Partial Factors for Displacement-based Seismic Design and Assessment Consistent with the Second-generation of Eurocodes

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ABSTRACT: Second-generation Eurocode 8 introduces the displacement-based approach for design and assessment, where action effects are determined via nonlinear analysis, in terms of deformations and forces, for ductile and brittle failure modes, respectively. The code also provides all the necessary capacity models (e.g., chord rotation). The introduction of this approach requires ad hoc calibration of new partial factors to ensure that a reliability level consistent with that implied by traditional force-based design is achieved. This paper discusses the formulation of these partial factors. Probability distributions for the seismic action effect and the resistance, as well as target reliability for different limit states and consequence classes are introduced and discussed. The problem can be simplified to one with only two lognormal variables and a uniform reliability can be achieved over the calibration space when two factors are used, one on the resistance-side and the other on the load side. It is also shown that the well-known König and Hosser formulation of partial factors with constant sensitivity factors can be used also in the seismic design situation, with values tailored to the seismic case reflecting the larger weight of the action-related uncertainty. The problem with this otherwise good result is that a load partial factor must be used and, further, that its value is site dependent. This approach is not in line with current practice, thus a corrected site-independent resistance-side partial factor is proposed, which is compatible with the framework of Eurocode 8. Notably, the proposed format coincides with that proposed in the second-generation EN1990 and EN1992 for the assessment of existing (concrete) structures under non-seismic design situations, resulting in sure benefit for the final user.

Eurocodes, the set of harmonized European technical norms for the design and construction of civil works, are a cornerstone of the common market policy of the European Union. Their first generation was drafted and published between the end of the 1990s and the early 2000s. Their revision started in 2015 under mandate M515 from Technical Committee 250 of the European Committee for Standardization (CEN/TC250) and its foreseen completion is in 2024. The task had three goals: technical update, scope increase and enhancement of ease of use. Sub-committee 8 (SC8) oversees Eurocode 8 (EN1998), devoted to seismic design and assessment. In terms of technical update, one of the main changes in the second generation EN1998 introduced by SC8 is the displacement-based approach. This latter has been long recognized as a more truthful, less conventional way to determine the response and performance of a structure or geotechnical system to seismic ground motion excitation (Working groups, 1997).

Traditional force-based seismic design is carried out in terms of stress resultants determined via linear analysis under equivalent forces reduced to account for global ductility, redundancy and overstrength. The new approach, based on the explicit check of deformations, determined via nonlinear analysis under an unreduced seismic action, requires ad hoc calibration of new partial factors specifically derived to achieve a level of reliability consistent with that implied by traditional design. This in turn required making explicit the target reliability for the seismic design situation (first generation EN1998 did not declare such a target).

This paper illustrates the derivation of the partial factors for the displacement-based approach, the reliability targets, and the seismic intensity levels as a function of performance level (limit state, LS) and importance of the structure (consequence class, CC). It should be noted that a partial-factor safety format for the problem at hand has been proposed in the past by Cornell et al (2002). As pointed out in Franchin and Noto (2023), the partial factors proposed herein are equivalent to those in Cornell et al (2002), but are derived in a different way, consistent with the derivation of partial factors for non-seismic design situations in the other Eurocodes. As shown later, the proposed partial factors follow a well-known format, which is the same adopted for existing structures in non-seismic design situations first in *fib* Bulletin 80 (Caspeele et al, 2016) and lately by Eurocode 2 on concrete structures. Overall, the outcome increases consistency across the Eurocodes, and thus ease of use, and is more transparent with respect to source of uncertainty and target safety.

1. NON-SEISMIC DESIGN SITUATIONS

Code-calibration aims to provide an economic design with accepted minimum safety over a desired set of structures and design situations, defined as scope of application of the code (set of design cases). In reliability-based calibration, the safety is explicitly expressed in terms of target reliability index β_t. For any given code format (number of partial factors and load combination factors; whether load partial factors should be material independent and/or material partial factors should be independent of load type, etc. US codes have a smaller set of factors, whereas the Eurocodes employ a larger one) calibration is a minimization such as:

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

where β_i is the reliability index of the i-th design case, function of the partial factors collected in the vector $\boldsymbol{\gamma}$, and w_i the weight reflecting its importance to the overall design practice.

