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RELIABILITY COMPONENT DIFFERENTIATION IN BUILDING STRUCTURES
MADE OF TIMBER

RÓŻNICOWANIE ELEMENTÓW NIEZAWODNOŚCI BUDOWLANYCH
KONSTRUKCJI DREWNIANYCH

Abstract

The design of timber structures according to the current generation of Eurocodes results in new requirements being set for the authors of architectural and building designs with respect to the reliability management of such structures. The reliability problems should be formulated in the building permit design in an unequivocal manner, obliging the authors of the detailed design, as well as the contractors to deliver structures, which have all the operational parameters fully conforming to the expectations of the investor. Substantive and formal basics in this regard are formulated in the Eurocodes: PN-EN 1990, PN-EN 1991, PN-EN 1995 as well as in the related European codes. The reliability management problems of contemporary timber structures are related to the cubature buildings of different life spans, including buildings subjected to the climate loads characterised by several hundred year-long return periods.

Keywords: timber, bearing capacity, reliability, destruction consequence classes, reliability classes, return period, reference period, loads

Streszczenie

Projektowanie konstrukcji drewnianych wg współczesnej generacji norm europejskich stawia przed autorami projektów architektoniczno-budowlanych nowe wymagania w zakresie zarządzania niezawodnością takich obiektów. Problemy niezawodności należy sformułować w projekcie budowlanym w sposób jednoznaczny, zobowiązujący autorów projektów wykonawczych, a także firmy wykonawcze do dostarczenia konstrukcji o parametrach eksploatacyjnych zgodnych z oczekiwaniami inwestora. Podstawy merytoryczne i formalne w tym zakresie są sformułowane w Eurokodach: PN-EN 1990, PN-EN 1991, PN-EN 1995 oraz w europejskich normach pokrewnych. Problemy zarządzania niezawodnością współczesnych konstrukcji drewnianych odniesiono do przypadków budynków kubaturowych o zróżnicowanym okresie użytkowania, w tym poddanym oddziaływaniom klimatycznym o okresie powrotu nawet kilkuset lat.

Słowa kluczowe: drewno, nośność, niezawodność, klasy konsekwencji zniszczenia, klasy niezawodności, okres powrotu, okres odniesienia, obciążenia

1. Introduction

The scope and form of the building permit design according to the Polish regulations [10], treats the reliability of buildings problem in a rather general manner. This makes it possible for the construction companies to erect even large structures according to the so-called replacement designs, even in the cases when the investor delivers his own detailed design with the approved building permit design. In many cases known to the Author, the cubature buildings erected according to the replacement designs have lowered reliability requirements as compared to the investors' original designs. In addition, looking for savings on materials, the authors of replacement designs often select prototypical design solutions, thus posing additional threats to the safe operation. Many buildings erected in the 1990's, such as large area halls covered with roofs of steel or a timber structure may be treated as a negative example of such an approach. The reliability requirements specified for such buildings in the technical description attached to the building permit design most often are limited to a brief reference to the country codes, thus making it possible to underestimate the operating parameters, especially in the area of snow loads. The underestimated cross sections of the roof structures resulted in a permanent need for snow removal, and the costs, which have to be borne by the users, have been incommensurable with the ad hoc benefits gained by the building contractors erecting such structures.

Table 1. Designed life span categories according to PN-EN 1990

| Designed life span category | Designed life span T_d in years | Sample structures |
|-----------------------------|-----------------------------------|-------------------------------------|
| (1) | (2) | (3) |
| 1 | 10 | Temporary structures |
| 2 | 10–25 | Replaceable elements |
| 3 | 15–30 | Agricultural structures and similar |
| 4 | 50 | Ordinary buildings |
| 5 | ≥ 100 | Monumental buildings, bridges |

