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# DESIGN OF STEEL STRUCTURES FOR A GIVEN LEVEL OF RELIABILITY USING PARTIAL SAFETY FACTORS CALIBRATION PROCEDURE

**J. Szafran[1) ] K. Juszczyk [2)] M. Kamiński [3)]**

[1)] Adjunct Professor, Chair of Structural Reliability, Department of Structural Mechanics, Faculty of Civil Engineering, Architecture and Environmental Engineering, Łódź University of Technology, Łódź, POLAND, *email: jacek.szafran@p.lodz.pl*

[2)] M.Sc. Eng, Chair of Structural Reliability, Department of Structural Mechanics, Faculty of Civil Engineering, Architecture and Environmental Engineering, Łódź University of Technology, Łódź, POLAND, *email: kjuszczyk@compactprojekt.pl*

[3)] Professor, Head of Chair of Structural Reliability, Department of Structural Mechanics, Faculty of Civil Engineering, Architecture and Environmental Engineering, Łódź University of Technology, Łódź, POLAND,
*email: marcin.kaminski@p.lodz.pl*

**ABSTRACT:** The main aim of this study was to perform calibration procedure of partial safety factors, which are used in Eurocodes calculation techniques to ensure necessary level of structural safety. Some representative examples of the slender steel structures were used to present this method including steel chimney and three examples of steel lattice telecommunication towers having different heights. Finite Element Method computational models were used to perform static analysis and the Ultimate Limit State was considered. The main attention of the analysis is focused on bending moments of the chimney basis and axial forces in legs of the towers. The generalized stochastic perturbation technique has been programmed in symbolic algebra package MAPLE and used to determine the first two moments of the necessary structural responses of the chimney and the towers. The characteristic wind pressure has been treated here as the input random variable with given expectation and variability interval for the coefficient of variation. Partial safety factors were calculated for different reliability indices β and different coefficients of variation of the environmental load effects.

**Keywords:** partial safety factors, calibration procedure, structural safety, perturbation method, reliability index

## 1. INTRODUCTION

It is obvious that buildings and engineering structures cannot be designed taking into consideration the mean values of all the environmental and dead loads only. Possible failures of these structures often makes the risk for the people and may also have relatively high economic, social and some environmental consequences. Therefore, the appropriate safety level must be maintained and, at the same time, designing of the structures should be accompanied with some optimization procedures. European Standards suggest all to design civil engineering structures using partial safety factors as the coefficients used for impact effects, load capacity or material properties, which result in the certain safety margins. Identification of the reliability level is done thanks to the reliability index β, which takes into account the accepted or adopted level of statistical scattering of these impact effects, carrying capacity and uncertainty of the model depending on the consequences of failure of the designed structure (Ref. 11). Statistical estimation or probabilistic determination of the partial safety factors is the subject of an extensive theoretical, computational and also experimental research. Hicks and Pennington in Ref. 4 present for instance the results from a reliability analysis of the resistance of composite beams in sagging bending, designed according to Eurocode 4. They evaluate these partial factors related to carrying capacity for the structural steel, concrete and shear connection using a methodology contained in Ref. 1. Calibration procedure of the resistance partial factors in modelling of the steel structures reliability according to Eurocodes is presented in Ref. 13 as well. Casas and Chambi in Ref. 1 describe the methodology for a reliability-based calibration of the partial safety factors used for the confined concrete elements in the design of strengthening or seismic retrofitting of bridge piers using fiber reinforced polymers. New material safety factors for the seismic safety assessment were also proposed by Pereira and Romao in Ref. 10 to characterize strength capacity of the existing buildings.

There are many new technologies and materials which could be used in civil engineering due to its easy application and excellent mechanical and chemical properties, however partial or a complete lack of the codes, standards and experience in the long term behavior make these solutions reluctantly used by the designers and engineers. Efficient estimation of partial safety factors for material properties allows getting an appropriate safety margin. Usage of the reliability-based design is also common in geotechnical designing. Partial factors are also often used in these codes to overcome the difficulties in performing probabilistic analysis. In Ref. 8 and Ref. 9 authors develop partial safety factors for different geotechnical design applications, including rock slopes stability and design of the support for a rock wedges in an underground opening. The external loads partial safety factors were estimated in Ref. 7 by Lenner and Sykora. These Authors deal with special purpose heavy vehicles on road bridges and propose a methodology for calibration of the related partial factors. Key steps of this approach consist of assessing static load effect, dynamic amplification, model uncertainty, sensitivity factors and final reliability.

There is no versatile procedure to calibrate partial safety factors. One of the available approaches was shown in Ref. 2. A quantile-based approach for calibrating reliability-based partial factors that is based on the equivalence principle between the design quantiles for the random variables and the target reliability was presented. According to the Authors the proposed approach enables to keep a uniform reliability over a wider range of design parameters with a single design quantile, which cannot be easily attained by other calibration methods such as the First Order Reliability Method. Somewhat different methodology, based on Eurocodes, has been presented by Sedlacek and Kraus in Ref. 12.

In this paper calibration procedure has been applied to calculate partial safety factors correlated to the wind pressure acting on some slender structures such as chimneys and towers. It enables to achieve suitable safety levels of the construction in conjunction with characteristic values of the loads. An impact of the reliability index $\beta$ and some relationships between coefficients of variation are observed and discussed in details.

