} = \ln a + b\mu_{\ln S_L} \text{ and } \sigma_{\ln E} = \sqrt{b^2\sigma^2_{\ln S_L} + \sigma^2_{\ln E|S}} \tag{32}$$

and reliability analysis can be performed both on the limit state (18) or on the two-variables limit state (2). In the latter case partial factors would be given by (14) and (15). The last two columns of Table 1 report the values of $\sigma_{\ln E}$ (obtained using the common value of $\sigma_{\ln E|S} = 0.3$), and of the corresponding coefficient of variation (CV), V_E, for the two sites of Milan and L'Aquila and the four IM considered before. Note for later use that the minimum and maximum value of $\sigma_{\ln E}$ are 0.56 and 1.1, respectively.

4.4 | Probability distribution of capacity

In the Eurocodes, verification according to the displacement-based approach is carried out in terms of deformation (chord rotation in members) or forces (shear in members or joints and connections), for ductile or brittle failure modes, respectively. The chord rotation models were first introduced in Part 3[50] with reference to reinforced concrete members, based on the initial work of the school of Patras.[52] These models, continuously improved over time,[53,54] have been later included also in the *fib* Model Code 2010.[55] Second-generation Part 3[56] contains their latest version,[57] and also covers extensively steel and composite structures,[58,59] as well as masonry and timber. The first-generation Part 3[58] also included a "seismic" shear strength model,[60] which was in line with other international proposals.[61–63] This model is included in the second generation Eurocode 8[56,64] in an updated[65,66] form, compatible with the shear strength model for vertical loads. Formally, all these deformation and strength models are now included in Part 1-1,[64] for displacement-based design of new structures whereby rules to modify them for existing non-conforming members are given in Part 3.[56]

All these models consist of semi-empirical formulas, $r(\boldsymbol{x})$, derived to be unbiased, and are provided with the estimate of the CV of the ratio of experimental to predicted resistances[11]. As shown in,[33] they can all be described as Lognormal (as it is customary for resistance variables[31]), as the product of $r(\boldsymbol{x})$ and of the model error, a unit-median Lognormal variable ϵ_R with logarithmic standard deviation function of the above CV. As shown in,[33] a Taylor expansion of the natural logarithm of this product around the median of the basic variables $\hat{\boldsymbol{x}}$ yields:

$$\begin{aligned}\ln R = \ln\left[r(\boldsymbol{x})\epsilon_R\right] &\cong \ln r(\hat{\boldsymbol{x}}) + \sum \left.\frac{\partial \ln r}{\partial \ln x_{\mathrm{i}}}\right|_{\hat{\boldsymbol{x}}} \left(\ln x_{\mathrm{i}} - \mu_{\ln x_{\mathrm{i}}}\right) + \tilde{\epsilon}_{\mathrm{R}} \\ &= \ln r(\hat{\boldsymbol{x}}) + \sum \frac{\hat{x}_{\mathrm{i}}}{r(\hat{\boldsymbol{x}})}\left.\frac{\partial r}{\partial x_{\mathrm{i}}}\right|_{\hat{\boldsymbol{x}}} \left(\ln x_{\mathrm{i}} - \mu_{\ln x_{\mathrm{i}}}\right) + \tilde{\epsilon}_{\mathrm{R}} = \ln r(\hat{\boldsymbol{x}}) + \sum c_{\mathrm{i}}\tilde{\epsilon}_{\mathrm{i}} + \tilde{\epsilon}_{\mathrm{R}}\end{aligned} \tag{33}$$

where $\tilde{\epsilon}_R = \ln\epsilon_R$ and the $\tilde{\epsilon}_i$'s are zero-mean Gaussian variables. It follows that the resistance can be approximated as a Lognormal random variable with median $r(\hat{\boldsymbol{x}})$ and a total logarithmic standard deviation, accounting for the model error \ and the variability in resistance resulting from uncertainty in the basic variables, given by:

$$\sigma_{\ln R} = \sqrt{\sigma^2_{\ln r} + \sum\left(c_i\sigma_{\ln x_i}\right)^2} \tag{34}$$

Note that, through the coefficients c_i, the above dispersion depends on the specific value of $\boldsymbol{x}$. Equation (28) was computed over many such calibration points for all resistance models in the process of drafting second-generation Eurocodes, with the model error $\sigma_{\ln r}$ supplied for each formula $r(\boldsymbol{x})$ and typical values of $\sigma_{\ln x_i}$ for the basic variables.[65] The result was initially used to evaluate resistance-side and formula-specific partial factors according to:

$$\gamma_R = \exp\left(\sigma_{\ln R}\right) \tag{35}$$

which is the same as (14), with $\kappa_R = 0$, since all formulas predict the median resistance[12], and setting $\alpha_R\beta = 1$. It should be noted that when the above values for the partial factors were derived, the choice of using as design (assessment) value of resistance its 16-th fractile (that is the meaning of (35), that is, taking the median minus one logarithmic standard deviation)

TABLE 2 Resistance models: Values of the model error, total logarithmic standard deviation, and corresponding coefficient of variation for a subset of the Eurocode resistance models

Model	Section shape	$\sigma_{\ln r}$	$\sigma_{\ln R}$	V_R
Ultimate chord rotation θ_u	Rectangular	0.20	0.22	0.22
	Circular	0.15	0.17	0.17
	Other (e.g., hollow)	0.20	0.21	0.21
Shear strength V_u	Rectangular	0.29	0.34	0.35
	Circular	0.21	0.30	0.30
	Other (e.g., hollow)	0.31	0.35	0.36

was not made to target any specific reliability index value, but, rather, it was made because it led to resistance values approximately equal to those resulting from the application of the previous code, as shown in.[33] Herein, the values of $\sigma_{\ln R}$, or, equivalently, of the corresponding V_R , are used in the reliability-based calibration. Note that Table 2 reports values for $\sigma_{\ln R}$ of the ultimate chord-rotation that are (much) lower than those reported in.[33] These values are those from,[57] where the authors have shown how performing better regression on the data from multiple experimental campaigns considerably reduces the model error. The values 0.20 and 0.50 are used in the following as lower and upper bound values for $\sigma_{\ln R}$ for reliability-based calibration. These values are appropriate to cover the range of variation of all models in Eurocode 8, including those for non-conforming members.

