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Calibration of partial factors for wind action: an application to the Italian wind climate

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ABSTRACT: The draft second generation Eurocodes have started to appear in the last couple of years. Among these, the Eurocode 1 Part 1-4 is dealing with the design of structures to wind action. In view of the opportunity of updating National Annexes (NAs), in this paper the attempt is made at calibrating wind load factors for Italy, also investigating the reliability of the current Italian extreme wind map. Overall, it is shown that load factor currently adopted by the Eurocodes and by the Italian NAs needs to be increased. On the other hand, the Italian extreme wind map should be revised to reduce the gap in terms of reliability level.

1. INTRODUCTION

In the last couple of years, the draft second generation Eurocodes have started to appear, and the process of their approval is expected to be completed in two more years. As to structural reliability aspects, some critical issues were detected, which triggered additional research in support of the new documents. Taking the Eurocode as a case study in the calibration of existing semi-probabilistic design codes, Köhler et al. (2019) stated the need for a revision of the load partial factors currently in use, showing that they "*seem too high for permanent loads*" and "*too low for variable loads*". As a consequence, load partial factors from EN 1990 (CEN, 2002) prove to lead to an average reliability level not always matching the target value, and in particular in the case of wind actions.

The spatial variation of extreme wind climate influences the reliability of structures. Therefore, a calibration of codes at the National level should include: (1) a wind hazard map providing the characteristic value of the wind speed (or velocity pressure), usually associated with a return period $T = 50$ yrs; (2) an equation accounting for return periods other than 50 yrs, and (3) a set of partial factors for the evaluation of the design value of the wind action. Current version of EN 1991-1-4 (CEN, 2005) does not provide a unitary wind hazard map for Europe, as design wind speeds are listed among Nationally Determined Parameters (NDPs). Therefore, their quantification is deferred to National Standardization Bodies (NSBs) through inclusion in the National Annexes (NAs). As per specific request of CEN, a merging of the existing national extreme wind maps is included in prEN 1991-1-4:2021 (CEN, 2021; Ricciardelli, 2023). Yet, many inconsistencies exist, among which is the heterogeneity of the wind speed datasets and that of the analysis techniques; these manifest themselves through discontinuities across the borders. On the other hand, a probability factor is considered in current Eurocode 1 which parameters are calibrated based on a coefficient of variation for the annual maximum velocity pressure equal to 0.23. In addition to that, since the pre-standard ENV 1991-1 (CEN, 1994), the choice is made to adopt a single load partial factor $\gamma_Q = 1.5$ for all variable loads.

Publication of the second-generation Eurocodes will be an opportunity to revise NAs, by updating wind maps based on a unified methodology for data processing and assessment of design wind speed, but also for calibrating partial factors at the National level, based on the specific wind climate. As to the latter issue, the current probabilistic model for wind actions could prove not to match the physical model adopted by EN1991-1-4 (CEN, 2005) and an update might be appropriate (Kasperski and Geurts, 2005; Picozzi, 2023). Moreover, it is common thought that the choices made within codes for calibration of model parameters lead to mainly conservative estimates of actions, i.e. to hidden safety (Teichgräber et al., 2022). However, this seems not to be the case for wind actions, for which a bias larger than 1 exists (Geurts et al., 2005) and the use of advanced models leads to lower reliability levels (Teichgräber et al., 2022).

Main purposes of current work are (1) the assessment of the actual reliability level of structures subject to wind action considering the Italian wind climate and (2) an attempt to calibrate partial factors for wind action according to EN 1990 (CEN, 2002) and to EN 1991-1-4 CEN, 2005). In detail, focus is on the effects of uncertainties associated with the existing Italian extreme wind map, and on the variability of the reliability level, therefore of the required partial factors due to variation of the extreme wind climate across the country.