Table 2. Values of conversion coefficients for climate loads according to own research

| Return period n [years] | Conversion coefficients η_d | | | |
|---------------------------|--|-------|------------|------------|
| | Actions on structures: snow, wind, temperature | | | |
| | s_k | v_b | T_{\max} | T_{\min} |
| (1) | (2) | (3) | (4) | (5) |
| 10 | 0.70 | 0.90 | 0.91 | 0.74 |
| 15 | 0.77 | 0.93 | 0.93 | 0.81 |
| 25 | 0.87 | 0.96 | 0.96 | 0.89 |
| 30 | 0.90 | 0.97 | 0.97 | 0.92 |
| 50 | 1.00 | 1.00 | 1.00 | 1.00 |
| 100 | 1.13 | 1.04 | 1.04 | 1.11 |
| 300 | 1.33 | 1.10 | 1.10 | 1.28 |
| 500 | 1.42 | 1.12 | 1.13 | 1.36 |

The Ordinance issued by the Minister of Infrastructure on July 3rd, 2003, amending the detailed scope and form of the building permit design, did not introduce any qualitative changes in the area of building reliability management against the previous regulations introduced in 1998 (compare the Polish Law Register no 140, pos. 906). Amendments to the scope of building permit design, taking into account the reliability management rules for the cubature structures according to the Eurocode PN-EN 1990, constitute an efficient remedy against the unfavourable processes described above.

The design of any building structure requires the designed life of the structure T_d to be set, i.e. the time span during which the structure or a component thereof is to serve as intended while subjected to expected maintenance, without the need for major repairs.

The systematic subdivision of the designed life spans into five categories has been introduced in the code PN-EN 1990. This subdivision is listed in Table 1. In most cases building structures are assigned to the fourth category, corresponding to a 50 year-long service time. If the buildings are designed with other assumed service times, the characteristic load values should be corrected, especially the climate loads $\eta_d F_k$, where η_d – a correction factor. Values of the reduction coefficients η_d for various return periods of maximum climate loads, according to the own research [5] are given in Table 2.

2. Analysis of the reliability components for timber structures

In order to differentiate the reliability of the designed building structures the code PN-EN 1990 defines three destruction consequence classes (CC), according to the description specified in Table B1 of the said code. Consequence classes are related to the reliability classes (RC) of structures in such a way, that the consequence class CC3 corresponds to the reliability class RC3, class CC2 – RC2 and class CC1 – RC1. The reliability classes (RC) of structures in the bearing capacity limit state have been defined depending on the recommended minimum value of the reliability factor β_u set for the reference period $t = 1$ year additionally, in the code PN-EN 1990 one may find the values of the reliability factor determined according to the formula (1) for $t = 50$ (cf. columns (2) and (7) in Table 3). The reliability factor β (Hasofer, Lind [8]) represents an idea well known in the reliability theory of building structures [1–4], [7–10], [11, 12] and is defined in the probabilistic calculation method of the second level FORM (*First Order Reliability Method*). In particular, the coefficient β is a measure of reliability, which may be specified in the statistical research for random loads and random bearing capacity of the structure. The coefficient β is related to the failure probability P_f of a structure by the following formula:

$$P_f = \Phi(-\beta), \quad (1)$$

where:

Φ – is the Laplace function of the probability distribution of standardised normal distribution, as depicted in Fig. 1.



For the mutually independent random maxima, one may assume that the following relationship holds between the reliability factors specified for the reference periods $T = n$ and $t = 1$ (cf. code PN-EN 1990):

$$\Phi(\beta_n) = \Phi(\beta_1)^n \quad (2)$$

For the selected reference periods $n = 10; 15; 25; 30; 50; 100; 300$ and 500 – years, the curves according to (2) [6] are compiled in Fig. 2. In addition, the values of reliability factor β_n calculated for these periods in the bearing capacity limit state (LS) for the three reliability classes (RC) are represented in Table 3.