4.5 | Reliability analysis

Reliability analysis is first used to show how differences obtained in the reliability assessed using the Fréchet or the Lognormal approximation for the seismic intensity are very small and acceptable for all practical purposes, over the range of seismicity, resistance uncertainty and values of b of interest. Based on this result, it is thus proposed to use the Lognormal approximation and therefore expressions (14) and (15) to derive the partial factors. For even simpler code use, approximate constant values for the sensitivity coefficients α_R and α_E are derived, different from the usual ones,[7] and fit for use in a seismic design or assessment situation. Finally, a modified formulation for the resistance coefficient α_R is derived, to be used in a code where the seismic action effect is not amplified by a partial factor, such as, for example, Eurocode 8.

First, to compare the two demand approximations, FORM is carried out on the three-variables limit state (18) for the entire range of interest for the considered variables, with reference to the Near Collapse limit state and ordinary (CC2) structures. Correspondingly, the fractile of the seismic action, according to the current Eurocode 8 draft, has a return period T_R of 1600 years. The reference period is taken equal to the common service life value of fifty years, that is, $L = 50$. It follows that the characteristic value for the seismic action is the 97-th fractile:

$$F_{S_L}(s_k) = \exp(-\lambda_S(s_k)L) = \exp\left(-\frac{50}{1600}\right) = 0.97 \tag{36}$$

Parameter k is varied between 2.0 and 4.0, values representative of high and low seismicity, respectively.[13] Parameter k_0 is taken as inversely proportional to k with values obtained by plotting k versus k_0 for the same sites in Figure 3, for the four spectral ordinates in Figure 2, which confirm the expected negative correlation between k_0 and k (not shown). The actual relation between k_0 and k depends on the chosen spectral ordinate but choosing one or the other does not change the outcome of what follows. The triplet (k_0, k, L) yields parameter u. Lognormal parameters are obtained from k and u through Equations (29) and (30).

As for the demand given the intensity, $a = 1.0$ and two values are considered for b, equal to 0.8 and 1.2, respectively. Variable η, it is recalled, is unit-median ($\eta_k = 1$ in the design equation) Lognormal with logarithmic standard deviation $\sigma_{\ln E|S} = 0.3$.

Finally, to represent different failure modes, the two values 0.2 and 0.5 are taken for $\sigma_{\ln R,tot}$. The representative value of resistance R_k in the design Equation (19) is taken equal to the median. Consistently with current practice, for both the first[22] and draft second-generation Eurocode 8,[66] the seismic action effect is not amplified by a partial factor, i.e., $\gamma_E = 1.0$. Factor γ_R is taken proportional to the uncertainty in resistance, equal to 1.5 and 2.7, for $\sigma_{\ln R} = 0.2$ and 0.5, respectively.

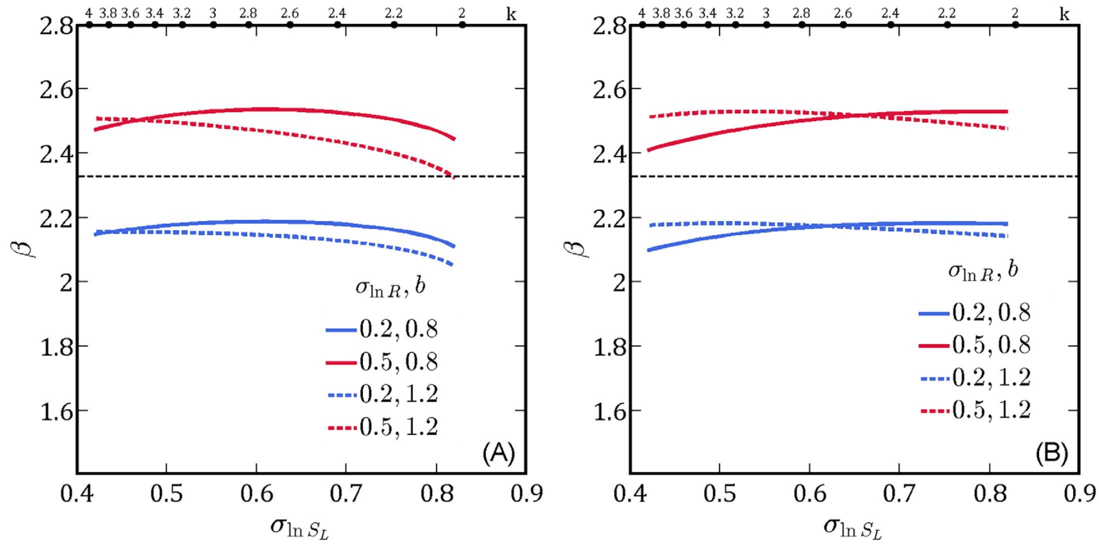

FIGURE 4 Variation of the reliability index over the calibration space: Fréchet (A) and Lognormal (B) approximation for S_L

Figure 4 shows the results, as plots of β versus either k (top) or $\sigma_{\ln S_L}$ (bottom). Panels A and B refer to the Fréchet and Lognormal approximation, respectively. Blue and red denote $\sigma_{\ln R} = 0.2$ and 0.5, while solid and dashed lines denote $b = 0.8$ and 1.2, respectively. The difference in reliability, for the same seismicity (same $\sigma_{\ln S_L}$), and values of b and $\sigma_{\ln R}$, are all lower than $\Delta\beta = 0.1$, confirming what was anticipated, that is, that the choice of the distribution of the seismic action is only leading to minor changes in β. The figure also shows that the chosen arbitrary values of γ_R lead to a reliability close to the target $\beta_t = 2.33$. This is because they are close to the optimal ones, introduced and justified later.

