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Performance-based wind design of lattice towers

Summary of the PhD dissertation

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1 Introduction

1.1 Problem statement

Large uncertainty is involved in the characterization of the wind load, which is one of the most significant actions influencing the integrity of several structural types. Compromise has to be found in the structural design process between safety considerations and extra costs of overdesign. Therefore, it is of utmost importance to examine the related modelling assumptions and to make sure that the model aptly describes reality. The design formulae of structural design codes or standards, such as the Eurocodes, are based on the semi-probabilistic method using partial safety factors on both the resistance and action sides where design values should be conservative estimates. Some studies pointed out that applying one single partial safety factor for all unfavourable environmental variable actions may not be reasonable since these environmental actions originate from different physical phenomena and are represented by different models. According to previous research and our observations, extreme windstorms caused significantly less structural stress or even damage in telecommunication lattice towers than expected. Therefore, the following questions are sought to be answered: What causes the discrepancy between the stresses calculated according to the Eurocode (EN 1993-3-1), and the measured values? Which parameter of the model can be adjusted to improve the alignment of the model with the real behaviour? The drag coefficient for wind-structure interaction in the equivalent static method, as per EN 1993-3-1, is originally derived from wind tunnel tests at a reduced scale of 1:6, conducted in 1977. It appears that the formula is more aligned with drag coefficients observed at the model scale rather than in full-scale conditions. Furthermore, in the probabilistic modelling of structural performance, the consideration of the parameter estimation uncertainty in extreme wind speed models is often overlooked, which may have substantial impact on the modelling of extreme environmental actions.

1.2 Motivation

The advancements in computer technology have paved the way for alternative design procedures that utilize fully probabilistic approaches and simulation techniques. Performance-based design (PBD) incorporates a comprehensive risk assessment that goes beyond structural safety considerations to include the cost of consequences. The application of Performance-based wind engineering (PBWE) methodology, which allows for the incorporation of measurements and real-time monitoring of the structure, has the potential to reduce uncertainties and prevent excessive overdesign in structures. The PBWE is gaining increasing popularity as it

effectively addresses various wind engineering problems, making it a widely adopted approach in the field. Therefore, the presented research work focuses on the integration of the Performance-based wind design (PBWD) framework with a monitoring system and finite element model to assess the structural reliability. Moreover, Bayesian analysis is introduced, an effective approach for combining various sources of information, including diverse measurements and probabilistic models for wind action, in order to determine the basic wind velocity. This integration reflects the development of the second-generation of the Eurocodes, aiming to incorporate the latest knowledge and enhance harmonization across different regions. The revision of EN 1991-1-4 includes deriving basic wind velocity from measurements at meteorological stations, utilizing probabilistic models for wind actions, and obtaining design parameters from wind tunnel tests and numerical simulations.

2 Research strategy

First, a thorough literature review examines prior research in the field of wind engineering, focusing on the impact of extreme wind conditions on towers. This includes modelling aspects related to the basic wind velocity, tower aerodynamics, and drag coefficient characterization. The research spans various disciplines, such as meteorology, probability theory, fluid dynamics, and structural analysis.

A Performance-based design framework is then introduced for analysing the response of telecommunication lattice masts to extreme wind conditions. This framework is applied to investigate a freestanding lattice tower situated in the suburban area of Sződliget, central Hungary. The tower is equipped with cup anemometer wind speed sensors at the top and middle, as well as strain gauges on the bottom chord members to measure its response to wind loads. Additionally, data from Sződliget is augmented with information from other meteorological stations under the Hungarian Meteorological Service. Various procedures and parameters for extreme wind speed distribution models are used to estimate the basic wind velocity from different measurement datasets. Bayesian inference techniques are employed to synthesize information from these diverse sources.

The wind-structure interaction is modelled using the equivalent static method, following the EN 1993-3-1 standard. This involves the utilization of the drag coefficient, and the results are compared with drag coefficients obtained from wind tunnel experiments. A finite element model with beam elements is developed to simulate the structural response. This model considers self-weight

10 Future research work

The drag coefficient stands as an area for potential future research. The drag coefficient for wind-structure interaction in the equivalent static method, as per EN 1993-3-1, is originally derived from wind tunnel tests at a reduced scale of 1:6, conducted by the National Maritime Institute (NMI) in the UK in 1977. Consequently, it appears that the formula provided by EN 1993-3-1 is more aligned with force coefficients observed at the model scale rather than in full-scale conditions. The standard distinguishes between circular members in the subcritical and supercritical range, setting the transition point at $\mathrm{Re} = 4 \cdot 10^5$ while conservatively omitting the critical range. Considering the industry's goal of conserving resources and reducing costs through the acquisition of precise load data, there is a need for a closer examination of drag coefficients obtained from scaled tests in the UK during the 1970s. The challenge arises in determining the Reynolds number at which drag coefficients significantly decrease. To address this, further investigations through full-scale wind tunnel tests and computational fluid dynamics simulations are essential.

Another significant area for future research, particularly due to high uncertainty, is the basic wind velocity. In line with European Union directives, the Hungarian Meteorological Service (HMS) has made its observation and measurement data (from 2002 onwards) freely available for use through an open data server 2021. The database allows for an in-depth examination of the fundamental wind speed in various regions across Hungary. A detailed comparison between the traditional method and the block method, along with the peak-over-threshold method, can be made using the data. Additionally, the implementation of the Offset Elliptical Normal model, as proposed by Cook (2019), could offer a more physically feasible depiction of joint and marginal wind speed and direction distributions, presenting a potentially effective strategy for wind climate modelling. The presented estimation process is based on the assumption of a stationary ergodic process, where the underlying process is considered constant over time with consistent parameter values. Future research can explore non-stationary analysis, which accounts for changes in the behaviour of extremes over time, allowing for the simultaneous variation of statistical properties. Moreover, a Markov chain Monte Carlo process can be explored to extend short data records by generating synthetic data series based on the statistics of the distribution revealed by the actual data sets.