## 2. COMPUTATIONAL EXAMPLES

### 2.1. Steel chimney - benchmark example

First analysis has been carried out for a simple example of the steel chimney with the height equal to 40.0 m and having 1.2 m in diameter, made of the structural steel S235. The entire structure is fully restrained at its bottom and has a thickness equal 6 mm at its top to 12 mm at its bottom (Fig. 1). Numerical analysis in civil engineering software Autodesk Robot Structural Analysis has been performed for the chimney modelled as the vertical cantilever beam (by using 8 2-noded linear 3D beam finite elements), where expectation of the wind velocity value E[v] equals to 22 m/s. A series of analyses has been performed for 11 cases of varying wind mean load, which has been generated by multiplying the mean value of wind velocity by the following factors: 0.5; 0.6; 0.7; 0.8; 0.9; 1.1; 1.2; 1.3; 1.4; 1.5. The values of average wind speeds for the individual cases differed correspondingly by 10% of their base values in this way. This wind load has been estimated based on aerodynamic properties and applied to the model as a linearly distributed load (Fig. 1) having its minimum value at the terrain level and moderately increasing up to the top of this chimney. It needs to be emphasized that this type of simplified computational approach is most frequently used in steel chimneys design procedure. This chimney load capacity was assumed as a bending load capacity of the cross section at the base, and it is equal 3980.1 kNm for the purpose of the analysis; standard deviation of this capacity has been assumed as equal to 10%.

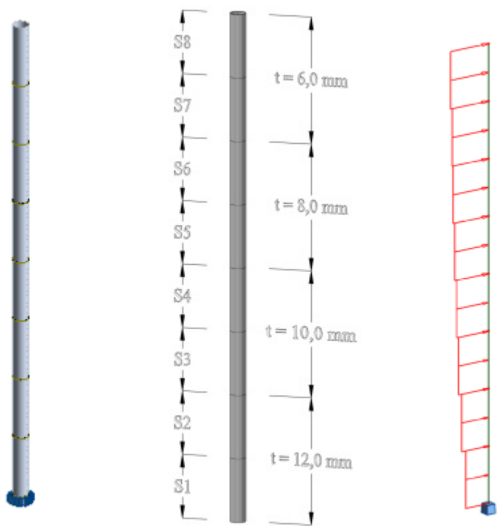


Fig. 1 The view (left), the scheme (middle) of the chimney and the wind load applied to the model (right).

### 2.2. Telecommunication towers

This part of numerical analysis includes three steel lattice telecommunication towers which have a similar structure and different height. These towers have been designed as the three-dimensional steel trusses of a triangular cross-sections and height of 40.0 m, 52.0 m and 58.0 m accordingly, and subdivided into separate structural segments. Their upper parts have triangular cross-sections and the bottom parts form a prism with a constant 5% convergence. Upper parts of these towers are a parallelepiped of a height equal to 6.0 m with the cross-section of an equilateral triangle with side length equal to 1.50 m.

Table 1. Tower no 1 (40.0 m high) - selected element profiles [mm].

| Section | Section height | Cross-sections of the legs | Cross-sections of the cross braces |
|---|---|---|---|
| S-1 | 6.0 | Ø 65 | ∟60x60x5 |
| S-2 | 5.0 | Ø 65 | ∟60x60x5 |
| S-3 | 5.0 | Ø 80 | ∟60x60x6 |
| S-4 | 6.0 | Ø 80 | ∟90x60x8 |
| S-5 | 6.0 | Ø 90 | ∟90x60x8<br>∟100x75x8 |
| S-6 | 6.0 | Ø 90 | ∟100x75x8 |
| S-7 | 6.0 | Ø 100 | ∟120x80x8 |

Table 2. Tower no 2 (52.0 m high) - selected element profiles [mm].

| Section | Section height | Cross-sections of the legs | Cross-sections of the cross braces |
|---|---|---|---|
| S-1 | 6.0 | Ø65 | ∟60x60x6 |
| S-2 | 5.0 | Ø65 | ∟60x60x6 |
| S-3 | 5.0 | Ø65 | ∟60x60x6 |
| S-4 | 6.0 | Ø80 | ∟60x60x6 |
| S-5 | 6.0 | Ø80 | ∟90x60x8 |
| S-6 | 6.0 | Ø90 | ∟90x60x8 |
| S-7 | 6.0 | Ø90 | ∟90x60x8 |
| S-8 | 6.0 | Ø95 | ∟90x60x8 |
| S-9 | 6.0 | Ø95 | ∟90x60x8 |

Table 3. Tower no 3 (58.0 m high) - selected element profiles [mm].

| Section | Section height | Cross-sections of legs | Cross-sections of cross braces |
|---|---|---|---|
| S-1 | 6.0 | Ø65 | ∟60x60x6 |
| S-2 | 5.0 | Ø65 | ∟60x60x6 |
| S-3 | 5.0 | Ø65 | ∟60x60x6 |
| S-4 | 6.0 | Ø80 | ∟60x60x6 |
| S-5 | 6.0 | Ø80 | ∟90x60x8 |
| S-6 | 6.0 | Ø90 | ∟90x60x8 |
| S-7 | 6.0 | Ø90 | ∟90x60x8 |
| S-8 | 6.0 | Ø95 | ∟90x60x8 |
| S-9 | 6.0 | Ø95 | ∟90x60x8 |
| S-10 | 6.0 | Ø100 | ∟120x80x8 |