The link between β_i and $\boldsymbol{\gamma}$ is reliability analysis. To this end, probability distributions for the load/action effect and the resistance are needed. The former is known, while the latter must be derived for each design case, in other words, the action is given, while the resistance is designed. For the i-th case, the design equation (obtained taking the equal sign in the design inequality) provides the resistance fractile $R_{k,i}$ for given values of the partial factors γ_E and γ_R, and fractile of the seismic action effect $E_{k,i}$:

$$R_{d,i} = R_{k,i}/\gamma_R = \gamma_E E_{k,i} = E_{d,i} \tag{2}$$

$$R_{k,i} = \gamma_R \gamma_E E_{k,i} \tag{3}$$

where $E_{d,i}$ and $R_{d,i}$ are the design values of load effect and resistance (notation as per Eurocodes). Once $R_{k,i}$ is known, given the uncertainty on resistance σ_R, the distribution of the resistance random variable R_i is known and reliability analysis on the limit state function (LSF) $g_i = R_i - E_i$ yields β_i.

Instead of using optimization with the objective function in Eq. (1), partial factors can be derived in a different, approximate way, called Design Value Method (ISO, 1998), using the same Equation (2). Rather than assigning γ_E and γ_R to determine β_i, the latter is set equal to β_t and the design values of action effect and resistance are obtained as the coordinates of the design point according to:

$$R_{d,i} = F_{R,i}^{-1}[\Phi(-\alpha_R \beta_t)] \tag{4}$$

$$E_{d,i} = F_{E,i}^{-1}[\Phi(-\alpha_E \beta_t)] \tag{5}$$

where $F_{R,i}$, $F_{E,i}$ and Φ are the cumulative distribution functions (CDF) of resistance, action effect and the standard Gaussian variable, while α_R and α_E are the FORM sensitivity factors. The partial factors follow as R_k/R_d and E_d/E_k. This relatively crude method is very useful when E and R are both either Gaussian or lognormal (LN), because analytical solutions are available.

For example, if both are LN, one has:

$$\gamma_R = \frac{e^{\mu_{\ln R}+\kappa_R \sigma_{\ln R}}}{e^{\mu_{\ln R}-\alpha_R \beta_t \sigma_{\ln R}}} = e^{\alpha_R^2 \beta_t \sigma_t} e^{\kappa_R \sigma_{\ln R}} \quad (6)$$

$$\gamma_E = \frac{e^{\mu_{\ln E}-\alpha_E \beta_t \sigma_{\ln E}}}{e^{\mu_{\ln E}+\kappa_E \sigma_{\ln E}}} = e^{\alpha_E^2 \beta_t \sigma_t} e^{-\kappa_E \sigma_{\ln E}} \quad (7)$$

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 sensitivity factors are:

$$\alpha_R = \frac{\sigma_{\ln R}}{\sigma_t} \text{ and } \alpha_E = \frac{-\sigma_{\ln E}}{\sigma_t} \quad (8)$$

and the total logarithmic standard deviation is:

$$\sigma_t = \sqrt{\sigma_{\ln R}^2 + \sigma_{\ln E}^2} \quad (9)$$

To make derivation of material partial factors load-independent and that of load partial factors material-independent (a choice that facilitates code committee work), a further simplification is to approximate sensitivity factors, which depend on both sides of the equation, as shown by (8)-(9), with constant values. König and Hosser (1982) did exactly this, showing that in a large interval of σ_E to σ_R (or $\sigma_{\ln E}$ to $\sigma_{\ln R}$), taking $\bar{\alpha}_R = 0.8$ and $\bar{\alpha}_E = -0.7$ guarantees deviations $|\Delta\beta| \leq 0.5$ from β_t. This "simplified Level II method" was very influential for European normative developments. Much later, it also informed the first attempts made to derive a safety format for the assessment of existing structures in non-seismic design situations (Caspeele et al, 2016), which in turn led to the draft second-generation Eurocode on assessment of existing structures for non-seismic design situations (CEN/TC250, 2021). For these reasons, a similar approach is adopted herein to derive partial factors for displacement-based seismic design and assessment, as illustrated in the next sections.