Kenéz, Á., and Joó, A. L. (2023b). The impact of parameter estimation uncertainty in extreme wind speed models. Pollack Periodica, 18 (2023) 1, pp. 78-83, https://doi.org/10.1556/606.2022.00645, Impact factor: 0.26 (Q3)

Kenéz, Á., and Joó, A. L. (2020). Parameter estimation and threshold selection uncertainty in extreme wind speed distribution – A frequentist approach with generalized Pareto distribution using automatic threshold selection. Idojárás, 124(3), pp. 311-330, https://doi.org/10.28974/idojaras.2020.3.1, Impact factor: 0.86 (Q4)

Conference papers:

Kenéz, Á., and Joó, A. L. (2023c), Performance-based wind engineering assessment of a telecommunication mast. ce/papers, 6(3-4), pp. 1099-1104. https://doi.org/10.1002/cepa.2322

Kenéz, Á., and Joó, A. L. (2019), Extrém szélsebesség eloszlás paraméterbecslési bizonytalansága és hatása a tartószerkezetek megbízhatóságára - POT módszer automatikus küszöbválasztással, XIII. Magyar Mechanikai Konferencia, 2019.08.27-29.

Irrelevant publications and conference papers:

Szabó, G., Völgyi, I., Kenéz, Á., (2022) Vibration Assessment of a New Danube Bridge at Komárom", Periodica Polytechnica Civil Engineering, 66(4), pp. 1014–1022. https://doi.org/10.3311/PPci.19508, Impact factor: 1.3 (Q3)

Kenéz, Á., Jakab, S., Opoldusz, M., Joó, A. L. (2016) Vékonyfalú, hidegen alakított Z-szelemen átlapolt kapcsolatának vizsgálata, XX. Nemzetközi Építéstudományi Konferencia, Csíksomlyó, Románia, 2016.06.01-05., pp. 109-110

Lendvai A., Kenéz, Á., Rácz A., Joó, A. L. (2015) Acélcsarnokok burkolati merevsége - 3. rész: az Eurocode módszerének pontosítása. MAGÉSZ Acélszerkezetek 12: (1. különszám) pp. 78-83.

Kenéz, Á., Rácz A., Joó, A. L., Dunai, L. (2015) Acélcsarnokok trapézlemez burkolatának merevítő hatása, XIX. Nemzetközi Építéstudományi Konferencia, Csíksomlyó, Románia, 2015.06.04-07., pp. 97-100

Fenyvesi, O., Kenéz, Á., Kovács, É, Rácz, A. (2014) Ipari hulladékok újrahasznosítása beton adalékanyagként, Meddő? Hulladék? Nem! Haszonanyag!, Budapest, Hungary, 2014.05.15., pp. 111-118

Fenyvesi, O., Kenéz, Á., Kovács, É, Rácz, A. (2014) Szerkezeti könnyűbeton előregyártott hídszerkezeti elemekhez, XVIII. Nemzetközi Építéstudományi Konferencia, Csíksomlyó, Románia, 2014.06.12-15., pp. 149-152

and extreme wind effects. The internal axial forces calculated from strain measurements on the bottom chord member are compared with numerical model results, which incorporate wind load drag coefficients determined according to EN 1993-3-1 and data from wind tunnel experiments.

3 Available observations

3.1 Lattice mast in Sződliget - case study

An investigation is conducted on a freestanding telecommunication lattice tower located in the suburban area of Sződliget, central Hungary, utilizing the PBWE framework. The lattice mast has a total height of 49.70 m and triangular cross-section. Width of the structures varies along the vertical direction, the maximum width at the bottom is 4.80 m, while it is reduced to 3.00 m at the top. Main dimensions of the structure are shown in Figure 1. Both the chord members and bracings have circular hollow sections with various diameter (76 – 159 mm) and wall thickness (2.9 – 12.5 mm); structural steel grade is S235. Cross-sectional dimensions are presented in Figure 8. The tower is equipped with linear ancillaries, i.e., with ladder and cables, and a total of 23 sector and microwave antennae are installed. The tower is equipped with cup anemometer wind speed sensors, thermometry, and strain gauges to measure its response to wind loads. The wind speed was measured for the period between August 2011 and March 2015

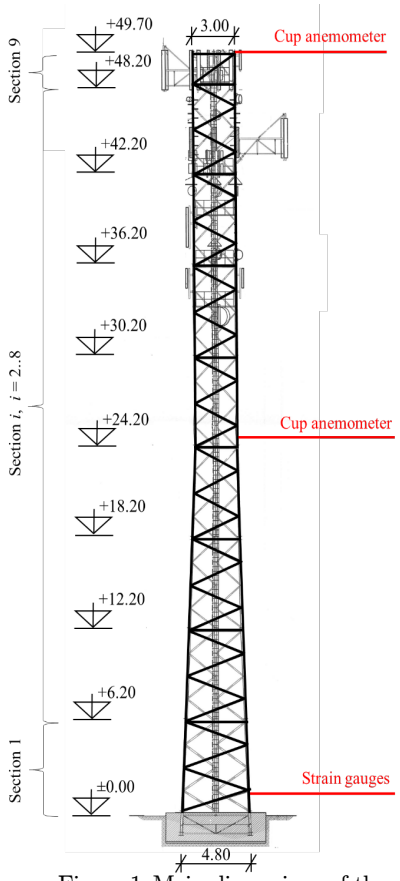

Figure 1. Main dimensions of the analysed telecommunication mast in Sződliget, Hungary

with a sampling rate of 0.9 s at the middle and the top of the lattice tower, i.e., 25 m and 49.70 m above ground level. Full-bridge strain gauges, with temperature compensation, are installed on chord members at 4.8 m above ground level. The north-western area of the lattice tower is characterized by dense urban development. Notations for wind direction α and chords with O159×13 tubular cross section (S1, S2 and S3) are shown in Figure 2.