Based on the previous step, the Lognormal approximation is adopted from now on. The three-variable LSF (18) is replaced by the two-variable LSF (2), and the action effect E is modelled as Lognormal with parameters in (32). Partial factors are obtained according to Equations (14) and (15), with $\kappa_R = 0$, since the fractile of resistance is the median, while the fractile for the action effect is very close but not coincident to that of the seismic action, which is the 97-th fractile. The two are related as follows:

$$\kappa_E = \kappa_S \frac{b\sigma_{\ln S}}{\sigma_{\ln E}} = \kappa_S \frac{b\left(\frac{1}{b}\sqrt{\sigma^2_{\ln E} - \sigma^2_{\ln E|S}}\right)}{\sigma_{\ln E}} = \kappa_S \sqrt{1 - \left(\frac{\sigma_{\ln E|S}}{\sigma_{\ln E}}\right)^2} \tag{37}$$

Before moving on, it is observed that a generally decreasing trend of β with increasing seismicity might be expected,[68] based on two reasons: the effect of code minima and gravity-loads design, which tend to govern in low-seismicity sites but are not accounted for herein[14], and the expected value of seismic action exceeding the design threshold, or peak over the threshold.[37] The latter effect is strictly related to the fractile of the seismic action chosen for design. The effect is not visible due to relatively high return period used for design at NC, $T_R = 1600$ years, since the expected value of seismic actions exceeding a threshold asymptotically decreases to the threshold as the latter increases. In fact, the results in[37,68] refer to a design seismic action characterized by $T_R = 475$ years. If the previous exercise were repeated using this lower fractile, the dependence of β on seismicity would be apparent. This is shown in Figure 5 from a different perspective. Figure 5 shows the product of the two partial factors, $\gamma_R\gamma_E$, as a function of $\sigma_{\ln E}$, for $\beta_t = 2.33$. Note that plotting versus $\sigma_{\ln E}$ there is no dependence on b, since, according to (32), b enters into $\sigma_{\ln E}$. This is also the reason why the range of $\sigma_{\ln E}$ is slightly shifted to larger values, with respect to that identified by the minimum and maximum value of $\sigma_{\ln S_L}$ (see Table 1, where b was set to 1.0, while in the figure it varies between 0.8 and 1.2). In passing, this is a good place to observe that, while the resistance model uncertainty $\sigma_{\ln r}$ has been included, since it is the main contributor to the total resistance uncertainty $\sigma_{\ln R}$, the analysis model uncertainty is not. This term, accounting for the additional uncertainty in the determination of the demand given the seismic intensity stemming from the analysis method, is usually modelled with a multiplicative unit-median Lognormal random variable, often denoted by Θ. Its inclusion, as a third factor in the product $aS_L^b\eta$ of Equation (18) is straightforward and would lead, once again, to the two variables limit state (2). What would change is the value of $\sigma_{\ln E}$, that would shift a bit more to the right. This shift, however, would be minimal, given

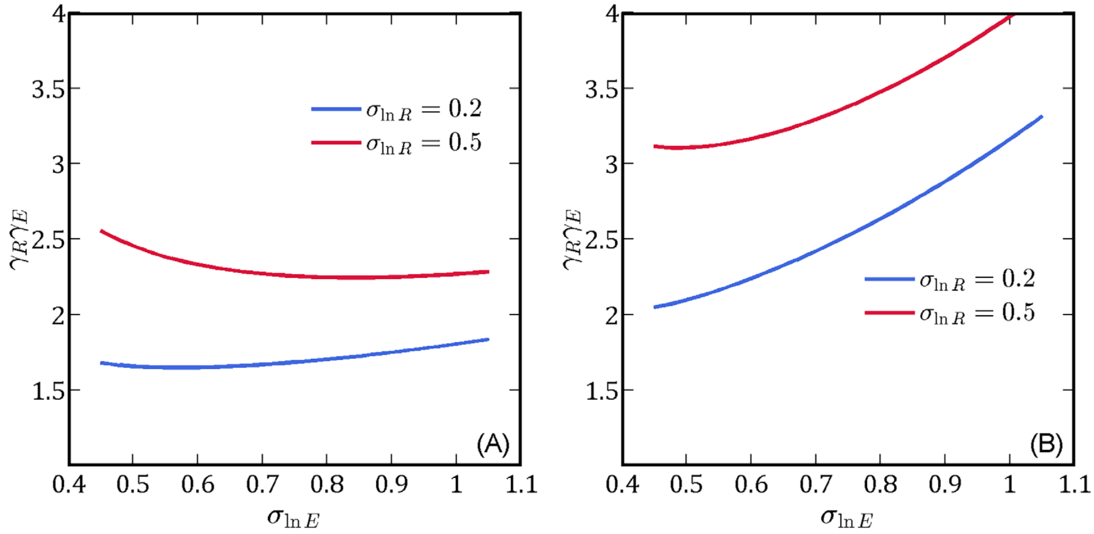

FIGURE 5 Values of $\gamma_R\gamma_E$ over the calibration space for $T_R =$ 1600 years (A) and $T_R =$ 475 years (B)