2. SEISMIC DESIGN SITUATION

2.1. Time-variant reliability problem

The seismic reliability problem is time-variant: earthquake occurrence is a stochastic process and resistance, in general, also varies with time. Using the approach summarized in the previous section requires two assumptions. The first is that the process of earthquake occurrences is Poisson, while the second is that deterioration of resistance is neglected. The first assumption is acceptable at least for mainshocks, while the second is justified for a well-maintained structure, not exposed to previous damaging earthquakes during its lifetime.

Under the above assumptions, probability of failure over design life L can then be found by comparing the maximum seismic action effect in the reference period, function of the rate of earthquakes, with the time-invariant resistance. As shown in the next two sections, both capacity and demand can be modelled as LN.

2.2. Resistance (capacity)

Verification according to the displacement-based approach is carried out in terms of deformations (chord rotation in members) or forces (shear in members or joints and connections), for ductile or brittle failure modes, respectively. In EN1998, chord rotation models (Pangiotakos and Fardis, 2001) and a 'seismic' shear strength model (Biskinis et al. 2004) are present since the first-generation in Part 3 (CEN/TC250, 2005), devoted to assessment and retrofit. Second-generation EN1998 contains the latest updated version of these models for reinforced concrete (RC) members and covers extensively also steel and composite, as well as masonry and timber structures. The models are now included in Part 1-1 (CEN/TC250, 2022b), for displacement-based design of new structures, while rules to modify them for existing non-conforming members are given in Part 3 (CEN/TC250, 2022a).

All these models consist of semi-empirical functions of basic mechanical variables $\boldsymbol{x}$, $r(\boldsymbol{x})$, are derived to be unbiased and provided with the estimate of the coefficient of variation (CV) of the ratio of experimental to predicted resistances. They can all be modelled as LN, as it is customary for resistance variables (ISO 1998), as the product of $r(\boldsymbol{x})$ and of a unit-median LN variable ϵ_R with logarithmic standard deviation, $\sigma_{\ln r}$, function of the above CV (model error). As shown in (Franchin and Pagnoni 2018), a Taylor expansion of the natural logarithm of this product around the median of the basic variables $\hat{\boldsymbol{x}}$ yields a LN random variable with median $r(\hat{\boldsymbol{x}})$ and total logarithmic standard deviation, accounting also for variability in resistance resulting from uncertainty in the basic variables, given by:

$$\sigma_{\ln R} = \sqrt{\sigma_{\ln r}^2 + \sum\left(c_i \sigma_{\ln x_i}\right)^2} \tag{10}$$

which is a function of $\boldsymbol{x}$ through the coefficients c_i. In the process of drafting second-generation EN1998, Eq.(10) was used to evaluate $\sigma_{\ln R}$ for all resistance models over a large calibration space, with $\sigma_{\ln r}$ supplied for each formula $r(\boldsymbol{x})$ and typical values of $\sigma_{ln\, x_i}$ for the basic variables (ISO 1998). For illustration purposes, Table 1 reports these values for the ultimate chord rotation of RC members (values appropriate for assessment, related to non-conforming members, are higher and dependent on Knowledge Level in EN1998).

Table 1: Model error and total dispersion of ultimate chord rotation θ_u for code-conforming RC members.

Model	*Section*	$\sigma_{ln\, r}$	$\sigma_{ln\, R}$
θ_u	*Rectangular*	*0.20*	*0.22*
	Circular	*0.15*	*0.17*
	Other (e.g., hollow)	*0.20*	*0.21*

The low model error values for the chord rotation models are those based on improved regression techniques reported in (Grammatikou et al. 2018).