2. METHODOLOGY

2.1. Limit State Function

The assessment of structural reliability is performed by estimating the reliability index β or, alternatively, the probability of failure p_f. For such purpose, it is required the definition of a generic Limit State Function accounting for the resistance and for the effects of actions on the structure. This is made explicit by:

$$g(\boldsymbol{X}, z) = zX_R R - X_E\{(1-a_Q)G + a_Q\theta_Q Q\} \quad (1)$$

where p is the design parameter, R is the material strength, X_R and X_E are the model uncertainty for resistance and load effects, respectively, a_Q is a factor representing different proportions between dead loads G and wind action effects, the latter expressed as the product between a time-variant part Q and a time-invariant component θ_Q. Dead loads G in Eq. (1) are expressed as:

$$G = a_G G_s + (1-a_G)\, G_p \quad (2)$$

proportions between self-weight G_s and permanent

Table 1: Probabilistic models (based on JCSS, 2001).

Variable	X	CDF	$E[X]$	$V[X]$	X_k	γ_X[a]
Steel						
Failure mode	X_{R1}	$\mathcal{LN}$	1.00	0.05	1.00	1.00
Strength	R_1	$\mathcal{LN}$	1.20	0.07	1.00	
Self-weight	G_{s1}	$\mathcal{N}$	1.00	0.025	1.00	1.35
Concrete						
Failure mode	X_{R2}	$\mathcal{LN}$	1.00	0.15	1.00	1.50
Strength	R_2	$\mathcal{LN}$	1.00	0.10	0.73	
Self-weight	G_{s2}	$\mathcal{N}$	1.00	0.05	1.00	1.35
Load effects	X_E	$\mathcal{LN}$	1.00	0.10	1.00	
Permanent Loads	G_p	$\mathcal{N}$	1.00	0.10	1.00	1.35
Wind Load (Time-Variant)	Q	$\mathcal{G}$	1.35[b,c]	0.17[c,d]	0.92	1.50
Wind Load (Time-Invariant)	θ_Q	$\mathcal{LN}$	0.80	0.26	1.00	

[a] EN1990 values;
[b] equal to 1.00 when neglecting downsampling error;
[c] average values based on the results from 40 sites;
[d] 50yrs maxima.

loads G_p. To derive the design parameter p, the design equation 6.10 of EN 1990 (CEN, 2002) is used leading to:

$$p = \frac{\gamma_M}{\theta_{R,k} R_k} \left\{ (1 - a_Q) G_k + a_Q \gamma_Q \theta_{Q,k} Q_k \right\} \quad (3)$$

where $G_k = a_G \gamma_{G,s} G_{s,k} + (1 - a_G) \gamma_{G,p} G_{p,k}$.

The probabilistic models used for the assessment of reliability index are summarized in Table 1 together with the corresponding characteristic values and the partial factors γ given in Eurocode 0. Note that instead of using the recommended partial factors, it could be relevant to use the design value format method to determine the partial factors, accounting for the uncertainties and bias. Concerning the resistance model, two materials are considered: structural steel characterized by steel bending as failure mode and which strength R_1 is related to the steel yielding; concrete failing in compression mode and which strength R_2 is therefore related to the concrete compression.

2.2. Wind action model

According to the Wind Loading Chain of Davenport (1961), the wind action in EN 1991-1-4 is expressed by:

$$w = q_b \cdot c_e \cdot c_p \cdot c_s c_d \quad (4)$$

where q_b is the 10-min averaged velocity pressure in standard conditions; c_e is the exposure coefficient accounting for all the deviations from the standard conditions, i.e. orography, topography, and ground surface roughness of the environment surrounding the construction; c_p is the pressure coefficient accounting for the (static) aerodynamic interaction; $c_s c_d$ is the structural factor accounting for the effects of the construction dimensions (c_s) and the dynamic effects (c_s) of wind action.

The uncertainty model for wind action must account for the uncertainties in all the four terms of Eq. (3). Except for q_b, the probabilistic model for time-invariant component of wind action used in current work is based on the Probabilistic Model Code (JCSS, 2001). In Section 2.3 uncertainty in q_b is specified.