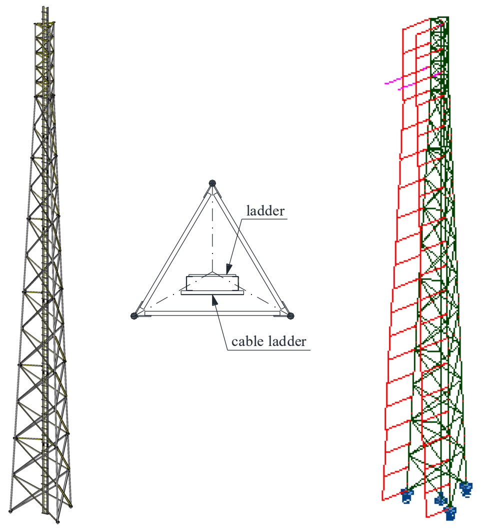


Fig. 2 Aerial view (left), the cross-section of the 40.0 m tower (middle) and the wind load applied to the model (right).

The leg members in each section consist of the round solid bars, while the bracing elements are hot-rolled symmetrical and non-symmetrical angle sections. The diagonal bracing system of this towers is X type (Fig. 2). The basic profiles of particular elements of each tower are presented in Tab. 1-3 below, while geometrical parameters are shown in Fig. 3. The “real load capacity” of the towers has been proposed in partial safety factors determination procedure as well as standard deviation of the load capacity, which has been obtained by the full-scale pushover tests (Ref. 14, 16, 17). The entire tower FEM model has been prepared (Ref. 15) in the civil engineering software Autodesk ROBOT Structural Analysis, v. 2015. This model has been created with the use of 396 3D linear beam finite elements, having 6 degrees of freedom in each node, that are connected in 278 nodal points. Additionally, geometrical imperfections have been introduced for the legs in two lower tower sections (S-7 and S-6) measured in situ before the experiment. Compatible nodes have been introduced in all the crossings for the X patterns of the tower bracings; elastic supports for these towers have been assumed accordingly also.

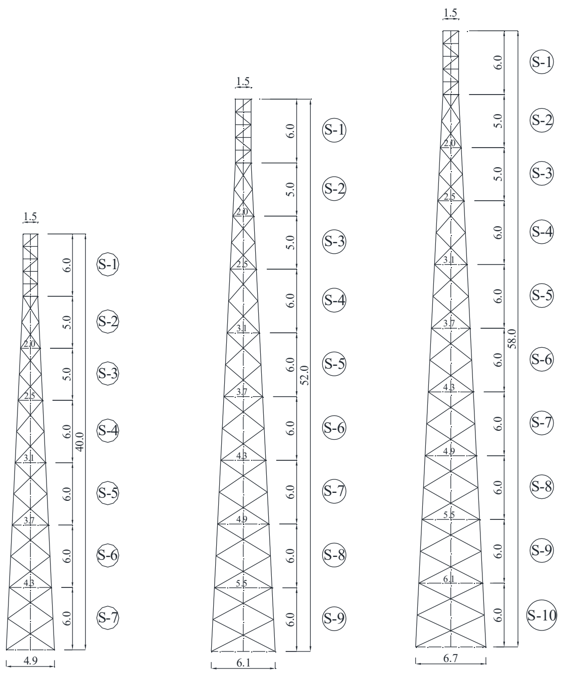


Fig. 3 Structural schemes of the tower 40.0 m (left), 52.0 m (middle) and 58.0 m (right).

The structure model was subjected to a strong wind load, which has been estimated based on the standards in force. The tower equipment configuration, telecommunication devices and supporting structures assumed for this analysis are shown in Fig. 4. While the wind load on the body of the tower was modelled as a linear and affecting the legs of the structure, the wind load for the elements of equipment has been introduced in the form of concentrated forces applied to the selected nodes (Fig. 2).

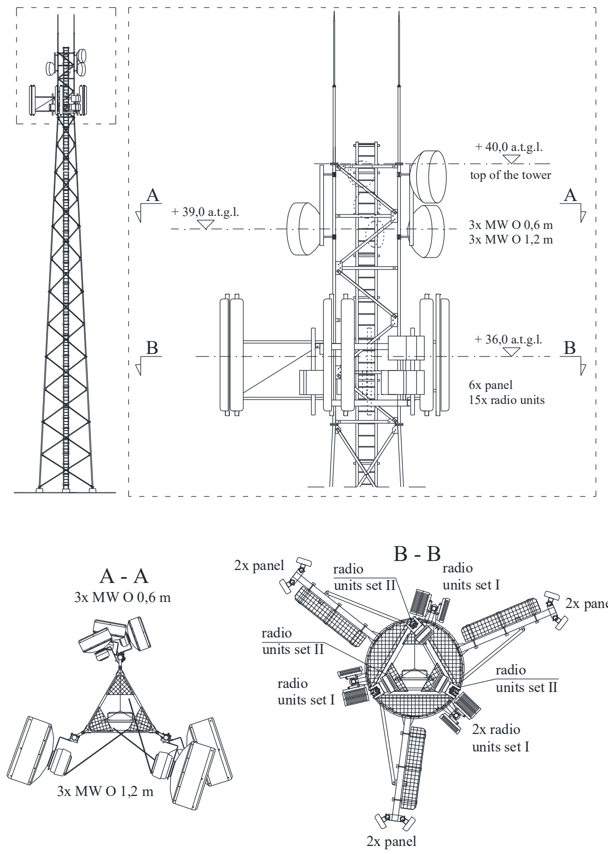


Fig. 4 Configuration of telecommunication devices and supporting structures: front view (top) and cross sections (bottom).