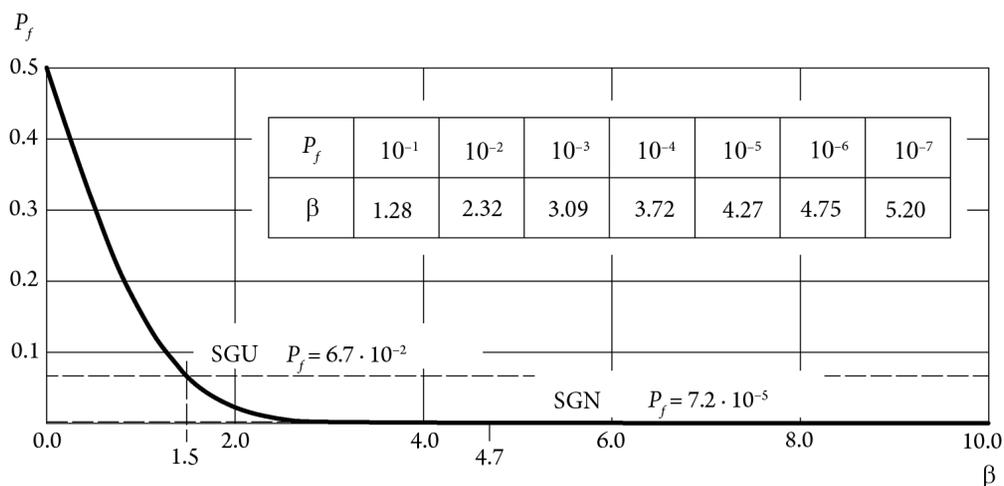


Fig. 1. Plot of the $P_f - \beta$ relationship [6]

In the probabilistic method of the second level, in the case of a linear bearing capacity function:

$$g = R - E, \quad (3)$$

the structural reliability criterion may be expressed by the following inequality

$$\beta = \frac{\bar{R} - \bar{E}}{\sqrt{\mu_R^2 + \mu_E^2}} \geq \beta_u, \quad (4)$$

where:

- \bar{R}, \bar{E} – average values,
- μ_R, μ_E – standard deviations of random bearing capacities R and load effects E , respectively.

In the basic case the inequality (4) may be replaced by the comparison of computational values: bearing capacity R_d and the corresponding load effect E_d .