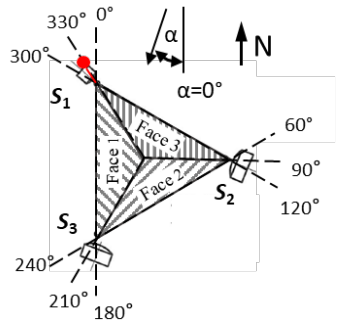

Figure 2. Notations of wind direction α and chords (S_1, S_2 and S_3);

3.2 Observations of the Hungarian Meteorological Service

In line with European Union directives, the Hungarian Meteorological Service (HMS) will make its observation and measurement data (from 2002 onwards) freely available for use through an open data server (https://odp.met.hu/) from 1 January 2021. The map with the locations of the wind measurement meteorological stations used in the research is shown in Figure 3. In this way, the information from Sződliget will be enhanced by incorporating measurement data series from various locations across the country. Annual maximum wind velocity data have been collected from 169 stations, and among these, 67 stations possess a minimum of 20 years of data. Alternatively, the possibility of augmenting the information from Sződliget with wind data obtained from the nearest meteorological station, Penc, is being considered.

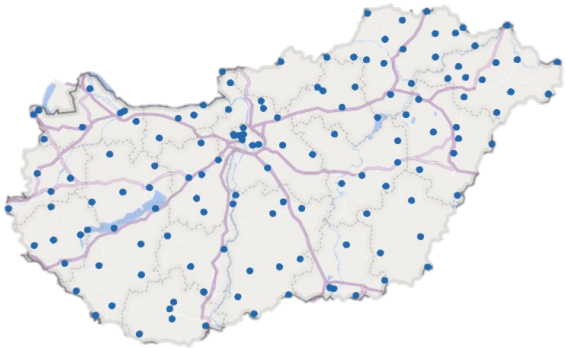

Figure 3. Meteorological stations of Hungarian Meteorological Service (HMS)

Thesis 4. I devised a Performance-based wind engineering framework based on finite element method (FEM) improved by Bayesian inference to assess the reliability of a freestanding telecommunication lattice tower exposed to extreme wind conditions.

a) Here are the key components of this framework:
   i. **Parameter Assessment:** Parameters for the stochastic extreme wind velocity model, including wind speed, direction, and roughness length, are determined through on-site data acquisition.
   ii. **Threshold Selection:** In addition to the traditional annual maxima method, the peaks over threshold (POT) method is employed, enhanced by an innovative automated threshold selection technique introduced by Thompson et al. (2009) This approach is verified using conventional graphical diagnostics. This approach is verified using conventional graphical diagnostics. The method had not been applied to wind velocity data before the studies were carried out.
   iii. **Bayesian Inference:** Bayesian inference is leveraged to synthesize information from diverse sources of wind data measurements and various extreme wind velocity models. This application within the PBWE framework is novel in its approach.
   iv. **Structural Response Modelling:** A finite element model is formulated to simulate the structural response of the tower. This facilitates structural engineers in accounting for various failure modes and complex phenomena.
   v. **Probabilistic Analysis:** Monte Carlo simulation, supported by random Latin hypercube sampling and quadratic response surfaces, is adopted for probabilistic analysis. This approach accelerates and enhances the efficiency of calculations, ensuring a comprehensive assessment.

b) I proved the applicability of the method through a Performance-based analysis of a telecommunication lattice tower located in Central Hungary.

c) The average differences in estimated reliability indices, calculated based on P_f assuming Gaussian distribution of EDPs, corresponding to MCS and RSM are 2% and 5% for the SLS and ULS calculations, respectively. The maximum differences for the SLS and ULS calculations are 9% and 17%.

Related publications: Kenéz, Á., and Joó, A. L. (2023a), Kenéz, Á., and Joó, A. L. (2020)

9.2 Relevant publications

Academic journal articles:

Kenéz, Á., and Joó, A. L. (2023a). Performance-based Wind Engineering assessment of a telecommunication mast utilizing Bayesian extreme wind velocity model, Stochastic Environmental Research and Risk Assessment, (2023), https://doi.org/10.1007/s00477-023-02596-w, Impact factor: 4.2 (Q1)

interest, results in a more reliable and consistent estimated P_f, aligning with other estimated exceedance probabilities where at least 20 year-long and multiple meteorological data series are considered. The Kriging spatial interpolation exhibits smaller estimated P_f by an order of magnitude compared to the reference case where the generalized extreme value distribution parameters calculated using the mean and standard deviation of the annual maxima wind velocity data. Furthermore, Bayesian updating of prior information associated with the Hungarian meteorological stations, incorporating observed data from the site of interest, reduces the P_f by approximately one order of magnitude compared to the reference case.

Related publications: Kenéz, Á., and Joó, A. L. (2023a), Kenéz, Á., and Joó, A. L. (2023b)

Thesis 3. I demonstrated the significant impact of the scale parameter of the extreme wind velocity distribution on the selected Engineering Demand Parameters (EDPs) in the context of the probabilistic analysis within the presented Performance-based wind engineering framework. This impact is particularly significant when the Peaks Over Threshold (POT) method with automated threshold selection technique is employed with a dataset of fewer than 500 wind measurements and the generalized extreme value distribution parameters are estimated utilizing maximum likelihood estimation. Monte Carlo simulation, utilizing random Latin hypercube sampling with 160 samples (MCS) and a finite element model for structural analysis, is used to fit quadratic response surfaces for engineering demand parameters and develop a surrogate model. Subsequently, Monte Carlo simulations with $N = 10^6$ simulation loops are conducted on the derived response surfaces (RSM) to determine the probabilities of exceeding the specified limit values. The estimated Spearman rank-order correlation coefficient and the Sobol index of the scale parameter is 0.27 and 0.08, respectively. Therefore, it is crucial to account for the epistemic uncertainty associated with the scale parameter during the POT procedure.