FEM static analysis has been carried out in the system called Autodesk Robot Structural Analysis 2015 also. The wind load based on the standard Eurocode Part 1 has been assessed assuming average velocity value $v = 22$ m/s. A series of FEM analyses is very similar as before and has been performed for 11 cases of the wind pressure.

## 3. PARTIAL SAFETY FACTORS CALIBRATION PROCEDURE

### 3.1. Steel chimney

A simplified procedure has been applied during analysis of the steel chimney. The first step consisted in a determination of the reliability indices $\beta$ for different values of the coefficient of variation of the structural response. In general, it can be assumed that the reliability index is a simple identifier of the structural safety state in the context of probability theory. On this basis one can introduce some required level of the structural safety and durability, varying requirements depending on the consequences of possible material or element failure and also of the global structural damage. The extreme expected value $E[M_x]$ and standard deviation $\sigma[M_x]$ of the real bending moments in the FEM model have been calculated for this purpose. We can express reliability index for this case study in the following manner:

$$\beta = \frac{\gamma - 1}{\sqrt{\alpha_E{}^2 + (\gamma \cdot \alpha_R)^2}}, \tag{1}$$

where $\alpha_E$, $\alpha_R$ are the coefficients of variation for the action effects and for the resistance respectively, which are defined as follows:

$$\alpha_E = \frac{\sigma[M_x]}{E[M_x]}, \tag{2}$$

$$\alpha_R = \frac{\sigma[M_{b,ex}]}{E[M_{b,ex}]}. \tag{3}$$

$E[M_{b,ex}]$ denotes the expected value of load capacity and $\sigma[M_{b,ex}]$ is standard deviation of this variable. The so-called central safety factor $\gamma$ is introduced as

$$\gamma = \frac{E[M_{b,ex}]}{E[M_x]}. \tag{4}$$

Partial safety factor has been determined using the reliability indices as follows:

$$\gamma_E = 1 + \beta \cdot |v_E| \cdot \alpha_E, \tag{5}$$

where $|v_E|$ represents the sensitivity factor (calibration factor) described by the following formula:

$$|v_E| = \frac{1}{\sqrt{1 + \left(\frac{\gamma \cdot \alpha_R}{\alpha_E}\right)^2}}. \tag{6}$$

### 3.2. Telecommunication towers

Further partial safety factors calibration procedure based on the Second Order Reliability Method (SORM) analysis for the cases of telecommunication towers has been performed. The generalized stochastic perturbation technique (Ref. 5, 6) based on a Taylor expansion has been implemented to calculate the basic probabilistic characteristics, such as expected values, variations and coefficients of variation of the observed parameters (axial forces in tower legs in lower sections) and this is done with the use of the 8th order stochastic perturbation method. Polynomial response functions of the observed design parameters were numerically determined using symbolic algebra system MAPLE, v. 2016 with the Least Squares Method included.

The next step of the analysis has been to determine reliability indices $\beta_{SORM}$ by SORM approach for different input coefficients of variation of the wind velocity $\alpha_{in}(v)$. The limit state function in case of the towers capacity analysis (with revealed "weakest link" equivalent to the buckling capacity of compressed leg) and random wind load, can be expressed in following form:

$$g = F_{b,ex} - F_x, \tag{7}$$

where: $F_x$ is the axial force in extremely compressed tower leg. According to the First Order Reliability Method reliability index was defined as a reciprocal in inverse proportion to the safety margin. We could express it in the following manner:

$$\beta_{FORM} = \frac{E[F_{b,ex}] - E[F_x]}{\sqrt{\sigma[F_{b,ex}]^2 + \sigma[F_x]^2}}, \tag{8}$$

where $E[F_{b,ex}]$ denotes the expected value of experimental buckling resistance, $E[F_x]$ is the expected value of the axial forces in tower leg under compression according to the random wind velocity and $\sigma[F_{b,ex}]$, $\sigma[F_x]$ are the standard deviations of the above variables, respectively.

General formula of the reliability index in the Second Order Reliability Method, applied in numerical example is the following one:

$$\beta_{SORM} = -\Phi^{-1}(P_{f2}), \tag{9}$$

where $P_{f2}$ denotes the probability of failure for the Gaussian probability distribution $\Phi$ of the function related to $\beta_{FORM}$ in the following manner:

$$P_{f2} = \frac{\Phi(\beta_{FORM})}{\sqrt{1 + \beta_{FORM}\kappa}}, \tag{10}$$

where $\kappa$ is curvature of the limit function g (surface) usually defined as

$$\kappa = \frac{\frac{\partial^2 g}{\partial v^2}}{\left(1 + \left(\frac{\partial g}{\partial v}\right)^2\right)^{\frac{3}{2}}}, \text{ where } \kappa > \begin{cases} \dfrac{-1}{\Phi(-\beta_{FORM})} \\ \dfrac{-1}{\beta_{FORM}} \end{cases}. \tag{11}$$

Partial safety factor has been determined from the SORM reliability index as follows

$$\gamma_E = 1 + \beta_{SORM} \cdot |v_E| \cdot \alpha_E, \tag{12}$$

where $|v_E|$ is the sensitivity factor (calibration factor) and $\alpha_E$ is the coefficient of variation for the environmental actions effects. Both parameters are analogous to these described in 3.1, but they concern the axial forces as these action results.