$$R_d = \bar{R} - \beta_R \mu_R \geq E_d = \bar{E} + \beta_E \mu_E, \quad (5)$$

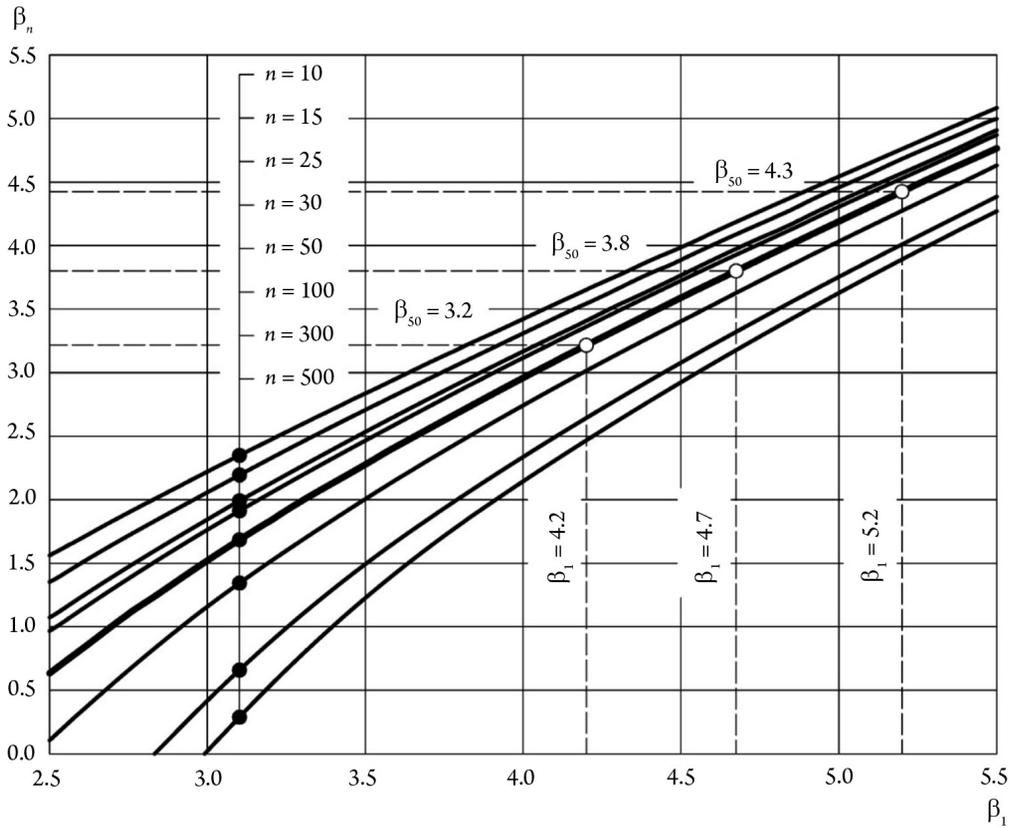


Fig. 2. Plots of the $\beta_1 - \beta_n$ relationships for the selected return periods n (own research)

Table 3. Minimum values of the reliability factor β_u in the bearing capacity limit case for the reliability classes RC1, RC2 and RC3 according to own research.

| Reliability factor $\beta_n = \beta_u$ | | | | | | | | | |
|--|-------------|------|------|------|------|-------------|------|------|------|
| Reference period n in years | | | | | | | | | |
| (RC) | 1 | 10 | 15 | 25 | 30 | 50 | 100 | 300 | 500 |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) |
| RC1 | 4.20 | 3.65 | 3.54 | 3.40 | 3.35 | 3.21 | 3.00 | 2.65 | 2.48 |
| RC2 | 4.70 | 4.21 | 4.11 | 3.99 | 3.95 | 3.83 | 3.65 | 3.36 | 3.22 |
| RC3 | 5.20 | 4.75 | 4.67 | 4.50 | 4.53 | 4.42 | 4.27 | 4.01 | 3.89 |

If the inequality (5) is satisfied, the second level criterion is satisfied as well, subject to the condition that partial coefficients are assumed for β_E and β_R . These coefficients are related to the reliability factor β by the relationships: $\beta_R = \beta|\alpha_R|$, $\beta_E = \beta|\alpha_E|$, where the multipliers $\alpha_R = 0.8$, and $\alpha_E = -0.7$ denote the sensitivity factors having the values listed in the code PN-EN 1990.

The design codes for building structures are usually calibrated for structures of average reliability requirements, i.e. for the RC 2 class. Assuming the specification of partial coefficients assigned to the bearing capacity γ_M for reliability classes other than RC 2 according to the Eurocode 5 one should apply a correction factor K_R of the form listed below to the left side of the formula (5):

$$\bar{R} - 0.8\beta_{RC2}\mu_R = K_R(\bar{R} - 0.8\beta_{RC}\mu_R), \quad (6)$$

thus

$$\frac{1 - 0.8\beta_{RC2}v_R}{1 - 0.8\beta_{RC}v_R}, \quad (7)$$

where:

$v_R = \mu_R / \bar{R}$ – timber strength random variation coefficient.

Analogous reasoning may be presented for variable loads Q (having the average value \bar{Q} and standard deviation μ_Q), present on the right-hand side of the formula (5):

$$K_{Fi}(\bar{Q} + 0.7\beta_{RC2}\mu_Q) = \bar{Q} + 0.7\beta_{RC}\mu_Q, \quad (8)$$

$$\frac{1 + 0.7\beta_{RC}v_Q}{1 + 0.7\beta_{RC2}v_Q}, \quad (9)$$

where:

$v_Q = \mu_Q / \bar{Q}$ – random load Q variation coefficient.