Conversely, when utilizing Bayesian inference to synthesize information from the POT procedure, considering a dataset of fewer than 500 samples, alongside the Annual Maxima method and a 20-year long data series, the scale σ and location μ parameters are found to be statistically insignificant at a significance level of 2.5%. Consequently, the associated epistemic uncertainty related to these parameters can be safely disregarded.

Related publications: Kenéz, Á., and Joó, A. L. (2023a), Kenéz, Á., and Joó, A. L. (2023b)

4 The PBWE methodology

Advanced investigation on quantitative assessment of structural performance is carried out based on the PBWE framework shown in Figure 4. Key parameters which describe sufficiently and efficiently the climate wind hazard should be chosen as intensity measure parameters (*IM*). Mechanical and material properties of the structure of interest, whose uncertainty may contribute significantly to the structural performance, should be considered as stochastic variable in structural parameters (*SP*). The interaction phenomenon between the environment and the structure should be modelled in probabilistic terms and can be described with interaction parameters (*IP*). In order to perform the probabilistic analysis of structural response, relevant engineering demand parameter (*EDP*) should be defined. Damage measure parameters (*DM*) should be also defined by one or combination of more *EDP*s to quantify the structural damage.

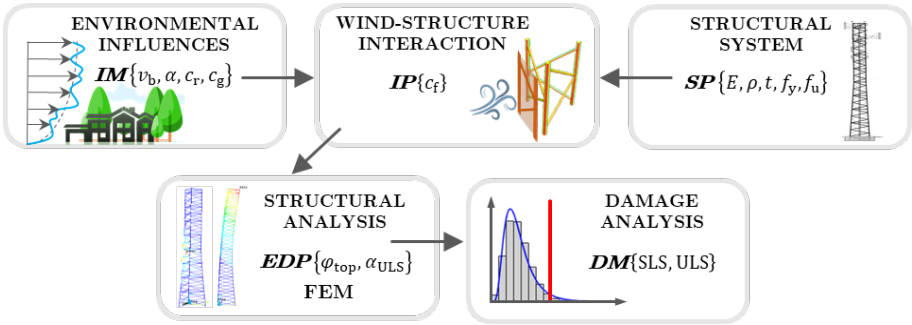

Figure 4. The implemented PBWE framework

The flowchart of the proposed analysing framework is depicted in Figure 5, where D denotes the data, σ_i and μ_i represent the scale and location parameters of the distributions of interest. The probability of the structural failure P_f is assessed using Monte Carlo simulation with random Latin hypercube sampling.

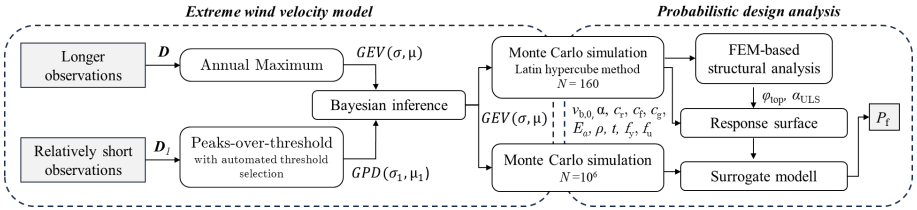

Figure 5. The flowchart of the proposed analysing framework

Quadratic response surfaces are fitted for output variables, such as the rotation of the top of the mast φ_{top} for serviceability limit state (SLS), while load amplification factor α_{ULS} is used for ultimate limit state (ULS) for response surface fitting. Monte Carlo simulations, with $N = 10^6$ simulation loops, are performed on the derived response surfaces to gain the distribution functions of *EDP*s and the probabilities of exceeding the targeted limit value. After determining the probability of the *EDP* exceeding the specified limit, which is set at 0.5° for the maximum allowable rotation of the top of the tower ϕ_{top} for the SLS and 1 for the limit of the load amplification factor for the ULS, the reliability index β is selected and computed as a conventional measure of reliability. Probabilistic design variables and the corresponding distribution functions and parameters are summarised in Table 1 for SLS and ULS simulations based on the recommendations of JCSS (Joint Committee on Structural Safety, 2001) and measurement results.

Table 1. Probabilistic design variables for PBWE

Limit state	Probabilistic input variable		Distribution function	Distribution parameters
SLS and ULS	Wind direction α		Gaussian	$\alpha_{mean} = 299.719°$ $\sigma_\alpha = 18.621°$
	Fundamental basic wind velocity $v_{b,0}$ ($\xi = 0$)	Scale σ	Gaussian	(Measurements)
		Location μ	Gaussian	(Measurements)
	Force coefficient c_f		Gaussian	$c_{f,mean}$ according to EN 1993-3-1 or wind tunnel test, CV = 0.15
	Roughness factor c_r		Gaussian	$c_{r,mean}$ according to EN 1991-1-4, CV = 0.20
	Gust factor c_g		Gaussian	$c_{g,mean}$ according to EN 1991-1-4, CV = 0.17
	Young's modulus E		Lognormal	$E_{mean} = 210\,000$ MPa $\sigma_E = 0.03 \times E_{mean} = 6300$ MPa
	Density ρ		Gaussian	$\rho_{mean} = 7700 kg/m^3 \times 1.05 = 8085 kg/m^3$ $\sigma_\rho = 0.01 \times \rho_{mean} = 80.85$ kg/m^3
	Wall thickness t		Gaussian	$t_{mean} = t_{nom}$, $\sigma_t = 0.05 \times t_{mean}$
ULS only	Yield strength f_y		Lognormal	$f_{y,mean} = 235MPa \times 1.00 \times e^{2.0 \times 0.07} - 20MPa = 250.3MPa$ $\sigma_{fy} = 0.07 \times f_{y,mean} = 17.5$ MPa
	Ultimate strength f_u		Lognormal	$f_{u,mean} = 1.5 \times f_{y,mean} = 375.5$ MPa $\sigma_{fu} = 0.04 \times f_{u,mean} = 15.0$ MPa