2.3. Maximum seismic action effect over the design life (demand)

Strong seismic ground motion excites the dynamic response of a structure well in the nonlinear range and in a frequency-dependent manner. To describe the seismic action effect E, the following model is adopted, after Cornell et al (2002):

$$E = aS^b\eta \tag{11}$$

where S is a scalar measure of intensity (IM) of the input ground motion, commonly a spectral ordinate at the fundamental period, a and b are the coefficients of a power-law accounting for the nonlinear relation between the input intensity and median effect, and η is a unit-median LN variable with dispersion $\sigma_{\ln E|S}$ modelling the so-called record-to-record variability (Shome et al, 1998).

Since Eq.(11) is a function of two random variables, S and η, the two-variable LSF becomes a three-variable LSF:

$$g = R - aS^b\eta \tag{12}$$

where R and η are LN. If S is taken to be the lifetime maximum spectral ordinate at a vibration period T, S_L, its CDF can be derived from the mean annual rate of S, $\lambda_S(s)$, or seismic hazard curve (SHC) under the Poisson assumption (probability of zero events):

$$F_{S_L}(s) = \exp(-\lambda_S(s)L) \tag{13}$$

which is not any known analytical distribution because the SHC is the result of an integration over all magnitudes and distances that has no closed form solution. An approximation is thus needed. Franchin and Noto (2023), show two such approximation, one as Fréchet, or extreme value (EV), Type II, the other as a LN. While EV models have long been known to be a good approximation (Cornell, 1968), the LN one is attractive because Eq.(12) would reduce again to a two-variable LSF with LN variables, for which the approximations presented in §1 hold. Since Franchin and Noto (2023) have already shown that the latter approximation is acceptable (by comparing reliability results in the three-variable LSF with both demand models), only the LN one is considered herein.

The LN fit can be written as:

$$\check{F}_{S_L}(s) = \Phi\left(\frac{\ln s - \mu_{\ln S_L}}{\sigma_{\ln S_L}}\right) \tag{14}$$

with parameters found performing a least square fit ($b_0 = -\mu_{\ln S_L}/\sigma_{\ln S_L}$ and $b_1 = 1/\sigma_{\ln S_L}$):

$$b_0 + b_1 \ln s + \epsilon = \Phi^{-1}\left(F_{S_L}\right) \tag{15}$$

It is intuitive and can be easily shown that $\sigma_{\ln S_L}$ is inversely proportional to the hazard slope k. The actual (inverse) proportionality coefficient depends on the fit interval. Numerical tests have shown that results are very limitedly impacted by this choice. Herein, the fit is carried out between $\lambda_S = 1/100$ and $1/2500$ years. Figure 1 shows four SHCs (spectral acceleration at four vibration periods), top, and the corresponding distributions of the lifetime maximum S_L, bottom, together with the LN approximation, for the site of L'Aquila, Italy. Black horizontal lines mark the fit interval, also highlighted with grey bands in the bottom panel.