For the structure belonging to the RC 3 class, designed for the sample reference period $T = 50$ years, the reliability coefficients according to Table 3 are equal to: $\beta_{RC2} = 3.83$ and $\beta_{RC} = 4.42$, respectively; thus, formulas (7) and (9) yield the estimates of the reduction coefficients for bearing capacity and loads:

$$K_R = \frac{1 - 0.8 \cdot 3.83 v_R}{1 - 0.8 \cdot 4.42 v_R} = \frac{1 - 3.06 v_R}{1 - 3.54 v_R} \quad (10)$$

$$K_{Fi} = \frac{1 + 0.7 \cdot 4.42 v_Q}{1 + 0.7 \cdot 3.83 v_Q} = \frac{1 + 3.09 v_Q}{1 + 2.68 v_Q} \quad (11)$$

For the class RC 1, the reliability coefficients according to Table 3 are equal to: $\beta_{RC2} = 3.83$ and $\beta_{RC} = 3.21$, respectively; and thus, formulas (7) and (9) yield the estimates of reduction coefficients:

$$K_R = \frac{1 - 0.8 \cdot 3.83 v_R}{1 - 0.8 \cdot 3.21 v_R} = \frac{1 - 3.06 v_R}{1 - 2.57 v_R} \quad (12)$$

$$K_{Fi} = \frac{1 + 0.7 \cdot 3.21 v_Q}{1 + 0.7 \cdot 3.83 v_Q} = \frac{1 + 2.25 v_Q}{1 + 2.68 v_Q}. \quad (13)$$

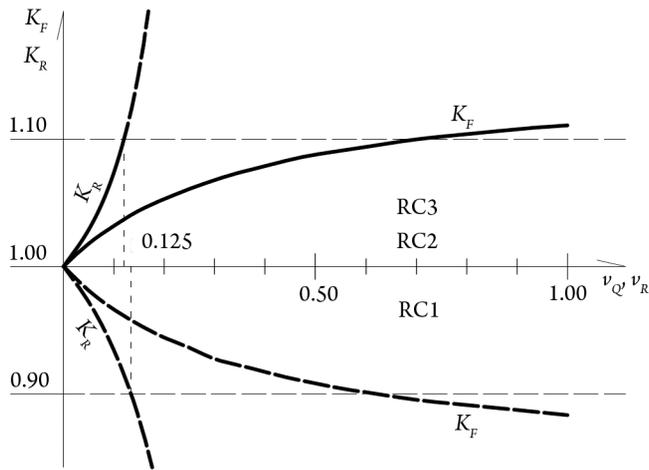


Fig. 3. Plots of the reduction coefficients K_R and K_{Fi} according to the formulas (10)÷(13) (own research)

Graphs of reduction coefficients plotted against the material strength variation coefficient v_R and variable loads variation coefficient v_Q are depicted in Fig. 3. The example values of those coefficients, calculated for the reference periods of $T = 50$ years and $T = 300$ years are listed in Table 4. Comparison of numerical data for both periods shows, that quantitative results are convergent, while K_F values recommended in the code PN-EN 1990 and listed in the Table 5 are fully justified for the values presented in Table 4, rounded up to 0.1. One should note, that even large values of the climate loads variation coefficient, for instance describing the uneven snow load on the ground (cf. [13]), for which $v_Q = 0.60 \div 1.00$, lead to differences in the reliability measure limited to about 10%.

Table 4. Values of reduction coefficients K_{Fi} and K_R calculated for the reference periods of $T = 50$ years and $T = 300$ years according to own research

| Reliability class RC3; reference period $T = 50$ year | | | | | | | | |
|--|-------|-------|-------|-------|-------|-------|-------|-------|
| v | 0.05 | 0.100 | 0.150 | 0.200 | 0.400 | 0.600 | 0.800 | 1.000 |
| K_F | 1.018 | 1.032 | 1.044 | 1.053 | 1.079 | 1.094 | 1.104 | 1.111 |
| K_R | 1.029 | 1.074 | 1.154 | 1.329 | | | | |
| Reliability class RC1; reference period $T = 50$ year | | | | | | | | |
| v | 0.05 | 0.100 | 0.150 | 0.200 | 0.400 | 0.600 | 0.800 | 1.000 |
| K_F | 0.981 | 0.966 | 0.954 | 0.944 | 0.917 | 0.901 | 0.891 | 0.883 |
| K_R | 0.972 | 0.934 | 0.880 | 0.798 | | | | |
| Reliability class RC3; reference period $T = 300$ year | | | | | | | | |
| v | 0.05 | 0.100 | 0.150 | 0.200 | 0.400 | 0.600 | 0.800 | 1.000 |
| K_F | 1.023 | 1.037 | 1.055 | 1.062 | 1.094 | 1.113 | 1.126 | 1.136 |
| K_R | 1.031 | 1.077 | 1.150 | 1.290 | | | | |