9 New academic contributions

9.1 Theses of the dissertation

Thesis 1. I compared the wind force, as calculated in accordance with EN 1993-3-1, to the wind force derived from wind tunnel experiments conducted by Balczó et al. (2006). These tests involved a bare mast section with a triangle cross-section with width of 1300 mm and a length of 1600 mm, constructed using hot-dip galvanized circular hollow sections with diameters of $D = 108$ mm for chord members and $D = 75$ mm for the bracing members. However, the EN 1993-3-1 standard formula conservatively omits the critical range and lacks specific definitions for the Reynolds number at which drag coefficients decrease. Consequently, there exists a disparity between the drag coefficient determined through wind tunnel measurements and the standard value and this difference falls within the range of 15-19% for Reynolds numbers spanning from $2.7\text{-}2.8 \cdot 10^5$, corresponding to the basic wind velocity determined in the Hungarian National Annex.

I conducted a comparison between the internal axial forces obtained from strain measurements on the bottom chord member of a full-scale telecommunication lattice mast and the results from a deterministic numerical model. This model incorporated wind load drag coefficients calculated according to EN 1993-3-1, as well as data from wind tunnel experiments. Differences in the internal axial forces under tension became apparent, with variations ranging from 20% to 40% for wind velocities of 20 and 30 m/s. To mitigate these discrepancies, wind forces measured in wind tunnel tests were applied exclusively to the upper tower segment, encompassing sections 7-9, because these sections align with the ones utilized in the wind tunnel experiments. For the lower part of the tower, comprising sections 1-6, standard wind forces were utilized. This adjustment resulted in a reduction of differences to approximately 5-20%.

Related publications: Kenéz, Á., and Joó, A. L. (2023a)

Thesis 2. In this investigation, the utilization of wind tunnel-derived drag/force coefficient c_f, instead of calculated according to EN 1993-3-1, significantly reduces the estimated probability of failure P_f by approximately two orders of magnitude. Adjusting the roughness length z_0 on the basis of on-site measurements, instead of the value proposed in EN 1991-1-4 standard for suburban terrain, can lead to one order of magnitude decrease in estimated P_f. The observed fundamental basic wind velocities at Sződliget and Penc are lower by 57% compared to the value considering all Hungarian meteorological stations. It results in significantly lower estimated P_f for these locations, with differences of approximately three orders of magnitude for SLS and around eight orders of magnitude for ULS. Applying Kriging spatial interpolation for extreme wind velocity calculations, along with Bayesian updating using observed data from the site of

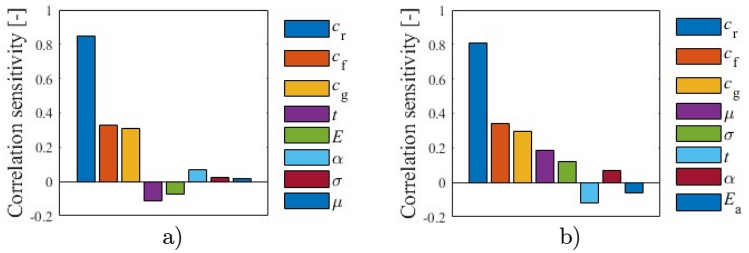

Figure 15. Spearman rank-order coefficients for SLS for a) Hungary, b) Hungary - Kriging, c) Hungary - Sződliget, and d) Sződliget

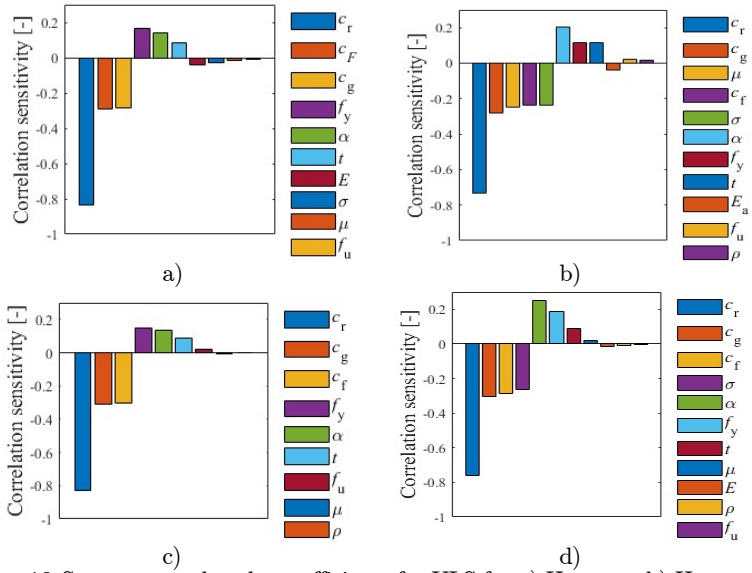

Figure 16. Spearman rank-order coefficients for ULS for a) Hungary, b) Hungary - Kriging, c) Hungary - Sződliget, and d) Sződliget

4.1 Model of extreme wind velocity

The dissertation presents the Bayesian updating of wind velocity model parameters, which combines information from measurements collected at the site of the tower over a relatively short duration with a more extensive dataset from measurement locations across the country provided by the HMS (Figure 6). Due to the limited amount of available data, there is a risk that the estimated shape parameter ξ of the generalized extreme value (GEV) distribution may exhibit high statistical uncertainty, potentially resulting in unreasonable predictions for extreme wind velocities. For this reason, and to facilitate the update of the scale σ and location μ parameters of the GEV distribution with a bivariate normal distribution in the Bayesian update process, the shape parameter ξ is set to zero. Hence, the value of the fundamental basic wind velocity has been conservatively determined. The peaks-over-threshold (POT) approach is implemented to determine the parameters of the distribution of the extreme wind speeds at Sződliget, due to the relatively short length of available measurement data series. An automated threshold selection technique, proposed by Thompson et al. (2009), is applied and verified using conventional graphical diagnostics.