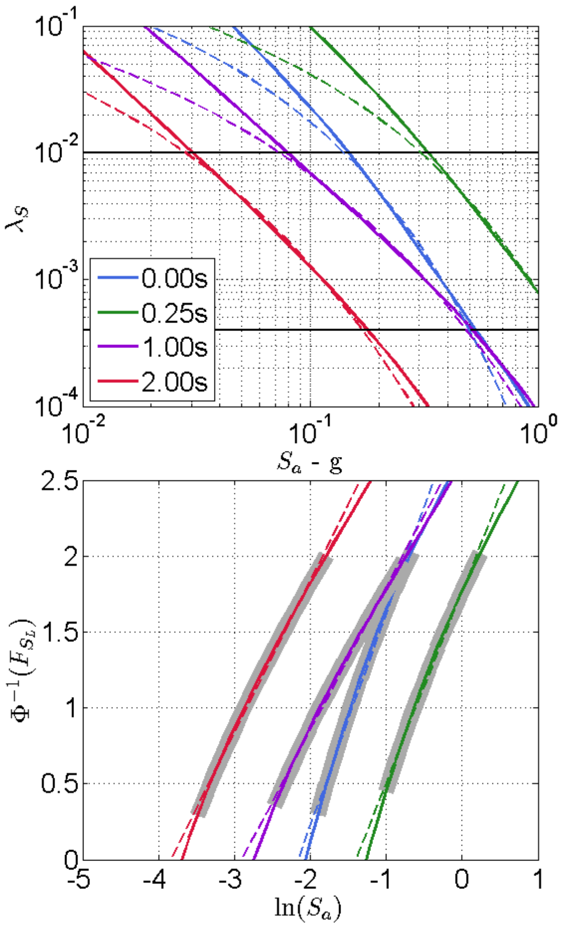

Figure 1: S_a at four vibration periods: seismic hazard curves (top) and LN approximation to the corresponding CDF of lifetime maximum S_L (bottom). Solid and dashed lines denote numerical curves and approximations, respectively. Site: L'Aquila, Italy.

If S_L is modelled as LN, then $E = aS_L^b\eta$ is also LN, with parameters:

$$\mu_{\ln E} = \ln a + b\mu_{\ln S_L} \tag{16}$$

$$\sigma_{\ln E} = \sqrt{b^2\sigma^2_{\ln S_L} + \sigma^2_{\ln E|S}} \tag{17}$$

and Eq.(6)-(9) hold.

2.4. Target reliability

The target reliability index β_t is needed for the considered reference period. If reliability over the lifetime L is considered, then $\beta_{L,t}$ can be specified directly, or indirectly in terms of the annual reliability. It should be noted that the latter option is a possibility also for computing the actual reliability, not just the target, i.e., computing the failure rate λ from the rate of all earthquakes times the probability of failure in one event, and using this with a Poisson assumption of failure occurrences. It involves, however, an approximation since failure events are not independent due to the common capacity and for this reason it has not been adopted. The approximation is inevitable, however, when establishing the target reliability if the starting point is the annual probability p_1. This was in fact the case for the draft EN1998, which specified $p_{NC,CC2,t} = 2 \times 10^{-4}$/year for the Near Collapse (NC) limit state of ordinary, e.g., residential structures (CC2), Table 2.

Table 2: Target reliability: p_1 by LS & CC ($\times 10^{-4}$).

LS	*CC1*	*CC2*	*CC3a*	*CC3b*
NC	*8,2*	*2,0*	*1,0*	*0,4*
SD	*24,4*	*11,3*	*8,0*	*4,6*
DL	*87,0*	*73,5*	*68,6*	*61,7*

Accounting for the numerical equivalence $p_1 = \lambda \leq 0.1$ (Der Kiureghian, 2005) one has:

$$1 - p_L = \mathrm{e}^{-\lambda L} = \left(e^{-\lambda}\right)^L = (1 - p_1)^L \tag{18}$$

and the reliability index in L years follows as:

$$\beta_L = -\Phi^{-1}(1 - (1 - p_1)^L) \tag{19}$$

Setting $p_1 = p_{NC,CC2,t}$ and $L = 50$ years, one gets $\beta_{NC,CC2,t} = 2.33$, as shown in Table 3. The tables show also values for the other CCs and LSs (Significant damage, SD, and Damage Limitation, DL). Apart from NC-CC2, where the starting point is $p_{NC,CC2,t}$, all other values have been established inverting Eq.(22), from the appropriate β_t. The latter have been established for each CC as a fraction (CC1) or a multiple (CC3a and b) of the corresponding CC2 value. They are suggested in the code as default values but European countries can change them as they wish, as they are in charge of safety in each respective territory. The only 'stiff' value is $\beta_{SD,CC2,t} = 1.6$, for reasons explained later.

Table 3: Target reliability: $\beta_{50,t}$ by LS and CC.