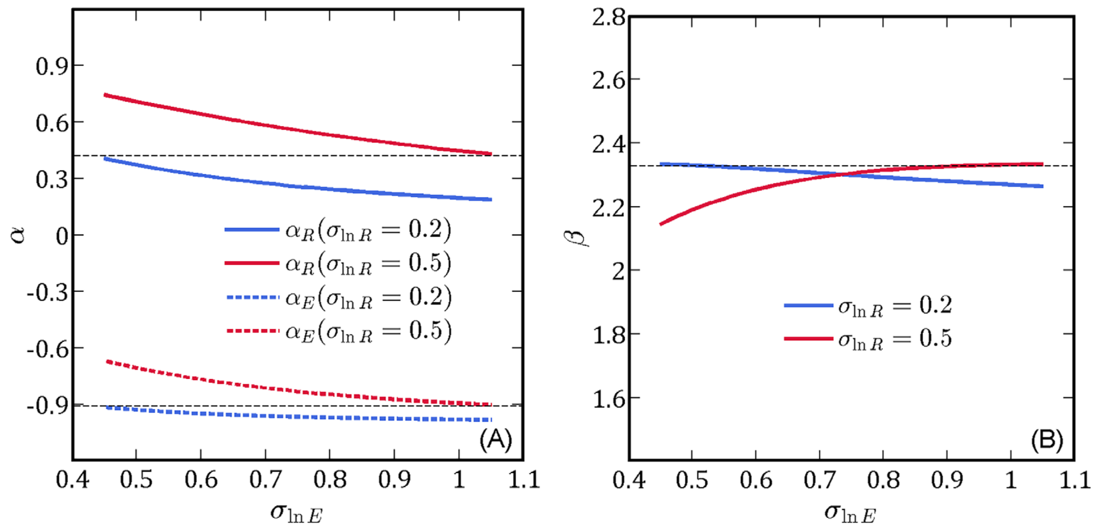

FIGURE 6 Values of α_R and α_E (A) and values of the reliability index obtained with $\bar{\alpha}_R = 0.42$ and $\bar{\alpha}_E = -0.91$ over the calibration space (B). Using the variable α_R and α_E in panel (A) would obviously lead to constant $\beta = \beta_t$

the relative size of the logarithmic standard deviation of Θ (usually in the order of 0.10), with respect to that of the other two components.

The product $\gamma_R\gamma_E$, according to the design Equation (4), provides the ratio of the required resistance R_k to the action effect E_k. Panel (a) and (b) show $\gamma_R\gamma_E$ for $T_R =$ 1600 years and $T_R =$ 475 years, respectively. Variability of $\gamma_R\gamma_E$ with seismicity ($\sigma_{\ln E}$) is more pronounced when $T_R =$ 475 years is used: the product increases with seismicity, that is, the capacity is required to be an increasing multiple of the demand to target the same reliability. This is consistent with the expectation of decreasing reliability with increasing seismicity for hazard-based design.[68]

Let us consider again now the sensitivity coefficients α_R an α_E. Figure 6A shows their value for the cases in the previous figure (both cases, as they do not depend on the fractile of the action), where again blue and red denote $\sigma_{\ln R} = 0.2$ and 0.5, while solid and dashed lines denote α_R and α_E, respectively. The values of α_R and α_E, for a fixed $\sigma_{\ln R}$, obviously decrease with $\sigma_{\ln E}$. Even for the lower seismicity, $\alpha_R < 0.8$ and $\alpha_E < -0.7$, that is, the usual values for other design situation.[18] This is consistent with the dominance of the seismic action uncertainty over that on resistance. As anticipated, one relevant question at this point is whether it is possible to use constant average values of the α coefficients, in the same way it is done for non-seismic design situation. This can be established by determining the reliability associated with designs carried out with the design Equation (4) and partial factors obtained by (14) and (15) using the average values $\bar{\alpha}_R =$ 0.42 and

$\bar{\alpha}_E = -0.91$ from Figure 6A. This has been done for the same values of k_0, k and therefore $\mu_{\ln S}, \sigma_{\ln S}$ used in Figure 4. For each case, knowing $\sigma_{\ln R}$ and R_k from (4) determines the Lognormal distribution of capacity. The reliability index follows in closed form according to Equation (13). The result is plotted in Figure 6B. The closeness to $\beta_t = 2.33$ is noteworthy, confirming that using partial factors according to Equations (14) and (15) with the suggested values $\bar{\alpha}_R = 0.42$ and $\bar{\alpha}_E = -0.91$ allow one controlling the reliability (within the limits of the Lognormal approximation for S_L).

4.6 | Comparison with known results

Before moving on, it is important to recognize that a reliability-based partial-factor format for seismic design and assessment has already been proposed earlier by Cornell and co-workers.[67–69] The paper is mostly known for the closed-form solution of the MAF of exceedance of a limit state, λ_{LS}, denoted in there as P_{PL}, probability of exceedance of the performance level (preferred to "limit state" in the US). The authors, however, also rearranged the formula in a LRFD-like safety-checking format for the seismic design situation, expressed as:

$$\phi \hat{C} \geq \gamma \hat{D}^{P_0} \tag{38}$$

where $\hat{C}$ is the median capacity, P_0 is the maximum acceptable annual probability of exceeding the limit-state, $\hat{D}^{P_0}$ is the median demand caused by a seismic action with probability P_0 of being exceeded (this would be aS^b with S characterized by a return period $T_R = 1/\lambda_S = 1/P_0$), while γ and ϕ are the load and resistance partial factors expressed, respectively, as:

$$\gamma = \exp\left(\frac{1}{2}\frac{k}{b}\beta_D^2\right) \tag{39}$$

$$\phi = \exp\left(-\frac{1}{2}\frac{k}{b}\beta_C^2\right) \tag{40}$$

In the previous equations k and b, as before, are the hazard slope and exponent of the power-law in (17), while β_C and β_D denote the logarithmic standard deviations of the capacity and the (conditional) demand, both assumed to distribute Lognormally. In the notation used so far, $\hat{C} = R_{50\%}$, $\beta_C = \sigma_{\ln R}$ and $\beta_D = \sigma_{\ln E|S}$.