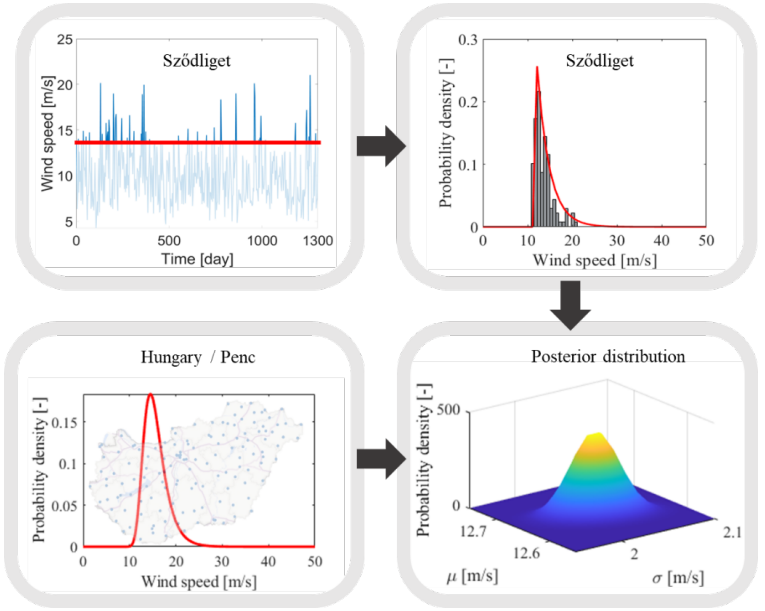

Figure 6. Flowchart of the POT method with Bayesian inference

In this study, the roughness length is determined based on the measured wind velocities at heights of 25 m and 50 m utilizing the logarithmic profile for wind speed averages greater than 10 m/s where the averaging period is 10 minutes, and the wind speed threshold of 10 m/s corresponds to a height of 25 m. For data analysis, including fitting probability distributions to data and conducting hypothesis tests, the MATLAB (R2022a) software is employed, the fitting of parameters for different distributions is performed using maximum likelihood estimation (MLE).

4.2 Wind – structure interaction

Equivalent static wind force acting on the structure is evaluated based on the procedure formulated in EN 1991-1-4 and EN 1993-3-1 (CEN, 2006). In comparison to the wind force calculated according to EN 1993-3-1, the structural performance is determined by applying the wind force obtained from wind tunnel tests conducted by Balczó et al. (2006), as well (Figure 7). The wind tunnel tests were conducted on a mast section with a diameter of $D = 108$ mm for the hot dip galvanized legs and $D = 75$ mm for the bracing members. The drag coefficient of the bare structure fluctuates between 0.87 and 1.03, contingent on the wind direction and computed in accordance with EN 1993-3-1. Conversely, it falls within the range of 0.75 to 0.85 based on wind tunnel measurements conducted within the Reynolds numbers range of $2.7\text{-}2.8 \cdot 10^5$. The problem arising from differences between drag coefficients derived from wind tunnel tests on full-size structures and on reduced-scale models is to be avoided. These differences are reflected not only in the coefficient magnitude but also in the wind velocity range (Reynolds number) at which they occur.

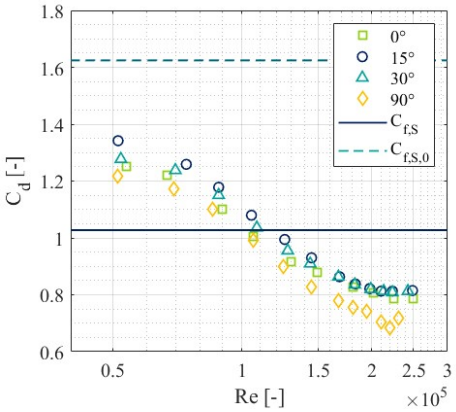

Figure 7. The drag coefficient of the mast section for different Reynolds numbers (based on Balczó et al. (2006))

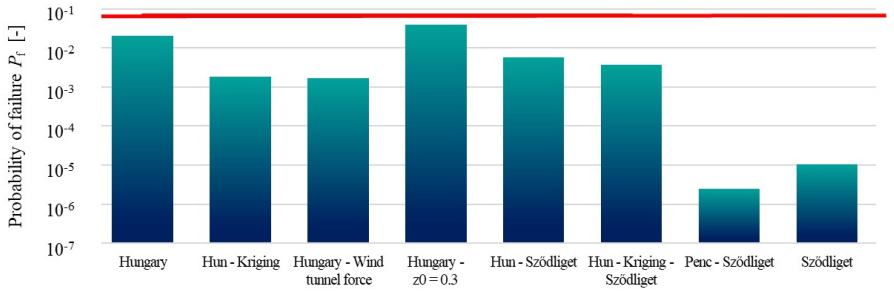

Figure 13. The probability of exceeding the serviceability limit for different scenarios and the target P_f according to EN 1990 (EC RC2)