LS	CC1	CC2	CC3a	CC3b
NC	1,75	2,33	2,56	2,91
SD	1,20	1,60	1,76	2,00
DL	0,38	0,50	0,55	0,63

3. RELIABILITY ANALYSIS

FORM analysis yields identical results when carried out on the three-variable LSF in Eq.(12) or the two-variable LSF with LN variables with parameters in Eq.(16)-(17). Reference is thus made exclusively to the latter case in the following.

Consider the NC limit state and CC2: $\beta_{NC,CC2,t} = 2.33$ and, consistently with EN1998, $T_R = 1600$ years. Figure 2a and b show the values of α_R, α_E obtained with Eq.(8) and the partial factors obtained with Eq. (6)-(7), respectively. They have been determined over the range of the hazard slope k between 2 (high seismicity) and 4 (low). As explained in detail in Franchin and Noto (2023), all other parameters ($k_0, \sigma_{\ln S_L}$, etc) follow. Further values considered are $\sigma_{\ln R}$= 0.2 and 0.5 (representative of lower and upper bound uncertainty on resistance for the models in EN1998); b= 0.8 and 1.2 ($a = 1.0$, since it cancels out); $\sigma_{\ln E|S} = 0.3$ ($\hat{\eta} = 1$). Results are plotted versus $\sigma_{\ln E}$ (see Eq.(17)).

Figure 2a also reports average values $\bar{\alpha}_R = 0.42$ and $\bar{\alpha}_E = -0.91$ (dashed lines). These are equivalent to the well-known König and Hosser values $\bar{\alpha}_R = 0.8$ and $\bar{\alpha}_E = -0.7$, but reflect the much larger weight of $\sigma_{\ln E}$ with respect to $\sigma_{\ln R}$ in the seismic case. Using these 'seismic' values to determine γ_E, γ_R leads to the reliability in Figure 2c, with deviations $|\Delta\beta| \leq 0.1$ from β_t (obviously using the exact values in Figure 2b gives $\beta = \beta_t$). These γ_E and γ_R (values reported in panel c) would therefore be an excellent solution to the problem, were not for the fact that current practice (at least in EN1998) does not include a load-side partial factor for the seismic action.

What results would be obtaind if the seismic action effect was not amplified, i.e., $\gamma_E = 1.0$. Figure 3 shows that, with $\gamma_R = 1.5$ and 2.7, for $\sigma_{\ln R} = 0.2$ and 0.5, respectively, β is within $|\Delta\beta| = 0.2$ of $\beta_{NC,CC2,t} = 2.33$ and quite uniform, if not as close as in Figure 2c (recall, this is a single factor, not two), over the considered range of seismicity and nonlinearity in response. Is this a general result? What if T_R or γ_R were different? The next section answers this question.