## 4. DISCUSSION OF THE RESULTS

### 4.1. Steel chimney

The first part of computational analysis consisted in assuming the projected reliability level using the reliability index $\beta$, the corresponding partial safety factors for different values of an input coefficients of variation. Sample values of the reliability index equal 2.0, 3.0, 4.0 and 5.0 have been used (Fig. 5).

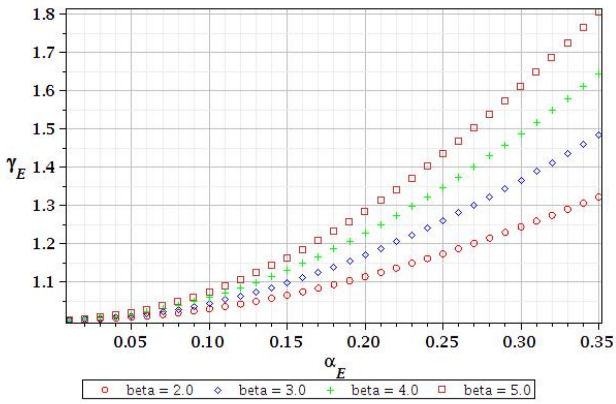


Fig. 5 A relationship between partial safety factors and coefficients of variation for the given reliability levels.

It could be noticed that with coefficient of variation close to 0, which means no uncertainty of the load effect, in any case partial safety factors equal to 1.0. They increase together with an increase of coefficients of variation, and this relationship is nonlinear. A difference between partial safety factors for particular reliability indices is about 5% with coefficient of variation equal 0.20, and about 10% with the value of this ratio equal to 0.35.

Partial safety factors for the given reliability indices contained within the range from 2.0 to 4.0, with the interval of 0.2, and coefficient of variation equal $\alpha_E = 0.15$ are presented in Fig. 6.

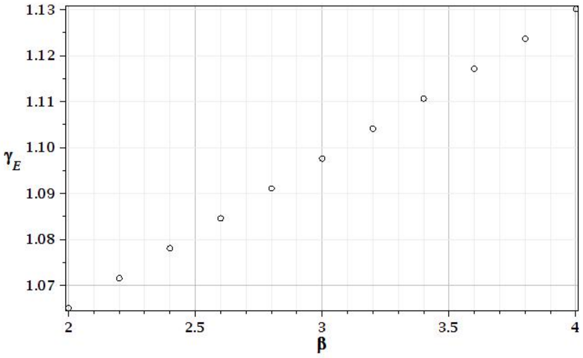


Fig. 6 **A r**elationship between partial safety factors and the given reliability indices for coefficient of variation equal α = 0.15.

It could be observed that an increase of the structure reliability level equals one grade (increase of reliability index $\beta$ equal 1.0) with this coefficient of variation value and it results in relatively small additional increase of the partial safety factor, equal about 0.03. It is worth noting that in case of this value of load effects standard deviation, for reliability index $\beta = 4.0$ partial safety factor is relatively small ($\gamma$ = 1.13). The value of reliability index included in Standard for reliability class RC1 (Fig. 11), which contains low consequence for loss of human life and small economic, social or environmental consequences equals $\beta$ = 3.3, which means that in calculation partial safety factor equal $\gamma = 1.11$ could be used. In the second part of calculations of partial safety factors for reliability indices directly dependent on coefficients of variation have been estimated with the results shown in Figs. 7-8.

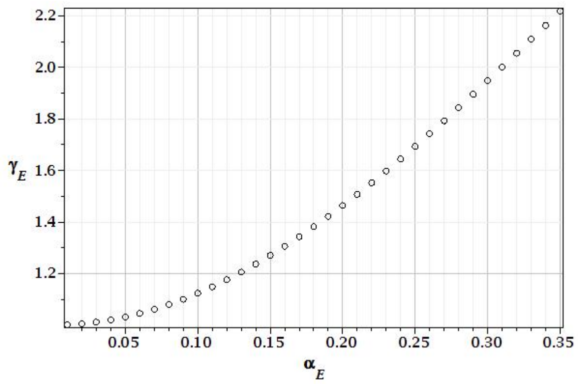


Fig. 7 A relationship between partial safety factors and coefficients of variation for the corresponding reliability levels.

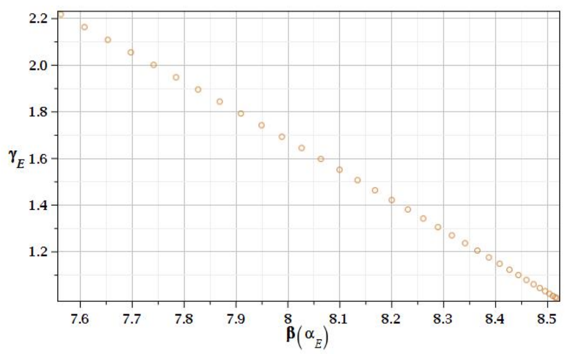


Fig. 8 **A** relationship between partial safety factors and reliability indices for the corresponding coefficients of variation.