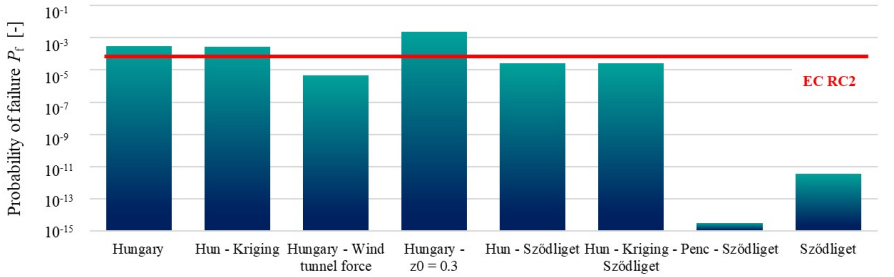

Figure 14. The probability of structural failure P_f for different scenarios and the target P_f according to EN 1990 (EC RC2)

8 Sensitivity analysis

The probabilistic sensitivities of Spearman rank-order correlation coefficients are also determined (Figure 15. and 16.). It is highlighted that the roughness factor c_r, force coefficient c_f and gust factor c_g have a dominant influence on the *EDP*s due to their higher CVs, while material parameters have smaller influence on the *EDP*s. The density of the steel ρ is found to be statistically insignificant at a significance level of 2.5% in all cases. In addition, it is observed that σ has a significant impact on the *EDP*s for data extracted from Sződliget, where only the POT method was utilized. In the case of the Hungarian 20-year-long measurement data series, the parameters σ and μ have a relatively small impact. However, in the Bayesian updated case, these two parameters are found to be statistically insignificant. Although, the present statements are valid for the dataset under consideration and for similar climatic conditions, methods presented in this paper are applicable for other wind engineering problem.

7 Discussion of the reliability of the structure

The probabilities of exceeding the serviceability limit and the structural failure P_f are calculated based on fitted lognormal distributions of RSM simulations (Figure 13-14.). The level of safety for reliability class RC2, in other words, the target probability of exceeding the limit for reference periods of 50 years is $6.68 \cdot 10^{-2}$ for SLS and $7.23 \cdot 10^{-5}$ for ULS, respectively (EN 1990:2002, 2002). By utilizing the force coefficient c_f derived from wind tunnel tests, the P_f is reduced by approximately two orders of magnitude. Assuming a roughness length z_0 of 0.3, as specified according to the EN 1991-1-4 standard for suburban terrain, instead of using the value of 0.877 based on measurement, leads to an increase in the P_f by approximately half an order of magnitude for the SLS and one order of magnitude for the ULS. Due to the considerably lower observed wind speeds at Sződliget and Penc compared to the Hungarian average, the P_f for Sződliget and "Penc- Sződliget" are significantly lower. The associated P_f are lower by approximately three orders of magnitude for the SLS and around eight orders of magnitude for the ULS. Taking into account raw wind field data from a nearby monitoring station with longer measurements as prior information when estimating the parameters of an on-site extreme wind speed model for a structure may be considered unconservative, as demonstrated in the case of "Penc-Sződliget". The topography around Sződliget is characterized by the town on flat terrain, with the Danube River situated behind it. In contrast, the surroundings of Penc feature hills with forested areas, which may contribute to lower wind speeds and assessed failure probabilities for „Penc" and „Penc - Sződliget" due to the influence of the terrain. Using Kriging spatial interpolation to calculate wind speed values at the site of interest from data sets of other stations, and even updating it with field measurement information, leads to a more reliable and consistent result in line with other estimated exceedance probabilities. The estimated exceedance probabilities of the "Hun - Kriging" are slightly smaller, by an order of magnitude for SLS, than the "Hungary" case. Bayesian updating of the prior information or beliefs associated with the Hungarian average by incorporating the observed data from Sződliget can reduce the probability of failure P_f by approximately one order of magnitude compared to the case of "Hungary".

4.3 Structural analysis

The numerical model of the telecommunication mast subjected to wind loading is developed in a general-purpose finite element software (ANSYS v19.0), in order to predict wind load-induced structural response. Main dimensions of the structure and designations of chords and bracings are shown in Figure 8. Three-dimensional two-node beam elements (BEAM188), with quadratic shape functions, are used in the finite element model. Nodal wind force loads are defined in the nodal coordinate system, which is rotated with respect to the actual wind direction for each simulated case. This investigation does not consider actions from ice, including their gravitational effects or their impact on wind actions. Furthermore, the analysis does not account for the impact of one-sided solar radiation and the consequent temperature difference.

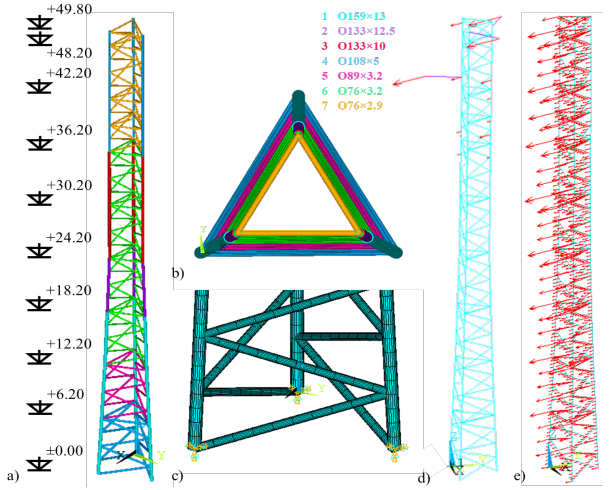

Figure 8. Finite element model of the mast: a) section types, b) top view, c) boundary conditions, nodal loads for d) antennae and e) structure with linear ancillaries

Linear material model is applied in the model for predicting rotations and displacements in serviceability limit state (SLS). Large deflection effects are included in all the simulations. On the other hand, nonlinear material properties, and equivalent geometrical imperfections are included as well in the geometrically and materially nonlinear analysis with imperfections (GMNIA) for assessing load-bearing capacity in ultimate limit state (ULS). Quad-linear isotropic hardening material model is defined for S235 structural steel grade according to Gardner et al. (2019) and prEN 1993-1-14 (CEN, 2020). The global (sway imperfection of the entire mast) and local (combined flexural buckling-type imperfection of bracing and chord) imperfections are also taken into consideration.