Given the similarities in the basic assumptions, even if their derivation is different, it is of interest to compare the partial factors (39) and (40) with those in (14) and (15). This cannot be done directly, however, since they refer to different fractiles. One possibility is in terms of β, evaluated by means of FORM on limit state (2), for designs carried out according to (38), that is, determining the median resistance as $\hat{C} = \hat{D}^{P_0}\,\gamma/\phi$. The resistance PDF then follows with the chosen $\beta_C = \sigma_{\ln R}$ and the demand PDF is the one consistent with $\hat{D}^{P_0} = a\,(s_{P_0})^b = a(P_0/k_0)^{-b/k}$ and $\beta_D = \sigma_{\ln E|S}$. Figure 7A shows the results, for the usual values of $\sigma_{\ln R,tot}$ and the range of $\sigma_{\ln E}$ considered in all previous figures (annual probability is set to $P_0 = 2 \times 10^{-4}$ consistently with $\beta_t = 2.33$, $T_R = 5000$ years). These partial factors perform well, with a relatively constant value over different $\sigma_{\ln R}$ and $\sigma_{\ln E}$. This is not surprising, since, two site-dependent factors are used, including one on the demand-side. Comparing the results to the similar ones in Figure 6B, highlights how (38), (39), and (40) lead to a systematic (minor) overshooting of the target safety. This is reflected in the median capacity. When plotted versus the median capacity obtained using (14) and (15) in design Equation (4), Cornell's median capacity is always larger (about 10% larger), as shown in Figure 7B.

Even though several researchers provided improvements on the original formulas, for example replacing the linear hazard-fit with a quadratic one, for example,[69–71] it must be recognized that (38) already solved the problem of controlling the seismic risk using a deterministic safety-checking format. Nonetheless, these factors, as those in the previous section, are difficult to use, since they depend on the site seismicity, and are not in line with the practice of Eurocode 8,[70] which has no load-side partial factor γ_E. For this reason, the next section proposes a solution that can be easily implemented in the current normative framework.

4.7 | Code proposal

The previous discussion has shown how there would not be a non-uniform risk problem if partial factors were used on both resistance and seismic action effect sides, as done for the other non-seismic design situations, with values explicitly

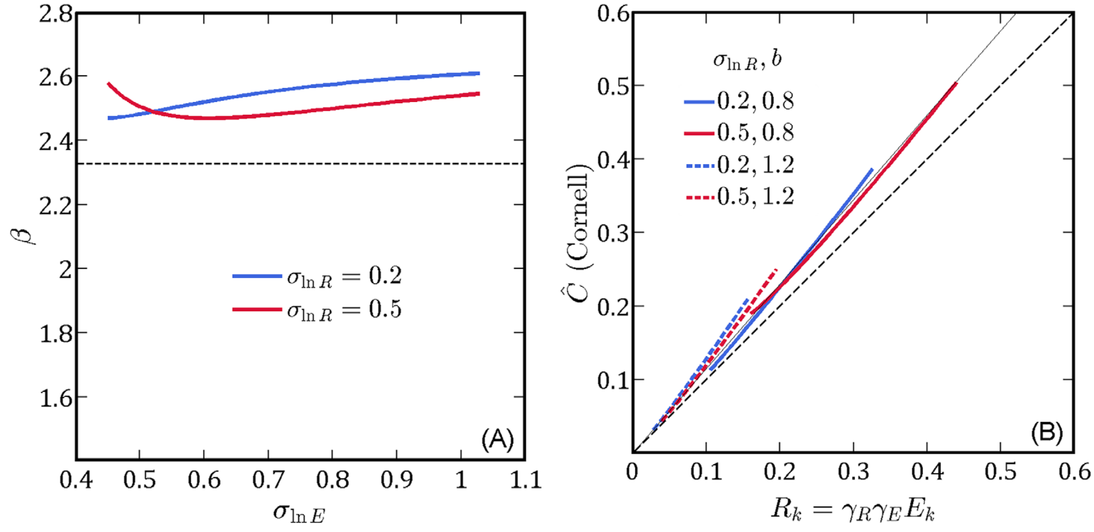

FIGURE 7 Reliability of designs carried out with Cornell et al partial factors (A) and comparison of median capacity obtained with Cornell's partial factors and those in Equations (14) and (15) (B)

dependent from the actual resistance and seismic action variabilities, that is, site dependent, even with approximate constant values $\bar{\alpha}_R$ and $\bar{\alpha}_E$. This is not current practice, however, since no γ_E is used and, more in general, partial factors are not site dependent. It is thus natural to think of correcting the single resistance-side partial factor to account for the missing factor on the demand side. Ideally, the corrected resistance-side partial factor should be equal to $\gamma_R\gamma_E$, the product of Equations (14) and (15):

$$\gamma_R^* = \exp\left(\alpha_R^*\beta_t\sigma_{\ln R}\right) = \gamma_R\gamma_E \tag{41}$$

Recalling that the median resistance is used ($\kappa_R = 0$) and that $\alpha_E^2 + \alpha_R^2 = 1$, $\gamma_R\gamma_E$ can be written as:

$$\gamma_R\gamma_E = \exp\left(\alpha_R^2\beta_t\sigma_t\right)\exp\left(\alpha_E^2\beta_t\sigma_t\right)\exp\left(-\kappa_E\sigma_{\ln E}\right) = \exp\left(\beta_t\sigma_t - \kappa_E\sigma_{\ln E}\right) \tag{42}$$

The corrected resistance-side sensitivity factor thus follows as:

$$\alpha_R^* = \frac{\beta_t\sigma_t - \kappa_E\sigma_{\ln E}}{\beta_t\sigma_{\ln R}} = \frac{\beta_t + \kappa_E\alpha_E}{\beta_t\alpha_R} = \frac{1 + \alpha_E\frac{\kappa_E}{\beta_t}}{\sqrt{1-\alpha_E^2}} \tag{43}$$