| Reliability class RC1; reference period $T = 300$ year | | | | | | | | |
|--|-------|-------|-------|-------|-------|-------|-------|-------|
| ν | 0.05 | 0.100 | 0.150 | 0.200 | 0.400 | 0.600 | 0.800 | 1.000 |
| K_F | 0.978 | 0.960 | 0.945 | 0.932 | 0.898 | 0.876 | 0.862 | 0.852 |
| K_R | 0.968 | 0.928 | 0.875 | 0.803 | | | | |

Table 5. Values of coefficients K_{Fi} for actions according to the code PN-EN 1990

| Correction factor | Reliability class | | |
|-------------------|-------------------|-----|-----|
| | RC1 | RC2 | RC3 |
| (1) | (2) | (3) | (4) |
| K_{Fi} | 0.9 | 1.0 | 1.1 |

Correction of the load coefficients γ_F via the correction factors K_{Fi} having the values listed in Table 5 constitutes a simple engineering method of differentiating the reliability requirements with respect to the variable loads according to the code PN-EN 1990.

In the recommendations of the code PN-EN 1990 pertaining to the design basics of building structures, a case is considered, where the conditions of the limit state (5) may be expressed by the bearing capacity R and the effect of actions E associated with it in the following form:

$$E_d = E\{F_{di}, a_{di}, \theta_{di}\} \leq R_d = R\{X_{dj}, a_{dj}, \theta_{dj}\}, \quad (14)$$

where index „d” denotes the computational values of:

- F_{di} – actions on the structure, $i = 1, 2, \dots, n$,
- X_{dj} – mechanical properties of the structural material, $j = 1, 2, \dots, m$,
- a_{di}, a_{dj} – geometrical properties of the structure,
- θ_{di}, θ_{dj} – uncertainty parameters of the computational model.

In the code PN-EN 1990, a reliability verification convention has been assumed, according to which the computational values X_d and F_d are usually not entered directly into the limit state equation, but the so-called *representative values* X_{rep} and F_{rep} are used instead. The following may be used as representative values:

- ▶ characteristic values, i.e. quintiles for: loads – ηF_k , material strength – ηX_k , and geometrical properties – a_d (where η – conversion coefficients),
- ▶ nominal values (central values of geometrical properties a_{nom}).

The computational values F_d and X_d are determined via the multiplication or division of representative values by the applied partial coefficients:

$$F_d = F_{rep} \gamma_F \rightarrow E_d = E(\eta F_k \gamma_{F'} a_d), \quad (15)$$

$$X_d = \eta X_k / \gamma_M \rightarrow R_d = R(\eta X_k / \gamma_{M'} a_d). \quad (16)$$

The partial coefficients – γ_F in the formula (15) and γ_M in the formula (16) – account for the random variation of actions (factor γ_F), material strength (factor γ_M) and error in the modelling of these random variables (factors γ_{sd} and γ_{Rd} , respectively):

$$\gamma_F = \gamma_f \gamma_{sd} \gamma_M = \gamma_m \gamma_{Rd} \quad (17)$$

The structure of partial coefficients according to the formulas (17) explains the basic difference between the limit states method, as applied in the Polish codes PN/B, and the load γ_F and bearing capacity γ_M coefficients method introduced in the Eurocodes. In the European codes perfect modelling of mechanical systems is postulated. Perfect modelling of timber structures results in the need for a numerical model, usually fully 3D, accounting for multi-sourced global and local imperfections. As a consequence of such an approach, the values of bearing capacity coefficients may be lowered with respect to the values known from Polish PN/B codes, since the modelling error of the structure may be justifiably assumed as: $\gamma_{Rd} = 1.0$.