5 Internal axial forces in chords from measurement

Assuming uniaxial stresses in chord members, internal axial forces are evaluated applying the Hook's law and using the Young's modulus E (210000 MPa) and the cross-sectional area of the measured chords (5963 mm^2). Internal axial forces in chords S_1, S_2 and S_3 are sorted corresponding to wind speed of $v = 10$, 15, 20, 30 m/s, and turbulence intensity of 10, 20, 30%. Then, sinusoidal functions are fitted to these data points and expected values are extracted regarding the interesting directions for the triangular section tower, i.e., 270°, 300° and 330°. Two examples can be seen in Figure 9 and Figure 10, for the case of wind speed $v = 15$ m/s.

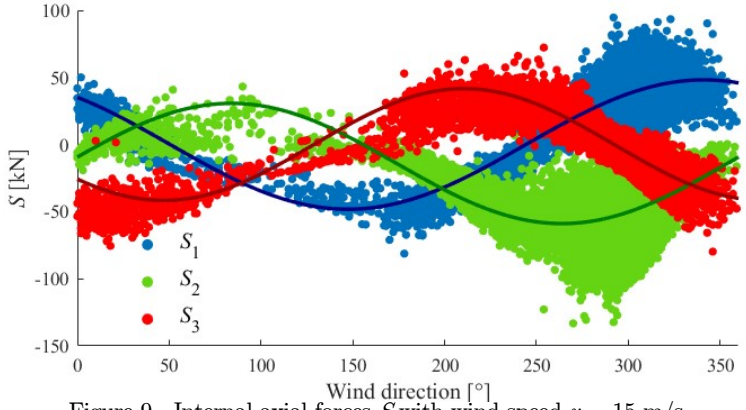

Figure 9. Internal axial forces S with wind speed $v = 15$ m/s

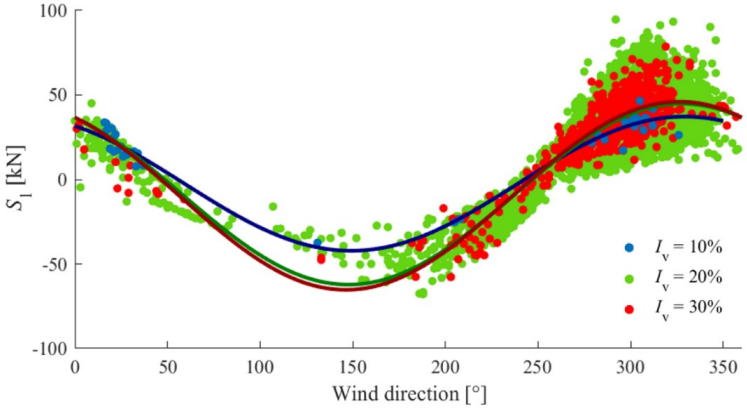

Figure 10. Internal axial force S_1 at $v = 15$ m/s and turbulence intensities of 10, 20, 30%

6 Discussion of force coefficients

The results of the deterministic numerical model based on EN 1993-3-1, specifically internal axial forces in chord members, are compared to the extracted values from measurements. Due to the north-western prevailing wind direction, the S_3 axial force is in the vicinity of the neutral axis; therefore, these values are not presented here. In the case of S_1, which is subjected to tension, a noticeable difference of approximately $20 - 40\%$, can be observed for high wind velocities (Figure 11). In the case of S_2, the differences for high wind velocities are lower, $\sim 10 - 25\%$ (Figure 12). By utilizing wind forces measured in the wind tunnel tests, the model was able to reduce these differences to approximately 10-20%. The differences for low wind velocities are less than 14%.

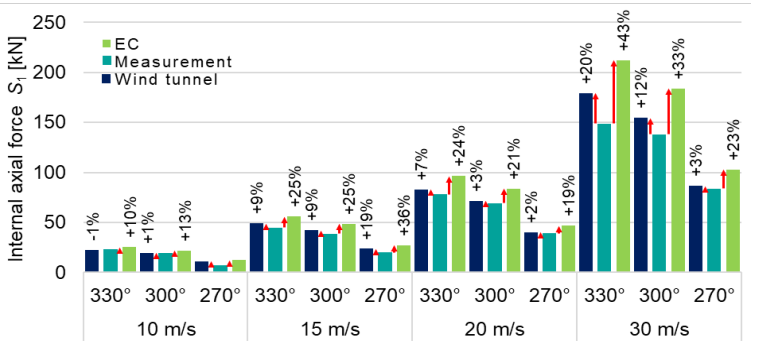

Figure 11. Internal axial force S_1 for $I_v = 20\%$, and for various wind velocity, wind direction

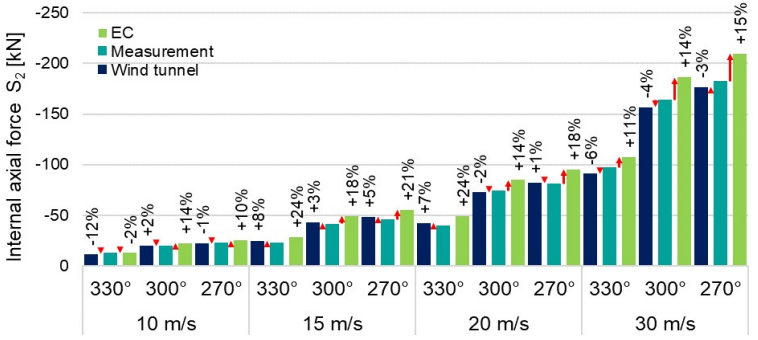

Figure 12. Internal axial force S_2 for $I_v = 20\%$, and for various wind velocity, wind direction