In conjunction with (37) and (13), giving κ_E as a function of κ_S, and α_E as a function of resistance mechanism ($\sigma_{\ln R}$) and site ($\sigma_{\ln E}$, computed from the hazard slope k through (29) and $\sigma_{\ln E|S} = 0.3$), respectively, Equation (43) provides the value of α_R^* for any desired β_t, but it is still site-dependent through α_E and κ_E. The goal is to have $\gamma_R\gamma_E$ and thus γ_R^* as site independent as possible. This can be achieved by setting the derivative of (42) with respect to $\sigma_{\ln S}$ to zero. Doing this with the argument of the exponential, for simplicity, after substitution of $\kappa_E\sigma_{\ln E}$ by means of (37), one gets:

$$\frac{\partial\left(\beta_t\sigma_t - \kappa_E\sigma_{\ln E}\right)}{\partial\sigma_{\ln S}} = \frac{\partial\left(\beta_t\sigma_t - \kappa_S b\sigma_{\ln S}\right)}{\partial\sigma_{\ln S}} = \frac{\beta_t b^2\sigma_{\ln S}}{\sigma_t} - b\kappa_S = 0 \rightarrow \frac{\kappa_S}{\beta_t} = \frac{b\sigma_{\ln S}}{\sigma_t} \tag{44}$$

Averaging over the calibration space (i.e., between $b = 0.8$ and 1.2, $\sigma_{\ln S} = 0.414$ and 0.829, and $\sigma_{\ln R} = 0.2$ and 0.5) yields $\kappa_S/\beta_t = 0.79$. Inserting the latter value in (43), together with the corresponding average value of Equation (37) over the range of $\sigma_{\ln E}$ of interest, that is, $\kappa_E \cong 0.9\kappa_S$, and, finally, the average value $\bar{\alpha}_E = -0.91$, a constant value for $\bar{\alpha}_R^*$ is obtained:

$$\bar{\alpha}_R^* = 2.41 - 1.97 \cdot 0.79 = 0.85 \tag{45}$$

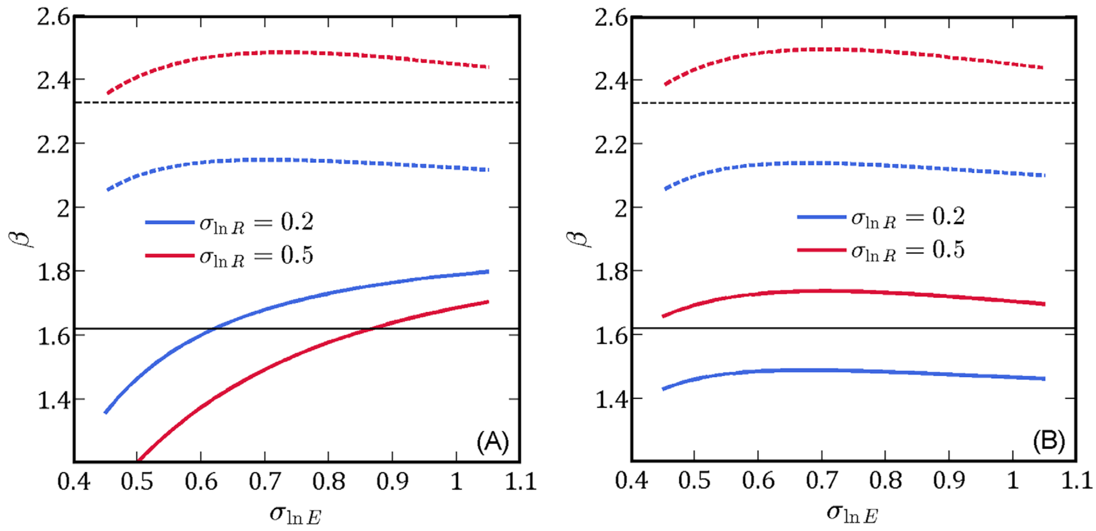

FIGURE 8 Reliability index of structures designed: (A) with $T_R = 1600$ years for different target reliabilities or (B) with the optimal T_R proportional to the target reliability index. Solid and dashed lines correspond to $\beta_t = 1.62$ and 2.33, respectively

This constant value, used in (41), leads to a scatter from β_t that is both low and sufficiently independent of the site seismicity, as it can be verified by looking at Figure 8. The figure shows the actual β versus $\sigma_{\ln E}$ for structures designed using γ_R^* from (41), $\sigma_{\ln R} = 0.2$ and 0.5 and two different target reliabilities 1.62 and 2.33. In panel (A), the same $\kappa_S = 1.87$ ($T_R = 1600$) is used, with ratios of κ_S to β_t equal to 1.17 and 0.80, respectively, and $\bar{\alpha}_R^*$ is computed from (45) replacing 0.79 with the latter values. In panel (B), instead, $\bar{\alpha}_R^* = 0.85$ and the value 0.79 is used for the ratio of κ_S to β_t, leading to $\kappa_S = 1.28$, or $T_R = 475$, while $\kappa_S = 1.87$ or $T_R = 1600$ is already almost the required one. It is apparent how in panel (A), the actual β shows marked non homogeneity with the site seismicity in the case where the T_R is not linked to the β_t. In panel (B), on the other hand, the scatter is within a $\Delta\beta = \pm 0.1$ for both cases.

In Europe, where safety of citizens is deemed a subject of exclusive national competence, national standardization bodies fix the values of the partial factors and the fractile of the seismic action. The previous results show that these values cannot be fixed independently of each other to meet a target reliability. If one wants to use a site-independent partial factor only on resistance, then T_R of the seismic action must be linked to β_t:

$$T_R = -\frac{L}{\ln \Phi(0,79\beta_t)} \tag{46}$$

Having the design seismic action linked to the target reliability is not an unprecedented novelty. This is what is prescribed also by the verification format in (38), with the only difference that the return period is related to the annual probability target, rather than to the reliability index in L years.

