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RELIABILITY LEVEL OF REINFORCED CONCRETE MEMBERS DESIGNED ACCORDING TO BELARUSSIAN NATIONAL ANNEXES TO STRUCTURAL EUROCODES

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Abstacts: The article presents the results of reliability analysis of reinforced concrete structures designed and built in accordance with design codes that are valid in Belarus. It is noted that such structures have different reliability levels as well as failure probabilities. Approaches to assessment actions on structures, which stated in European and Belarusian codes were analysed. It is shown that in most cases codes, which are used for designing of the existing structures do not meet the modern requirements for safety of structures. Additionally the results of reliability-based calibration of partial factors using in precast concrete members design are presented. The calibration resulted in the reduced value of partial factors for permanent loads on precast elements

Key words: reliability, limit state, partial factor, load, probabilistic model, calibration.
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1. Introduction

According to the actual concept of structural reliability stated in the international standard ISO 2394 structures and structural elements should be designed basing on the standardized target reliability parameters that are expressed in terms of permissible probability of failure P_f or in terms of reliability indices β. Therefore, the comparison of all the standards based on the numerical values of β seems to be the most objective.

The rules for estimating the characteristic values of all the basic variables together with the system of partial factors (also called safety factors) and actions combination factors create the safety margin for structures. Ideally, it should correspond to the target reliability levels stated in structural codes.

There are a number of publications devoted to reliability level assessment for different countries (Faber and Sørensen, 2002; Sýkora and Holický, 2011; etc.). The mentioned publications aimed to assess existing reliability level and to calibrate some partial factors within the bound of Eurocodes, considering its unified rules and approaches to assess loads and combine them.

Performing the same study, namely to assess the reliability level of structures designed in accordance with Eurocodes is needed for the national conditions. It will assess the level of reliability of new and existing structures.

The standardized approaches for assessing actions on structures have an essential influence on reliability level. The comparative in-depth analysis of all the mentioned standards regulating the rules for assessing loads from the position of the reliability theory has not been carried out till this moment.

Moreover, there is a lack of data on reliability levels of structures designed and erected by former USSR Codes, as well by modern Belarusian and Ukrainian Structural Codes. The main challenge of such study consists in creating the base for comparing different standards. It should be point that the considered standards comprise completely different rules for deriving design combinations of loads on structures.

The aim of the present paper is to estimate the level of design reliability of structures (provided by using a system of partial factors and combination factors for loads and resistance of structures) in persistent design situations, according to the design codes that have been valid in the Republic of Belarus for the last decade. The following problems must to be consider for this purpose:

- to formulate the state functions for structural elements that allow considering different ratios of permanent, live, and snow loads;
- to develop the probabilistic models of basic variables contained in the state functions;

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– to estimate the reliability level of structures, designed in accordance with different standards. At that, different systems of safety factors and combination factors as well as the difference in combination rules for loads should be taken into account;
– to perform reliability-based calibration of the partial factor for self weight of precast structural elements.

2. Analysis of the Codes provisions for actions assessing

The comparative analysis of the codes regulating the rules for assessing loads while designing the reinforced concrete structures is carried out in this article.

Three groups of standards are valid in the Republic of Belarus at present. These are:

– Eurocodes TCP-EN 1990 – TCP-EN 1991 (*hereinafter referred to as Eurocodes*);
– Belarusian National Annex to TCP-EN 1992-1-1;
– Design code SNiP 2.01.07-85 “Loads and actions”.

It should be pointed that there are inconsistencies in the classification of actions, in values of partial factors for actions, in combination rules for actions (effects of actions) for *Ultimate* as well as *Serviceability Limit State* design of structures.

For comparison, the Design Code DBN B.1.2-2-2006 «Loads and actions» is also analyzed in the article. This document mainly repeats the concept and content of SNiP, but also contains some approaches similar to those used in EN 1991.

The Eurocodes and SNiP 2.01.07-85 are of different generations of standards, and the requirements to safety level for SNiP are already out of date. They are both based on Limit State Design principles. A system of partial factors and combination factors makes it possible to present limit state functions in a semi-probabilistic form. However, there are certain differences both in the rules for deriving design combinations of loads on structures, and in numerical values of partial safety factors γ and combination factors ψ.