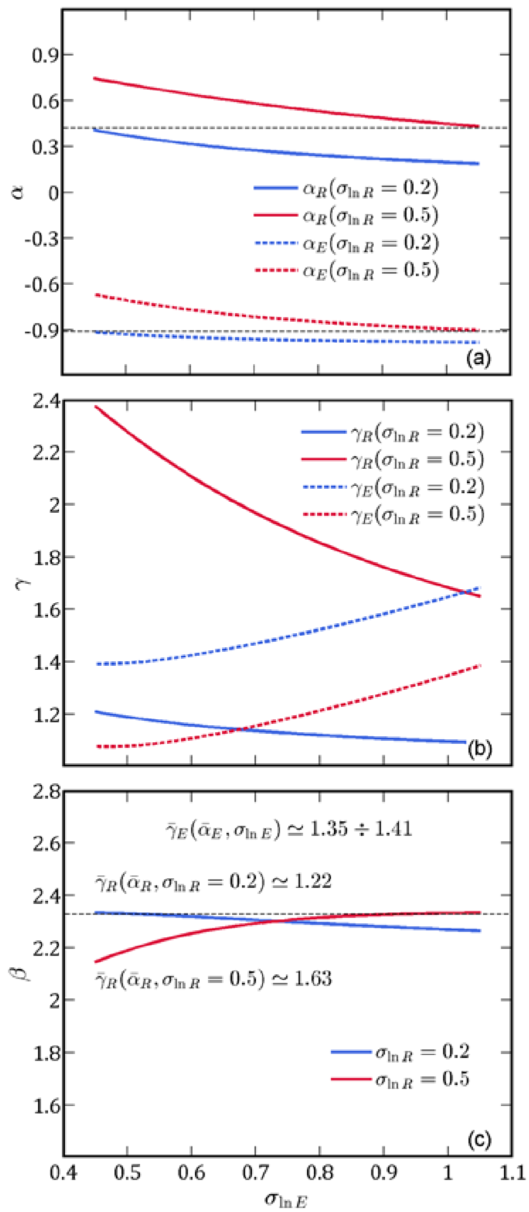

Figure 2: FORM sensitivity factors (a), corresponding partial factors (b), reliability index obtained with average values for α_E and α_R (c).

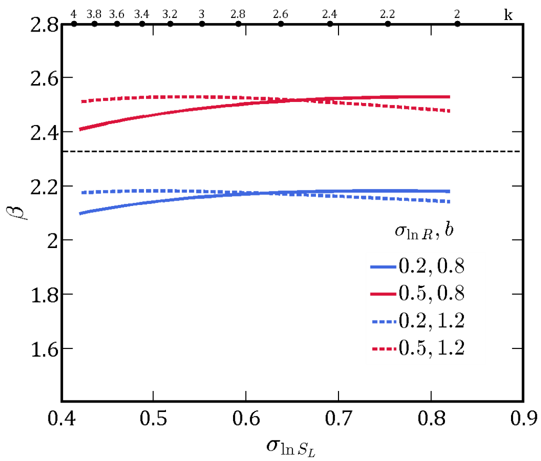

Figure 3: Reliability index obtained with $\gamma_E = 1.0$ and $\gamma_R = 1.5$ and 2.7, for $\sigma_{\ln R} = 0.2$ and 0.5, respectively.

4. SINGLE-FACTOR CODE PROPOSAL

To provide a reliability close to the target and approximately uniform over the range of interest of $\sigma_{\ln E}$ (i.e., k and b), a corrected resistance-only partial factor is proposed, starting from:

$$\gamma_R^* = \exp(\alpha_R^* \beta_t \sigma_{\ln R}) = \gamma_R \gamma_E \qquad (20)$$

Recalling Eq.(6)-(7) and, further, that $\alpha_E^2 + \alpha_R^2 = 1$ and $\kappa_R = 0$, $\gamma_R \gamma_E$ can be written as:

$$\gamma_R \gamma_E = \exp(\beta_t \sigma_t - \kappa_E \sigma_{\ln E}) \qquad (21)$$

which leads to:

$$\alpha_R^* = \frac{\beta_t \sigma_t - \kappa_E \sigma_{\ln E}}{\beta_t \sigma_{\ln R}} = \frac{\beta_t + \kappa_E \alpha_E}{\beta_t \alpha_R} = \frac{1 + \alpha_E \frac{\kappa_E}{\beta_t}}{\sqrt{1 - \alpha_E^2}} \qquad (22)$$

This expression for α_R^* depends on seismicity, through α_E and κ_E, which depend on $\sigma_{\ln S_L}$, and on β_t. Ideally, one would like α_R^* to be constant and site independent. Such an approximate value can be obtained as follows.

First, a useful relation between κ_E and κ_S is derived. Taking the ratio of two alternative expressions of the demand fractile E_k:

$$\frac{E_k}{a s_k^b \eta_k} = \frac{e^{\mu_{\ln E} + \kappa_E \sigma_{\ln E}}}{a \left(e^{\mu_{\ln S} + \kappa_S \sigma_{\ln S}}\right)^b 1} = \frac{e^{\kappa_E \sigma_{\ln E}}}{e^{\kappa_S \sigma_{\ln S} b}} = 1 \qquad (23)$$

since $\hat{E} = a \hat{S}^b$, i.e., $\exp(\mu_{\ln E}) = a \exp\left(\mu_{\ln S_L}\right)^b$, one gets:

$$\kappa_E = \kappa_S \frac{\sigma_{\ln S} b}{\sigma_{\ln E}} = \kappa_S \sqrt{1 - \left(\frac{\sigma_{\ln E|S}}{\sigma_{\ln E}}\right)^2} \qquad (24)$$