Based on (46), it is shown that the value of $\beta_t = 1.62$ in 50 years corresponds to $T_R = 475$ years, that is, the return period of the seismic action for the verification of the Significant Damage (SD) limit state. It is important that $\beta_t = 1.62$ be established as a target for the SD limit state, for the force-based approach to remain unaffected by the introduction of these partial factors for the displacement-based one (same T_R). In the future, if deemed appropriate, the β_t for the significant damage limit state might be changed, getting rid of the much discussed and almost untouchable $T_R = 475$ years.

As a final remark, it should be observed that the proposed solution is only one possible solution. It is chosen to be compatible with the general Eurocodes framework and to minimize impact on Eurocode 8. It should be mentioned that, in North America, the US Geological Survey provides design-basis seismic action as spectral acceleration with a return period of 2475 years, or 2% probability of being exceeded in 50 years. Such a return period, when used with a load factor of 1.0, places seismic action effect very close to the design point from FORM, with $\beta_t \cong 2.0$ for different sites, and for this reason is denoted as "risk-targeted,"[72] obviating the need for an adjustment to capacity to take site dependence into account. The proposed formulation has the benefit of making the relation of return periods and partial factors with target reliability explicit, allowing easy reliability differentiation.

4.8 Remarks on assessment of existing structures

The previous results extend directly to the case of existing structures, with just two minor changes. First, target reliability can be set to a value different from that accepted for the design of new structures. This can be done to consider a different reference period: a project-specific value of the residual service life $L < 50$ may be selected in an assessment and retrofit situation (e.g., for $L = 30$ years Equation (16), with the same annual probability, would yield $\beta_t = 1.5 < 2.33$). Even at parity of reference period, a maximum annual probability $p_1 > 2 \times 10^{-4}$ could be accepted in a trade-off between safety and other criteria. A typical example are structures of historical value, where conservation of architectural heritage is the driving factor.

The second change regards the probabilistic model for resistance. As shown in,[33] in the case of assessment Equation (34) can be rewritten as a function of the knowledge available at the time of assessment as:

$$\sigma_{\ln R}\left(\boldsymbol{KL}\right) = \sqrt{\sigma^2_{\ln r} + \sum \left(c_i s_{\ln x_i}\right)^2} \tag{47}$$

where $\boldsymbol{KL} = \{KLG, KLD, KLM\}$ is the vector collecting the KLs, ranging between 1 (minimum) and 3 (high), in the Geometry, construction Details and Materials knowledge categories, while $s_{\ln x_i} = r_i\, \sigma_{\ln x_i}$ denotes an estimate, based on limited testing/inspection, of the actual $\sigma_{\ln x_i}$. The ratio r_i depends on the KL attained on the knowledge category to which the i-th variable belongs (e.g., KLM for concrete strength f_c). The code[56] describes in detail the relation between the quantity and quality of information and the KL's.

5 CONCLUSION

In displacement-based design and assessment, an unreduced seismic action is applied to a model in a nonlinear analysis to determine action effects, in terms of deformations for ductile failure modes and forces for brittle ones. The second generation of Eurocode 8 refers to this method, previously confined to assessment only, as displacement-based approach, and presents it alongside the traditional force-based one, which employs force-reduction for ductility, redundancy and overstrength. It also provides the necessary resistance models to apply the approach to reinforced concrete, as well as steel and composite, frame and wall-frame structures: chord rotation models for members, and shear strength models for both members and joints. Finally, the second-generation Eurocode 8 declares for the first time a target reliability level for the near collapse (collapse prevention) limit state.

This paper discusses the formulation of partial factors to achieve that target reliability, or any other desired target reliability, when using the displacement-based approach.

The partial factors are derived for the seismic design situation following a traditional way of performing reliability-based code-calibration for other non-seismic design situations. Probability distributions for the maximum seismic action effect E over the service life L, and for the resistance R are introduced and discussed. The problem is then simplified to one with two Lognormal variables, for which closed-form expressions for the partial factors are available. It is shown that, when two factors are used, one on the resistance-side and the other on the load side, a uniform reliability is achieved over the calibration space (in terms of seismicity, with $\sigma_{\ln E}$ varying in an interval corresponding to values of k between 2 and 4; resistance models, with $\sigma_{\ln R}$ between 0.2 and 0.5; and exponent b of the demand-action relationship, i.e., degree of nonlinearity in response). It is also shown that the well-known formulation of partial factors with constant sensitivity coefficients by König and Hosser can be used also in the seismic design situation. Better constant values for the seismic case, however, are found to be $\bar{\alpha}_R = 0.42$ and $\bar{\alpha}_S = -0.91$. These results are compared with a previous proposal of a LRFD-like reliability-based safety-checking format for seismic design and are found to be in good agreement.

Once again, however, it is stressed that uniform reliability is achieved at the cost of a partial factor on the load side. Further, this factor requires knowledge of the logarithmic standard deviation of the action effect, $\sigma_{\ln E}$, which makes it site dependent. This approach is thus not in line with the current Eurocode framework, even though the necessity of two factors was clear already in the NBS report from 1980, where, for the seismic design situation, the proposed load combination included a 1.5 factor in front of the earthquake load effect[15]. A corrected site-independent resistance-side partial factor is thus proposed, which is compatible with the framework of Eurocode 8.

The proposed resistance-only partial factor requires only three inputs: the constant corrected resistance sensitivity factor $\alpha^*_R = 0.85$, the target reliability index β_t and the total logarithmic standard deviation of resistance $\sigma_{\ln R}$. The latter depends on the resistance model and has been already computed for all such models in the second-generation Eurocode 8, for both design and assessment situations. The only free choice is β_t.

A further important advantage of this format is that it easily seen to coincide with that proposed in the second-generation EN1990 and EN1992 for the assessment of existing (concrete) structures for other loads, once the total logarithmic standard deviation of resistance $\sigma_{\ln R}$ is replaced by the coefficient of variation V_R (the former is the correct one but, in any case, the two almost coincide numerically in the considered range of values). This, together with the fact that α_R^* is conveniently close to the traditional value 0.8, makes for a very easy to use format for practitioners.