A different interpretation pertains to the modelling of loads, for which $\gamma_{sd} \geq 1.0$, as all the forecasts of technological, climate and other actions on the structure are inherently burdened with an error.

The values of bearing capacity coefficients for wooden structural components are listed in the part 1-1 of the Eurocode 5 as follows:

- ▶ bearing capacity coefficient for components made of solid wood, wooden particle boards and beaverboards $\gamma_M = 1.30$;
- ▶ bearing capacity coefficient for components made of glued laminated timber and barbed plate $\gamma_M = 1.25$;
- ▶ bearing capacity coefficient for components made of plywood, laminated veneer lumber (LVL) and oriented strand boards (OSB) $\gamma_M = 1.20$.

The dependence of the bearing capacity coefficient γ_M on the variance coefficient of timber strength (cf. PN-EN 1995-1-5) follows from the formula (16):

$$\gamma_M = \frac{R_k}{R_d} = \frac{1 - 1.64v_R}{1 - 3.04v_R} \quad (18)$$

where:

R_k, R_d – lower quintiles of the timber bearing capacity at the probability level ω – 5% and ω – 1.35%, respectively.

For the bearing capacity coefficient $g_M = 1.20$, one obtains from the formula (18) the value of variance coefficient $v_R = 0.100$, and for $g_M = 1.30$ the value of $v_R = 0.130$. According to Table 4, these estimates correspond to the timber strength reduction factor $K_R \approx 1.1$ for the structures of the required reliability class RC3 and $K_R \approx 0.9$ for the reliability class RC1. For highly homogeneous structural materials, such as for instance steel, for which $v_R \leq 0.075$, the reduction coefficients assume the values of $K_R \leq 1.05$ or $K_R \geq 0.95$ for the classes RC3 and RC1, respectively, according to Table 4.



3. An example of the structural components reliability verification

The building structure reliability differentiation method is presented on the example of a timber structure covering the sanctuary in Łagiewniki, erected in Cracow in the years 1999–2002, cf. Fig. 4. Up to 5,000 people may stay in this building at one time; – this justifies the application of destruction consequence class CC3 and the reliability class RC3.

The nominal service time of this structure has been assumed as equal to $T_d = 300$ years. For this service time one may find in Table 2 the conversion coefficients for climate loads: $\eta_d = 1.33$ for snow and $\eta_d = 1.10$ for wind speed.

The corrective multiplier for climate loads, according to Table 5, is equal to $K_{Fi} = 1.1$, and for the roof structure made of glued laminated timber the corrective coefficient has been assumed according to Fig. 3, with a value of $K_R = 1.1$. The above values result in the following values of load and bearing capacity factors:

- ▶ for permanent loads $\gamma_G = 1.35$,
- ▶ for variable loads $K_{Fi} \gamma_Q = 1.1 \cdot 1.50 = 1.65$,
- ▶ for strength $K_R \gamma_M = 1.1 \cdot 1.25 = 1.375$.

Characteristic actions are subject to correction as well, and especially:

- ▶ base value of wind speed pressure according to PN-EN 1991-1-4

$$q_b = 0.5 \cdot 1.25 \cdot 10^{-3} (1.10 \cdot 22)^2 = 0.366 \text{ kN/m}^2,$$

- ▶ characteristic value of the snow load on the ground

$$s_k = 1.33 \cdot 1.2 = 1.60 \text{ kN/m}^2.$$

The code values of snow load on the ground, assumed for the building structures assigned to the destruction consequence class CC3, by recommendation should be verified by the statistical forecast.