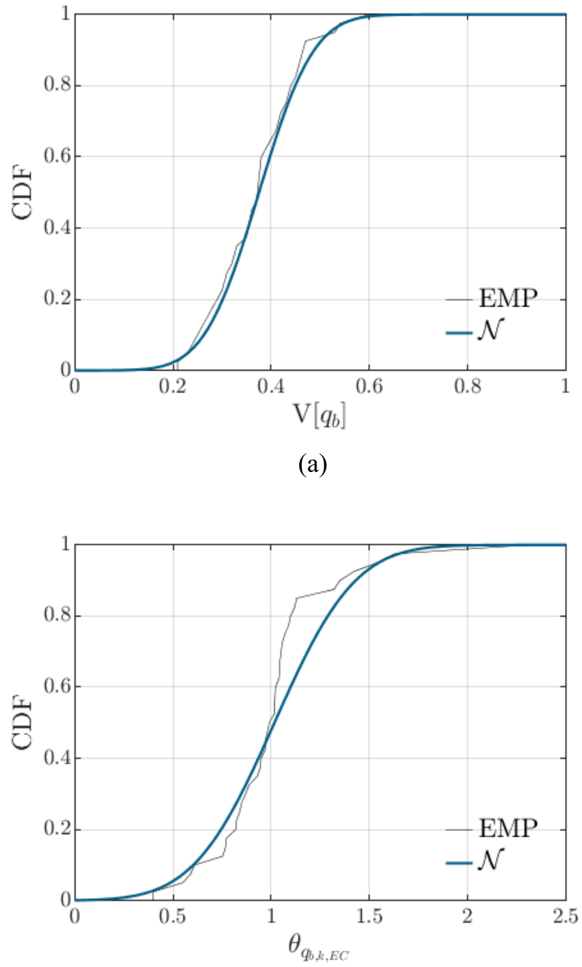
2.3. Uncertainty in the velocity pressure

The work of Ballio et al. (1991a, 1991b, 1999) is the basis of the current Italian extreme wind map. It describes the methods used for the assessment of the design wind speeds and provides the coefficients of variation of annual maximum values, thus allowing the modelling of time-variant uncertainty of wind actions. Both Extreme Value Analysis (EVA) and the Parent Population Methods (PPM) are used, showing the tendency of EVA to provide larger return wind speeds with respect to PPM. However, this aspect is neglected in current work and only results from EVA are used, being it the most common method for assessment of return wind speeds (or velocity pressures).

Based on the results from 40 meteorological stations, it is shown that the extreme wind climate – representing the natural variability of yearly maximum wind speed or velocity pressure – for Italy can be represented by a Gumbel ($\mathcal{G}$) distribution having a coefficient of variation $V[q_b]$ranging between 0.21 and 0.60, with a mean value of 0.37. The empirical Cumulative Distribution Function (CDF) of $V[q_b]$ for the considered sites is shown in Figure 1a. It can be noted that it may be summarized by a Normal ($\mathcal{N}$) distribution with coefficient of variation equal to 0.23.

The availability of 50-yrs return wind speeds, therefore of the characteristic values $q_{b,k}$ for each site, allows uncertainty $\theta_{q_{b,k,EC}}$ to be assessed. To this aim, first the characteristic values $q_{b,k,EC}$ of velocity pressure are estimated according to the Italian NA to EN 1991-1-4 (UNI, 2010); then, the ratios $\theta_{q_{b,k,EC}} = q_{b,k}/q_{b,k,EC}$ are evaluated for single sites. In Figure 1b, the empirical CDF of $\theta_{q_{b,k,EC}}$ is reported, showing values ranging between 0.40 and 1.63 (a value equal to 2.31 is considered as outlier, thus neglected), with a sample mean equal to 1.01 and coefficient of variation equal to 0.28.