As one can see, partial safety factors for the same values of variation coefficients as analyzed in the previous part of the results are higher, which means that also reliability indices calculated based on these coefficients are larger. The value of load effect partial factor for the coefficient of variation equal about 0 is equal $\gamma = 1.0$ as well, but it reaches the value of $\gamma$ = 2.2 with the coefficient of variation equal $\alpha_E = 0.35$, which corresponds to the reliability index value equal to about $\beta = 7.6$. A linear relationship between safety factors and reliability indices for corresponding coefficients of variation could be observed while analyzing Fig. 8. It is noticed that the safety factors decrease with the additional increase of the reliability index. It is due to a direct dependency in-between the reliability index and coefficient of variation - with reliability index equal about 8.52 there is no uncertainty in the environmental load effect (coefficient of variation equals 0).

### 4.2. Telecommunication towers

Comparable results for the analyzed steel lattice telecommunication towers have been obtained. As one can see in Fig. 9a and 9b different values of maximum coefficient of variation could be observed on the horizontal axes. This is due to the fact that the input parameter is the wind velocity coefficient of variation $\alpha_{in}(v)$, and the coefficient of variation for load effects is the resulting value of the stochastic analysis in the analysis conducted for the towers. This is in opposite to the chimney analysis, where the coefficient of variation for load effects ($\alpha_E$) is one of the input parameters. Nevertheless, similar values of partial safety factors for the given reliability levels are obtained for lattice towers under consideration. There is a difference in the corresponding relationship obtained for the chimney, where partial safety factors are lower, and the difference is grater for higher values of variation coefficients with the same random scattering of external load effects.

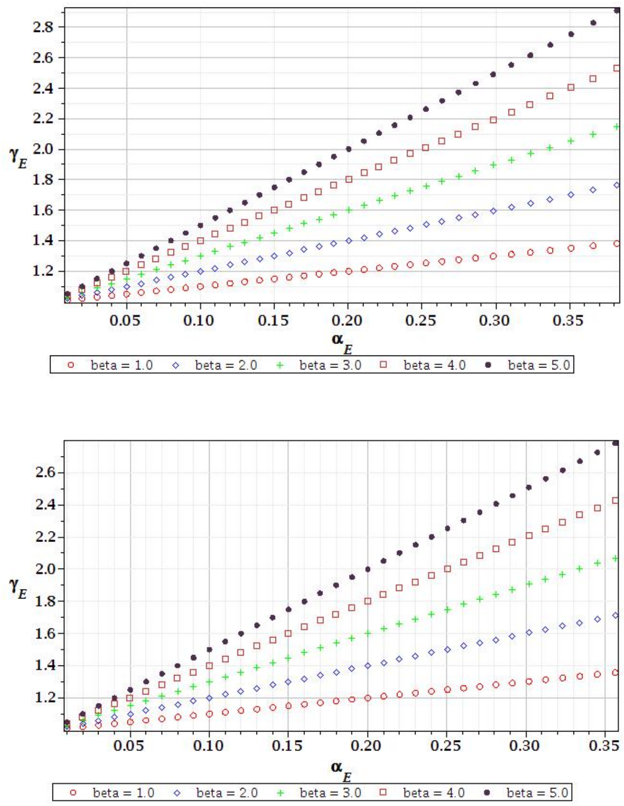


Fig. 9a A relationship between partial safety factors and coefficients of variation for the given reliability levels: 40.0 m tower (top) and 52.0 m tower (bottom).

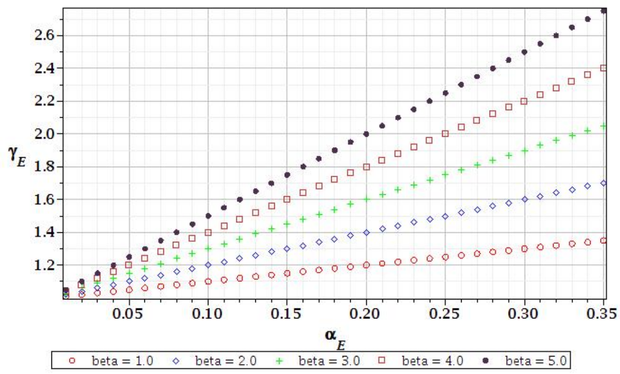


Fig. 9b A relationship between partial safety factors and coefficients of variation for the given reliability levels and 58.0 m tower.

It should be noticed that there is a difference in charts characters for different type of structure – the linear relationships could be observed in case of any tower. Partial safety factors for all three tower structures are similar, and higher than for the chimney analysis (Fig. 10a and 10b) when the reliability indices have fixed values (in the range from 2.0 to 4.0, with the interval of 0.2, as well as in the case of the chimney) and coefficient of variation equal $\alpha_E$ = 0.15. Also the difference in safety factors values is greater for particular values of reliability indices than in case of the chimney - with an increase of reliability index $\beta$ equal to 1.0 the corresponding increment of the partial safety factor equals to 0.15. Very interesting results are obtained for the second approach - for the analysis where all the variables: reliability indices, coefficients of variation and partial safety factors depend on each other (Fig. 11). The relationships between safety factors and coefficients of variation for both cases - the chimney and the towers – are remarkably nonlinear. With similar spread of $\alpha_E$ coefficient there is a great difference between results in $\gamma$ factors obtained for the chimney, where the range in values is equal about 1.2, and for the towers, where it is equal from about 0.17 (52.0 m tower) to 0.3 (58 m tower). Partial safety factors equal 1.0 are observed in the graphs presented in Fig. 11 for 52.0 m and 58.0 m towers for the certain range of the input uncertainty level. These particular results mean that the load capacity of the structure has been exceeded (reliability index is equal $\beta$ = 0, Fig. 12) for the coefficient of variation equal 0.16 for the 52.0 m tower, and equal 0.2 for the 58.0 m one. Load capacity within the entire analyzed spectrum is retained for the smallest tower structure only. The reason of this fact is that the largest difference between environmental loads effects and carrying capacity of the structure resulted in relatively large cross sections of structural elements. The reliability indices for this structure have the largest values, but the safety factors are the highest as well, to ensure an adequate level of structural safety at the same time (Tab. 4).