The Design Code DBN B.1.2-2-2006 «Loads and actions» mainly repeats the concept of SNiP 2.01.07-85, but for evaluation of the characteristic values of snow and wind loads an approach similar to the one used in TCP EN 1990 and TCP EN 1991-1-3 is applied.

The rules for deriving design combinations of loads on structures in persistent design situations presented in Table 1. The combination of the permanent, live (imposed), and snow loads is considered.

Besides the differences shown in Table 1 it should be stipulated that coefficients γ and ψ have disparate treatment and mathematical concept within the bounds of corresponding standards. As well, there are distinctions in loads classification and in method of setting characteristic values of loads and actions. These aspects listed in Table 2.

One can see in Table 2 that there is a significant difference between the approaches to setting characteristic values of loads. The partial factor *for permanent loads* γ_G in Eurocodes has a greater value, but it should be use together with combination coefficient ξ that is not specified in the other two groups of standards. Another important difference comes from the fact that within the bounds of SNiP the factor γ_G has a physical meaning of overload factor, and its value is assigned using this consideration.

A striking difference in approaches to setting characteristic values *for snow loads* should be noted: in EN 1991-1-3 the characteristic value is the value which on average is exceeded once in 50 year. An analogous approach accepted in Design Code DBN B1.2-2:2006. Meanwhile, within the bounds of SNiP 2.01.07-85, the characteristic value of a snow load is the mean value of 1-year maximums.

Wind loads not considered in this paper because the approaches to setting characteristic values of wind load are similar to the ones just described.

According to SNiP 2.01.07-85 and DBN B1.2-2:2006, in contrast to Eurocodes, *variable live loads* divided into full and reduced values. The ratio of full and reduced values in Table 2 estimated using the characteristic values of live loads on floor slabs in residential buildings (given in SNiP and DBN).

Table 1. The rules for deriving design combinations of loads on structures in persistent design situations.

Standard	Design value of load effect on a structure or a structural element
TCP-EN 1990:2002	$\max\begin{cases}\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{0,Q} \cdot Q_k + \gamma_S \cdot \psi_{0,S} \cdot S_k \\ \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot Q_k + \gamma_S \cdot \psi_{0,S} \cdot S_k \\ \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot \psi_{0,Q} \cdot Q_k + \gamma_S \cdot S_k\end{cases}$
SNiP 2.01.07-85	$\max\begin{cases}\left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,reduced} \cdot Q_k^{(reduced)} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n \\ \left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,full} \cdot Q_k^{(full)} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n\end{cases}$
DBN B.1.2-2-2006	$\max\begin{cases}\left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,reduced} \cdot Q_k^{(reduced)} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n \\ \left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,full} \cdot Q_k^{(full)} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n\end{cases}$

Explanation: detailed symbol definitions may be found in Table 2.

Table 2. The comparison of approaches to setting characteristic values of loads in the structural codes.

	Eurocodes	SNiP 2.01.07-1985	DBN B.1.2-2-2006
		Permanent load	
Characteristic value	$G_k = E[G]$	$G_k = E[G]$	$G_k = E[G]$
Partial safety factor	$\gamma_G = 1.35$	$\gamma_G = 1.1$	$\gamma_G = 1.1$
Combination factor	$\xi = 0.85$	–	–
		Snow load	
Characteristic value	$S_k = E[S_{\max}]$ for $T = 50$ years	$S_k = E[S_{\max}]$ for $T = 1$ year	$S_k = E[S_{\max}]$ for $T = 50$ years
Partial safety factor	$\gamma_S = 1.5$	$\gamma_S = 1.4$ when $(G_k + Q_k) / S_k \geq 0.8$ $\gamma_S = 1.6$ when $(G_k + Q_k) / S_k < 0.8$	$\gamma_S = 1.0$
Combination factor	$\psi_{0,S} = 0.6$	$\psi_S = 0.9$	$\psi_S = 0.9$
		Variable (live) load	
Characteristic value	Q_k	$Q_k^{(\text{full})} = Q_k$ $Q_k^{(\text{reduced})} = 0.2Q_k$	$Q_k^{(\text{full})} = Q_k$ $Q_k^{(\text{reduced})} = 0.23Q_k$
Partial safety factor	$\gamma_Q = 1.5$	$\gamma_Q = 1.3$	$\gamma_Q = 1.3$
Combination factor	$\psi_{0,Q} = 0.7$	$\psi_{Q,\text{full}} = 0.9$ $\psi_{Q,\text{reduced}} = 0.95$	$\psi_{Q,\text{ full}} = 0.9$ $\psi_{Q,\text{reduced}} = 0.95$
Reliability coefficient depending on importance of a structure	–	$\gamma_n = 0.95$	$\gamma_n = 0.95$