Next, the site-independence is aimed at setting to zero the derivative of (the argument of) Eq.(21), using (24) in the process:

$$\frac{\partial(\beta_t \sigma_t - \kappa_E \sigma_{\ln E})}{\partial \sigma_{\ln S}} = 0 \rightarrow \frac{\kappa_S}{\beta_t} = \frac{b \sigma_{\ln S}}{\sigma_t} \qquad (25)$$

Both Eq.(24) and (25) can be averaged over the already defined calibration space, getting $\kappa_E = 0.9\kappa_S$ and $\kappa_S/\beta_t = 0.79$. Replacing these values, as well as α_E with $\bar{\alpha}_E = -0.91$, in Eq.(22), leads to the sought constant value $\alpha_R^* = 0.85$. In other words, a single value of α_R^* can be used, provided that the design action for each LS is linked to the target reliability. This can be done based on Eq.(13), (14) and $\kappa_S/\beta_t = 0.79$:

$$\breve{F}_{S_L}(s_k) = \Phi(\kappa_S) = e^{-\lambda_S L} = e^{-\frac{L}{T_R}} \qquad (26)$$

leading to:

$$T_R = -\frac{L}{\ln \Phi(0.79\beta_t)} \qquad (27)$$

Figure 4 confirms that using an arbitrary seismic action leads to an average $\beta \neq \beta_t$, possibly non-uniform as a function of seismicity (for $T_R = 475$ years β decreases with increasing seismicity), while using an action determined by Eq.(27) ensures uniformity with deviations $|\Delta\beta| \leq 0.2$.

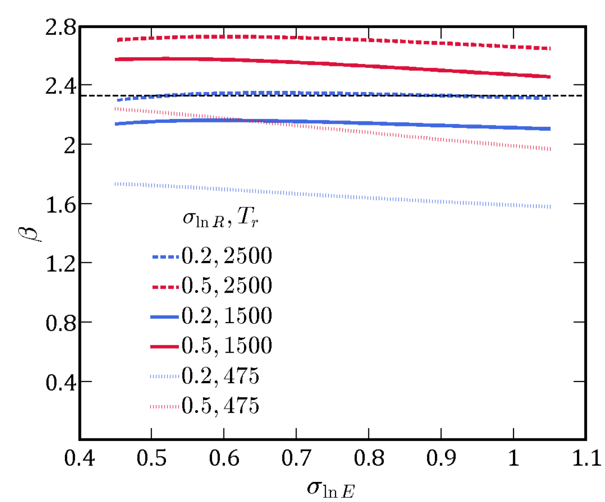

Figure 4: Reliability index obtained using partial factors according to Eq.(20), $\alpha_R^* = 0.85$ with T_R arbitrary (475, 2500) or determined with Eq.(27).

Table 4 reports the values of T_R obtained with $\kappa_s/\beta_t = 0.8$ and further rounded for code use. $T_R = 475$ for SD and CC2 is the exception, it is β_t in Table 3 derived from it inverting Eq.(27).

Table 4: T_R of the seismic action by LS and CC.

LS	*CC1*	*CC2*	*CC3a*	*CC3b*
NC	*600*	*1600*	*2500*	*5000*
SD	*275*	*475*	*600*	*900*
DL	*100*	*115*	*125*	*140*

5. CONCLUSIONS

The second-generation EN1998 introduces displacement-based design alongside the traditional force-based one. Further, unlike its first-generation counterpart, it declares explicitly the reliability targets. The paper presents a partial safety factor format that can be used consistently for the design of new structures or the assessment of existing ones, while being also compatible with the format adopted for non-seismic design situations. The new format is a step forward in terms of transparency on safety objectives and the actual uncertainty in the resistance models.

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