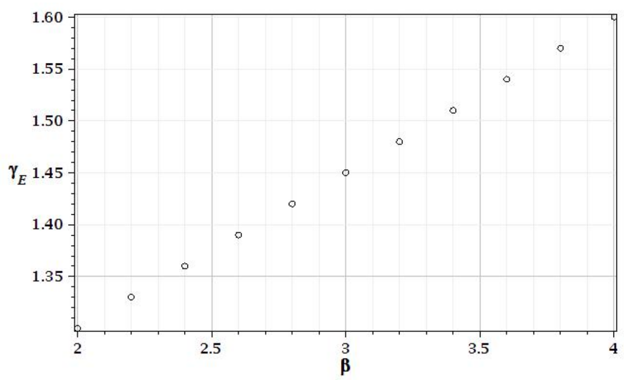


Fig. 10a A relationship between partial safety factors and the given reliability indices for coefficient of variation equal $\alpha_E$ = 0.15 and 40.0 m tower.

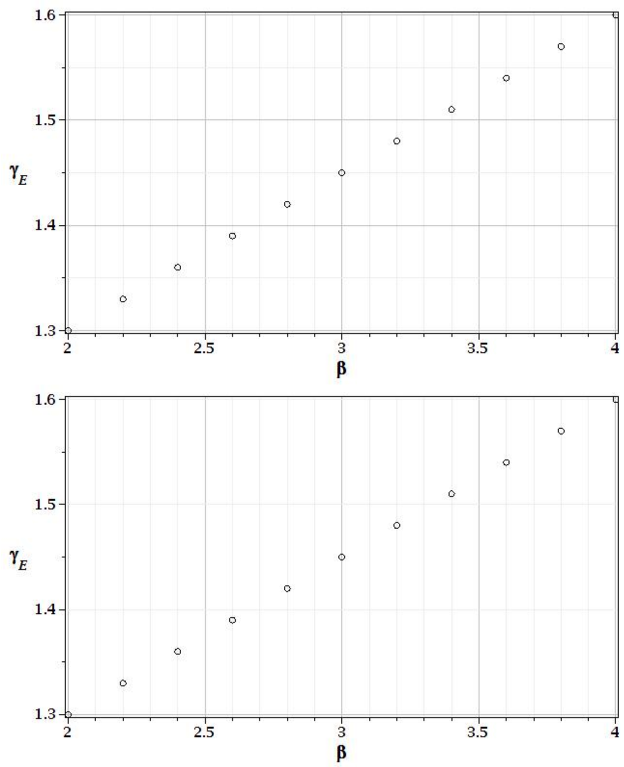


Fig. 10b A relationship between partial safety factors and the given reliability indices for coefficient of variation equal $\alpha_E$ = 0.15: 52.0 m tower (top) and 58.0 m tower (bottom).

Table 4. Reliability indices and partial safety factors for the given structures with coefficient of variation equal $\alpha_E$ = 0.15.

| Structure type | β | γ |
|---|---|---|
| chimney | 8.32 | 1.27 |
| tower 40.0 m | 5.10 | 1.76 |
| tower 52.0 m | 2.17 | 1.33 |
| tower 58.0 m | 2.33 | 1.35 |

Numerical results corresponding to the minimum standard deviation of environmental loads effects are shown in Table 5 are all equal almost 0. The reliability indices (also presented in Fig. 12) have extreme values for 40.0 m tower and the differences between these values and corresponding indices calculated for the other towers are significant. The values obtained here allow to conclude that partial safety factor equals $\gamma$ = 1.0 only for the chimney only, when no randomness in the environmental loads effects is considered. Safety factors for all the remaining case studies are larger than 0, even with minimal coefficients of variation.

Table 5. Reliability indices and partial safety factors for analyzed structures with coefficient of variation equal $\alpha_E$ = 0.01.

| Structure type | β | γ |
|---|---|---|
| chimney | 8.5 | 1.00 |
| tower 40.0 m | 70.9 | 1.71 |
| tower 52.0 m | 15.4 | 1.15 |
| tower 58.0 m | 20.1 | 1.20 |