Explanation: 1) operator E […] means the mathematical expectation of a parameter; 2) subscript k (e.g. in Q_k) means the characteristic value; 3) return period T is a statistical measurement based on historic data denoting the average recurrence interval over an extended period of time for an event.

3. Reliability models

In the fundamental case the state function (or the failure function) of a structure comprises two groups of *basic variables*, namely R (related to resistance of the structure), and L (related to the loads on the structure). A state function can be formulated as:

$$g(R, L) = R - L \quad (1)$$

The probability of failure of the structure may be assessed through

$$P_f = \text{Probability}[g(R, L) \leq 0] = \text{Probability}[R - L \leq 0] \quad (2)$$

The reliability index β is a conventional measure of reliability. It related with probability of failure through the following equation

$$P_f = \Phi[-\beta] \quad (3)$$

where $\Phi[\ldots]$ is the cumulative distribution function of the standardized Normal distribution. The relation between β and P_f given in Table 3.

The reliability index β was introduced as the complete solution of the problem with two normally distributed basic variables, which is having as well the simple geometrical interpretation. Nowadays it is still widely used in different reliability problems as the numerical values of β are more convenient to operate with than very small numbers of failure probabilities.

Table 3. Relation between β and P_f.

P_f	β
10^{-1}	1.28
10^{-2}	2.32
10^{-3}	3.09
10^{-4}	3.72
10^{-5}	4.27
10^{-6}	4.75
10^{-7}	5.20

For estimating reliability level of structural elements, which is provided by the system of partial factors and combination factors, the following procedure is applied. It is based on the First Order Reliability Method (FORM) as well as the method of quickest descent (which are both used for analysis of probabilistic state functions of structures and for estimation of the values of reliability indices). The Ferry Borges – Castanheta model (Ferry Borges and Castanheta, 1971) and Turkstra's rule (Turkstra and Madsen, 1980) are used for probabilistic modelling of actions and combinations of actions. This approach provide for transformation random processes of loading into appropriate random variables, for which probabilistic models should be determined.

The value of target reliability index for structures is accepted as $\beta = 4.7$ for the reference period $T = 1$ year in accordance with TCP-EN 1990. Normal distribution is adopted for modelling permanent loads, Gumbel distribution – for modelling variable loads, Normal distribution – for load effect uncertainties, LogNormal distribution – for modelling resistance of structural elements.

The probabilistic state function $g(\mathbf{X})$ which characterizes safety margin of a structural element (*Ultimate Limit State*) includes basic variables describing loads as well as resistance:

$$g(\mathbf{X}) = z \cdot R - \Theta \cdot [(1-\eta) \cdot G + \eta((1-ks)Q + ks \cdot S)] \quad (4)$$

where: $\mathbf{X} = \{R, \Theta, G, Q, S\}$ is a vector of basic variables; z is a cumulative design parameter, e.g. cross-sectional area, reinforcement area; k_s is factor between 0 and 1, giving the relative importance of snow load among two variable loads (*live load – snow load*); $\eta = (Q_k + S_k)/(G_k + Q_k + S_k)$ is factor between 0 and 1, giving the relative importance of permanent load among other loads (*permanent load – variable loads*).

In the general case the process of making probabilistic model comprises two steps: the selection of the appropriate distribution law for the considered random variable or random process, and the setting of the parameters of this distribution.

The probabilistic models of basic variables **X** included in state function (4) are described in Table 4. They characterize resistance of structural elements R, permanent loads G, variable live Q and snow S loads, as well as basic variable Θ, which makes it possible to take into account uncertainty in load effect model.