A final remark is due on the return period of the seismic action, a most fundamental choice in the current uniform-hazard-based framework. While according to European rules, all safety-related choices should be left to National authorities, the ultimate decision-makers for the safety of their citizens, this does not imply that all safety-related parameters are independent and can be individually changed. For the corrected resistance-side partial factor to be site-independent and the discrepancy between target and actual safety to be acceptably low, it is shown that the return period of the seismic action needs to be specified as a function of the target reliability. In a way, this is bound to make life easier to those who need to specify these numbers, as once the β_t is chosen for any limit state and consequence class, the rest follows. Besides, this is also the trend in North America, where a different but conceptually equivalent choice has been made to increase the action, rather than correcting the resistance.

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DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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NOTES

[1] Note that the displacement-based approach in the Eurocode is just a set of performance requirements and indications about nonlinear analysis. How to achieve a structure that does satisfy the requirements is not a matter of concern for the code. This can be done by iteration. Direct displacement-based design, as proposed in,[23] is a possible way to do it without iteration.

[2] The lower reliability implied by current seismic design practice is a topic of ongoing discussion, with criticism directed at the code when (totally expected) collapses occur in the epicentral areas of strong earthquakes,[37] and resistance to increase design actions or introduce more severe construction details in "peace-time."

[3] The code format includes[2,36]: (1) the number of partial factors and load combination factors; (2) whether load partial factors should be material independent; (3) whether material partial factors should be independent of load type; (4) how to use the partial factors in the design equations rules for load combinations. This is also a problem of optimum, since a small number of partial factors would enhance ease of use, while a large number are needed to obtain economical and safe structures for a wide range of different types of structures. US codes have a smaller set of factors, whereas the Eurocodes employ a larger one.

[4] The designer has no control over mean resistance, only on some of the basic variables, e.g., like the amount of reinforcement A_s and the lever arm z, in the resisting flexural moment $\mu_{M_R} = \mu_{f_y} A_s z$. In a seismic design situation, things complicate a bit, since the variables that influence the mean resistance, affect in general also the demand distribution. Indeed, by changing reinforcement both strength and stiffness are changed,[15] and the response to a seismic excitation depends in the dynamic properties of the structure. This complication is not needed for the present more general discussion.

[5] Representative values are normally specified as predetermined lower fractiles of resistances and upper fractiles of the action effects. In the Eurocodes, for example, climatic actions are specified in terms of their 95% fractiles from the respective 50 years' distribution. Seismic action is specified differently in terms of its return period T_R, as discussed later.

[6]The values are approximated to one decimal digit in[39] and slightly violate the condition $\alpha_E^2 + \alpha_R^2 = 1$ to be direction cosines.

[7]The expression is obtained from the usual relation $\beta_L = -\Phi^{-1}(p_L)$, between reliability index in L years and the corresponding probability of failure, where the latter is obtained from the expression, consistent with assumptions made about the failure process being Poisson, $1 - p_L = \exp(-\lambda L) = [\exp(-\lambda)]^L = (1 - p_1)^L$. The approximate nature of

[8]These factors could be used as well for verifications carried out in global terms, for example, via nonlinear static analysis. The probabilistic model for the demand would remain essentially the same. What would change, however, is the probabilistic model of resistance, with the need for the quantification of the uncertainty in the global displacement limit for each limit state of interest.

[9]It may be worth noting that the logarithmic standard deviations $\sigma_{\ln S|m,r,i}$ and $\sigma_{\ln E|S}$ describe related but different record-to-record variabilities. The former is the variability in S between motions from different events with the same magnitude and source-to-site distance. The latter is the variability in the demand given S between motions from different events. If, for example, $S = S_a(T_i)$, while the structure is an oscillator with period $T_j \neq T_i$, then $\sigma_{\ln E|S}$ describes the variation of $S_a(T_j|S_a(T_i) = s)$, which is smaller than the unconditional $\sigma_{\ln S_a(T_j)}$ and different from $\sigma_{\ln S_a(T_j)|m,r,i}$.

[10]The choice of these return periods has a very limited impact on the numerical results in the paper, and none on the conclusions. The statement is based on rerunning all the calculations for different such choices (obviously not shown).

[11]For some models, the best functional form and statistical regression technique to minimize this CV have also been explored[57] leading to much reduced model errors.

[12]Use of a central value (mean or median), rather than a lower fractile, as a representative value for resistances, seems to be the international consensus for the seismic assessment situation, see, for example, also.[67] The advantages of this choice are discussed in[33] and in any case, the partial factors follows from this choice according to (12).

[13]Since the parameter k of the Fréchet distribution cannot be lower than 2.0 for the variance of the distribution to be finite, this is used as a lower bound for this comparison even though some values are lower than 2.0.

[14]Not considering code minima or gravity loads design is not a limitation or a contradiction. Design is always carried out to meet multiple requirements, one of which, at each site and for each structure, will turn out to be dominant. Gravity-loads and code minima dominate the design where seismicity is lower. Where the seismic action is not the governing one, application of the partial factors described in this paper would lead to a required capacity lower than that corresponding to the design for other actions, so member sizes will be larger than needed in this respect, resulting in higher than target (seismic) reliability index. To have a homogenous reliability for all sites, from low to high seismicity ones, one could raise the target reliability to higher values, for example, representative of the reliability implied by the code minima, artificially making the seismic design situation dominant also at lower seismicity sites. This seems not a reasonable thing to do.

[15]It should be noted that also seismic design practice in North America has not adopted such a factored seismic action effect for more than the three decades now, and the most recent codes have moved to risk-targeted actions, which essentially amount to using larger return periods, as already explained.

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