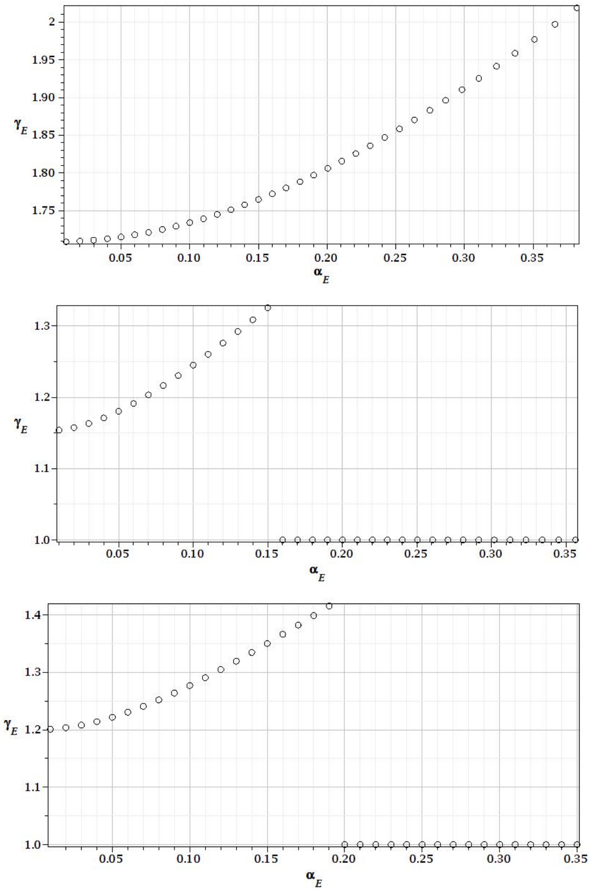


Fig. 11 A relationship between partial safety factors and coefficients of variation for corresponding reliability levels: 40.0 m tower (top), 52.0 m tower (middle) and 58.0 m tower (bottom).

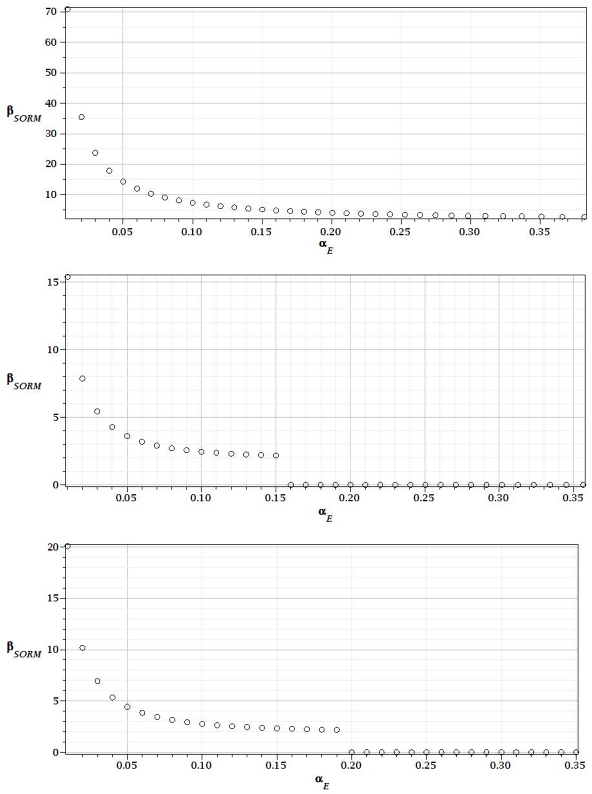


Fig. 12 A relationship between reliability indices and coefficients of variation: 40.0 m tower (top), 52.0 m tower (middle) and 58.0 m tower (bottom).

## 5. CONCLUSIONS

Designing of the structures using partial safety factors based on the SORM reliability indices allow to optimize these structures and to keep the necessary safety margins. This article was entirely devoted to a partial safety factors calibration procedure and its practical application on the examples of steel chimney and lattice telecommunication towers chosen as the very slender structures highly sensitive to Gaussian random wind pressure. The following conclusions could be drawn based on the results obtained:

- a combination of the generalized stochastic perturbation method and safety factors calibration procedure allows to adopt such values of partial safety factors $\gamma_E$ to ensure design reliability level $\beta$,
- this approach allows to notice several important relationships, such as the reliability index $\beta$ as a function of the coefficient $\alpha_E$ (the curves representing $\beta(\alpha_E)$); it is possible to find out in particular, when the reliability level equal $\beta = 0$ is achieved; the loads effects partial safety factors $\gamma_E$ in a function of $\alpha_E$ could be observed as well, where the range of $\gamma_E$ corresponding to the safe usage is presented,
- in case of existing engineering structures it could be stated that there exists critical value of partial factor, which is equivalent to the exceeding of the structural capacity; the approach presented allows to determine the maximum safety level of the given structure,
- the generalized stochastic perturbation method combined with the SORM approach and partial factors calibration procedure allows to obtain the results with relatively small computational effort and time, and apply them further into the engineering designs,
- the highest reliability level has been obtained for the 40.0 m tower structure, and the lowest is for the 52.0 m one,
- taking into account the analysis where reliability indices are established, it is easier to keep the necessary reliability level for the chimney construction than for the considered towers; the partial safety factor could be much lower for this purpose,
- a relationship between partial safety factors and coefficients of variation for towers structures is directly proportional for the given reliability indices,
- a character of the partial safety factors and the coefficients of variations relationship may depend upon the chosen engineering structure and its particular parameters.

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[1]) J. Szafran, email: *jacek.szafran@p.lodz.pl,* POLAND
[2]) K. Juszczyk, email: *kjuszczyk@compactprojekt.pl,* POLAND
[3]) M. Kamiński, email: *marcin.kaminski@p.lodz.pl,* POLAND