While developing the probabilistic models the contradictions of standards Eurocodes, SNiP, and DBN in loads classification as well as in mathematical treatment of a characteristic value are taken into consideration.

The proposed probabilistic models for variable loads correspond to the return period $T = 1$ year.

The probabilistic models of *live load* (see Table 4) are developed basing on the investigation of statistical parameters of loads on structures in residential buildings presented in JCSS Probabilistic Model Code.

The probabilistic models of *snow load* are based on the own results of the current statistical investigation of long-term data collected from 48 weather stations which are spread proportionally on the territory of Belarus. Moreover, the zoning of the territory by characteristic values of snow load according to the Belarusian National Annex to EN 1991-1-3 and SNiP 2.01.07 also taken into account. While considering the Design Code DBN B.1.2-2-2006 we accepted that the same approach as in Eurocodes is applied for defining a characteristic value of snow load. Therefore the probabilistic models are described here identical to those corresponding to Eurocodes.

The probabilistic model of the *resistance of structural elements R* is developed for flexural reinforced concrete members basing on the experimental and theoretical investigation (Markouski, 2009).

The following assumptions were adopted:

- resistance of the element is calculated according to Belarussian National Annex to Eurocode 2. This means that all the coefficients related to the resistance as well as partial factors for concrete and steel strength are taken from NA TCP- EN 1992-1-1:2004;
- loads and actions on the element are set in accordance with the concerned standard (Eurocodes, SNiP, or DBN) with appropriate partial factors and combination rules;
- the element is supposed to be part of a structure or a building located in Belarus. This conditio is relevant for assessment of snow loading only; it is caused by the fact that we have comprehensive statistical data on snow loads available only for the territory of Belarus.

Table 4. Proposed probabilistic models of basic variables.

Basic variable	Characteristic value	Distrib.	μ	σ	V
Permanent load (G)	G_k	Normal	G_k	$0.1G_k$	0.1
Live load (Q) (for residential building)		Gumbel			
Eurocodes ($Q_k = 1.5\text{kN/m}^2$)	Q_k		$0.2Q_k$	$0.19Q_k$	0.95
SNiP 2.01.07-1985 ($Q_k^{(full)} = 1.5\text{kN/m}^2$) ($Q_k^{(reduced)} = 0.3\text{kN/m}^2$)	$Q_k^{(full)} = Q_k$ $Q_k^{(reduced)} = 0.2Q_k$		$0.2Q_k$	$0.19Q_k$	0.95
DBN B.1.2-2-2006 ($Q_k^{(full)} = 1.5\text{kN/m}^2$) ($Q_k^{(reduced)} = 0.35\text{kN/m}^2$)	$Q_k^{(full)} = Q_k$ $Q_k^{(reduced)} = 0.23Q_k$		$0.2Q_k$	$0.19Q_k$	0.95
Snow load (S)	S_k	Gumbel			
Eurocodes			$0.38S_k$	$0.21S_k$	0.55
SNiP 2.01.07-85			$0.58S_k$	$0.32S_k$	0.55
DBN B.1.2-2-2006			$0.38S_k$	$0.21S_k$	0.55
Resistance (R)	R_d (design value)	LogNormal	$1.4R_d$	$0.15R_d$	0.11
Model uncertainty (Θ) for load effect	Θ_k	Normal	Θ_k	$0.05\Theta_k$	0.05

4. Reliability levels comparison

Figure 1 shows the reliability index β as a function of load parameters η and k_s, which define the ratio of permanent, variable live and snow loads.

The reliability index $\beta_t = 4.7$ is stated as a target value in TCP-EN 1990-2011 for RC2 reliability class of structures and for the reference period $T = 1$ year.

The compiled reliability diagrams make it possible to conclude that provided the proposed probabilistic models of basic variables (Table 4) are valid the system of partial safety factors and combination factors stated in Eurocodes gives the required level of reliability of designed structures in most of the design situations. However, in some cases reliability of structures in persistent design situations does not meet the requirements of RC2 reliability class; and the actual average reliability level corresponds to the minimum recommended level. At the same time the rules for assessing loads on structures in accordance with SNiP 2.01.07-85 ***do not meet*** modern reliability and safety of structures requirements. It means that the probability of failure for the latter can 10-100 times exceed the maximum permissible values (!).