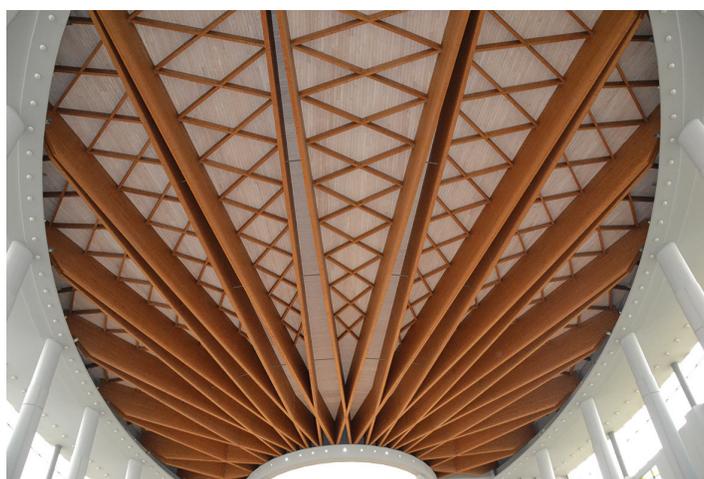


Fig. 4. The glued laminated timber structure of a roof covering the sanctuary in Łagiewniki. Source: own research

The results of snow load measurements recorded by Polish meteorological stations are documented in [13]. Especially the measurement results for the Cracow Balice station are depicted in Fig. 5, and the 50 years forecast prepared on the basis of those measurements is depicted in Fig. 6.

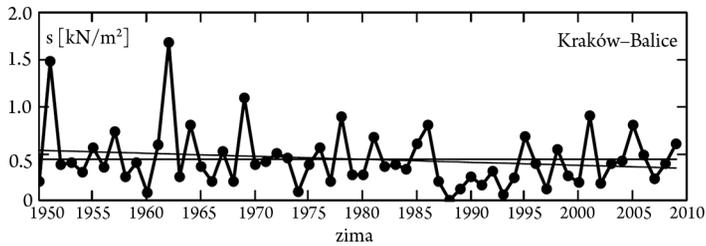


Fig. 5. Results of statistical analyses of snow load on the ground for the 1950–2010 time period at the Cracow Balice meteorological station [13]

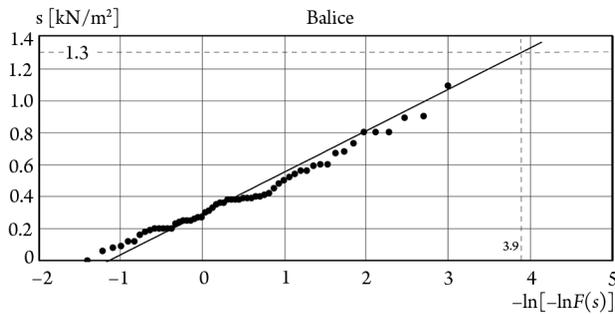


Fig. 6. Statistical forecast of the snow load on the ground for Cracow Balice (own research)

4. Conclusions

Timber structures are especially prone to the differentiation of reliability requirements, as most often these structures are designed as temporary ones, for which the values of reliability measures may and shall be lowered. At the same time, because of high aesthetical values, such structures are willingly applied in many prestigious buildings, to which, as has been presented above, the raised reliability requirements may apply. The verification of the code procedure for the differentiation of reliability requirements performed in this paper has shown, that the reduction coefficient for variable loads K_{Fi} has been correctly specified in the code PN-EN 1990. However, the same verification indicates that for the timber structures belonging to the RC3 class, a value of the bearing capacity reduction coefficient $K_R > 1$ is fully justified (a value of $K_R = 1.10$ is suggested). Additionally, for this reliability class, a statistical verification of the snow loads on the ground assumed according to the code is advised. Long-term observations and measurements performed in 115 meteorological stations, located all over Poland have been

compiled, statistically elaborated and published by the Building Research Institute in Warsaw [13]. In the example considered here, based on Fig. 6, the 50-year forecast indicates the value of $s_k = 1.3 \text{ kN/m}^2 > 1.2 \text{ kN/m}^2$ (ordinate on the plot corresponding to the abscissa equal to 3.9).

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