Most of datasets used for the drafting of the Italian extreme wind map are 3-hrs downsampled, i.e. they collect observations of 10-min averaged wind speed stored every 3 hours. It is proved that the effect is an underestimation of extreme wind speeds, thus of their characteristic value (Picozzi et al., 2022). Based on analyses of 26 Italian sites, in Akbaba et al. (2022) an underestimation in wind speed is evaluated, ranging between 10.6% and 27.6%, with a mean value of 14%, corresponding to a bias of 1.16 and a coefficient of variation equal to 0.04. In current work, an additional error in estimating $q_{b,k}$ is therefore considered, on average equal to $1.16^2 = 1.35$.



(a)

(b)

Figure 1: Coefficients of Variation of yearly maxima (a) and uncertainty in the characteristic velocity pressure $q_{b,k,EC}$ *(b).*

2.4. Calibration of wind action partial factors

The assessment of partial factors can be seen as an optimization process, where the reliability indexes $\beta_i(\boldsymbol{\gamma})$ – evaluated for a number of design situations and assuming a set of partial factors $\boldsymbol{\gamma}$ – are as close as possible to the target reliability index β_t. Being the aim of current work the calibration of partial factor for wind action, the following optimization problem can be formulated:

$$\gamma_Q = \arg\min \sum_{i=1}^{L} w_i\left(\beta_i(\gamma_Q) - \beta_t\right)^2 \quad (5)$$

where the L design situations derive from the choice of different materials and different weighting factors a_G and a_Q.

In current work, the choice is made to keep the analyses separate for the different materials. Moreover, the same importance is considered for the different situations derived by adoption of different factors a_Q and a_G. According to Köhler et al. (2019), the weighting factor a_G for dead loads is assumed to range from 0.6 to 1.0. Instead, for the weighting factor a_Q, the range 0.3 to 0.8 is investigated for structural steel structures, while the range 0.1 to 0.7 is considered for concrete structures. For both a_G and a_Q factors, a step of 0.1 is adopted, giving rise to 30 design situations for steel and 35 design situations for concrete.

Availability of statistical models for yearly maxima of different Italian sites allows calibration to be made for single sites. In this case, Eq. (5) holds and the velocity pressure for each j-th site must be modelled through a Gumbel distribution having coefficient of variation $V[q_{b,j}]$ and bias equal to the error $q_{b,k,j}/q_{b,k,EC,j}$. As an alternative, the optimization can be done by considering the ensemble of 40 sites as contributing to the definition of the design situations portfolio. In this case, 30x40 design situations are investigated for structural steel, and 35x40 design situations are investigated for concrete.

To investigate the effects of the uncertainties in velocity pressure, three different probabilistic models are considered for wind action. In Model 1, only the aleatory uncertainty in velocity pressure is considered. Therefore, the time-variant uncertainty of wind action (Q) for each j-th site is modeled with a Gumbel distribution having bias equal to 1.0 and coefficient of variation $V[q_{b,j}]$.

In Model 2 the uncertainty in the evaluation of the design velocity pressure $q_{b,k,EC}$ is also considered. Consequently, Q for each j-th site is modeled with a Gumbel distribution having bias equal to $q_{b,k,j}/q_{b,k,EC,j}$ and coefficient of variation $V[q_{b,j}]$.

Finally, in Model 3 the bias due to downsampling is added to the previous uncertainties. Within this model, the velocity pressure for each j-th site is thus modeled as a random variable having a Gumbel CDF with bias equal to the product between $q_{b,k,j}/q_{b,k,EC,j}$ and the mean error due to downsampling, and coefficient of variation $V[q_{b,j}]$.

3. RESULTS

3.1. Current reliability level

The current reliability level of a structure to wind actions is first investigated for all the design situations described in Section 2, by adopting a load factor for wind actions $\gamma_Q = 1.5$. In the following, all the reliability indices refer to a period $T = 50$ yrs.

In Figure 2, the variation of the reliability index β is shown for steel (S) and concrete (C) structures and for three different uncertainty models of velocity pressure.

When only the natural variability of maximum velocity pressure is considered (Model 1), it is shown that the reliability index of steel structures – indicated as S1 in the figure – is on overage equal to 2.92, therefore lower than the target reliability index $\beta_t = 3.8$ prescribed by Eurocode 0 (CEN, 2002). Instead, it is on average equal to 4.47 for concrete structures (C1 in the figure), therefore larger than β_t. Such results are in agreement with those obtained by Köhler et al. (2019). As for the variability of the reliability index deriving from the different design situations, a Coefficient of Variation is evaluated equal to 0.11 for the S1 model, and equal to 0.09 for the C1 model.

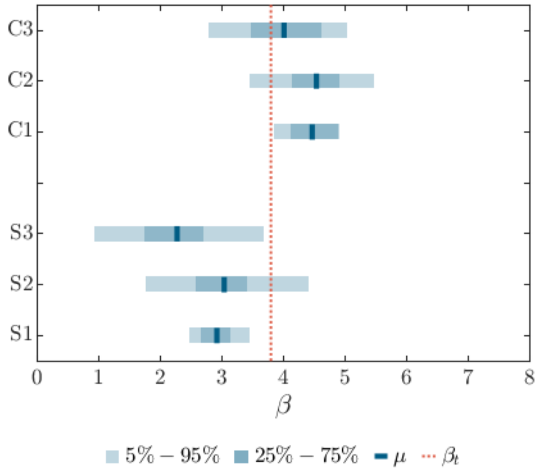


Figure 2: Variability of the reliability index (50 yrs) for the investigated design situations when $\gamma_Q = 1.5$ is adopted for wind actions, according to Eurocode 0 (CEN, 2002).

The effects of uncertainty in assessing the characteristic value $q_{b,k}$ of the velocity pressure can be seen by adopting the Model 2. In Figure 2, the results are denoted as S2 and C2 for steel and concrete structures, respectively. Since the bias in $\theta_{q_{b,k,EC}}$ is close to 1.0, it is shown that the reliability index β is on average close to that estimated adopting Model 1. However, the main effect of such uncertainty is the increased variability of β among the different design situations. Indeed, a coefficient of variation is evaluated equal to 0.25 for steel structures, and equal to 0.13 for concrete structures.

Finally, when also the error due to downsampling is accounted for in the analyses, then a reduction is shown of the reliability indices – as a consequence of the underestimation of $q_{b,k,EC}$ –

Table 2: Statistics of the reliability index (50 yrs) when $\gamma_Q = 1.5$ is adopted for wind action, according to Eurocode 0 (CEN, 2002).

	S1	S2	S3	C1	C2	C3
$E[\beta]$	2.92	3.04	2.27	4.47	4.54	4.01
$V[\beta]$	0.11	0.25	0.35	0.09	0.13	0.18

and an increase of the variability of β among the different design situations. The results of the adoption of Model 3 for wind action are denoted in Figure 2 as S3 for steel structures and C3 for concrete structures.

A summary of the statistics of the reliability index estimated when using the load factor for wind action $\gamma_Q = 1.5$ is shown in Table 2.

3.2. Partial factors for wind action

The reliability indices discussed in Section 3.1 for the Italian wind climate suggest the need for a new calibration of partial factors for wind actions. In current section, the attempt is made to calibrate them for the three different statistical models identified in Section 2.3, in order to show the effects of the different sources of velocity pressure uncertainty. The results are summarized in Table 3.

When the optimization process of Eq. (5) is applied, then an optimum value $\gamma_{Q,opt}$ of the partial factor for wind action is found equal to 2.25 for steel structures, and equal to 1.05 for concrete structures when probabilistic Model 1 is adopted. As expected, when uncertainty $\theta_{q_{b,k,EC}}$ is considered, i.e. when probabilistic Model 2 is investigated, the wind partial factors are close to those estimated in the case of probabilistic Model 1. Instead, $\gamma_{Q,opt}$ grows up to 3.05 for steel structures and to 1.50 for concrete structures when the bias due to downsampling is accounted for in the analyses, i.e. when probabilistic Model 3 is considered.

Calibration of the partial factor γ_Q for single sites allows its variability to be estimated. Overall, it is shown that the sample means $E[\gamma_Q]$ of partial factors calibrated for single sites are in quite good agreement with the optimum values $\gamma_{Q,opt}$. Their Coefficient of Variation $V[\gamma_Q]$ is as low as 0.04 when the Model 1 is used, i.e. when only the variability of annual maxima is considered (S1 and C1 in Table 3). However, when also uncertainty in the assessment of the characteristic value $q_{b,k}$ of the velocity pressure is considered (Model 2), then it grows to 0.29 in the case of steel structures (S2 in Table 3), and to 0.35 in the case of concrete structures (C2 in Table 3). The same variability is observed when using Model 3.

Table 3: Results of the wind partial factors calibration.

	S1	S2	S3	C1	C2	C3
$\gamma_{Q,opt}$	2.25	2.20	3.05	1.05	1.00	1.50
$E[\gamma_Q]$	2.27	2.23	3.09	1.06	1.04	1.54
$V[\gamma_Q]$	0.04	0.29	0.29	0.04	0.35	0.33

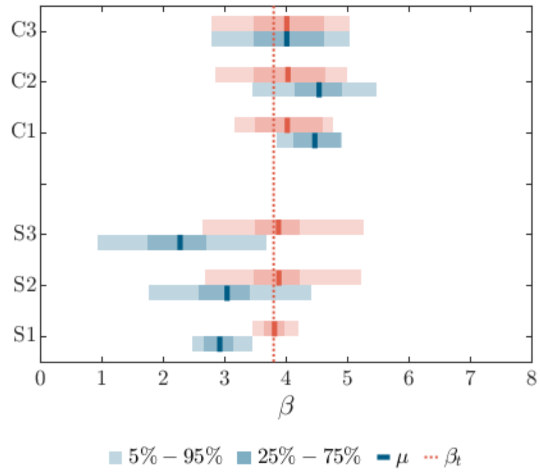


Figure 3: Variability of the 50-yrs reliability index before (in blue) and after (in red) the calibration.

In Figure 3 the results of the calibration of wind partial factors are shown in terms of reliability index β (red values). It is shown that such calibration allows the mean reliability index β to get closer to the target value β_t. Note that since the optimization problem aims at minimizing the difference between β and β_t for all the design situations, then it is not obvious that the mean value of β corresponds to the target one.

After calibration, the variability of β remains and this is mainly associated with the variability of the ratios $q_{b,k}/q_{b,k,EC}$. Its reduction may only take place through a redrafting of the extreme wind map, which zoning should aim at minimizing such uncertainty.

4. CONCLUSIONS

The aim of current work was the investigation of the effects of uncertainty in velocity pressure on the structural reliability and on the load factors calibration for wind action. The Italian National Annex to the Eurocodes was considered as a case study.

Two main results were observed. First, when adopting the current value of partial factor for variable actions, then the reliability level of steel structures resulted lower than the target value, while it was larger in the case of concrete structures. Accordingly, the calibration of partial factors showed the need for increasing the wind load factor for steel structures up to 3.1, while the current value 1.5 could be acceptable for concrete structures.

On the other hand, the high variability of the reliability index for the analyzed design situations highlighted the need of redrafting the Italian extreme wind map. This must be done with the aim of reducing the scatter of error in the evaluation of the characteristic wind speed or velocity pressure.

Current work is limited to the investigation of steel and concrete structures, whose analyses were carried out individually. However, in view of the calibration of Italian structural code, a single load factor must be calibrated for wind action accounting for all the possible design situations, thus including all the structural material and the corresponding failure modes.

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