In respect of the Design Code DBN B1.2-2:2006 it is evident that there will be no significant increase in reliability of structures if the characteristic values for snow and wind loads are defined basing on 50-years return periods but using an old approach (those stated in SNiP 2.01.07) to deriving design combinations of loads.

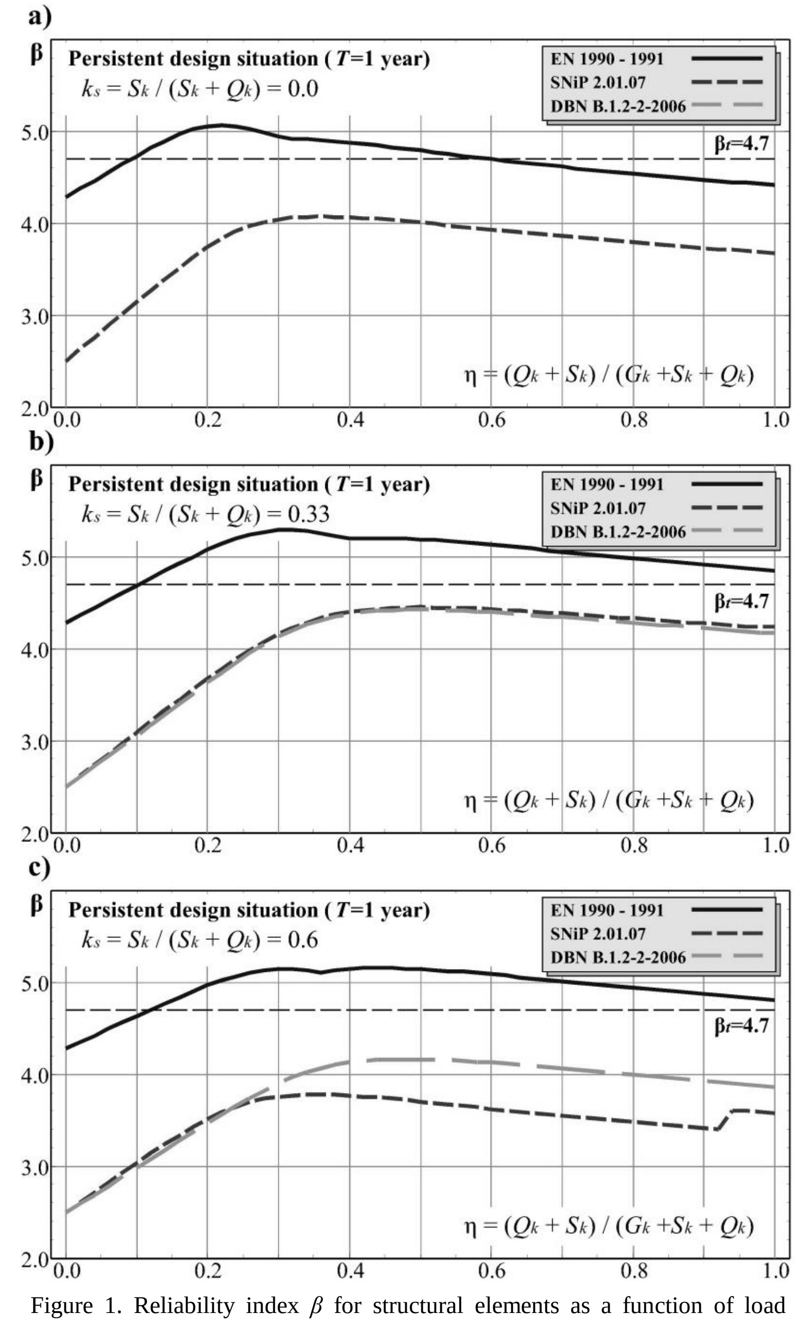

Figure 1. Reliability index β for structural elements as a function of load parameter η with: a) $k_s = 0.0$; b) $k_s = 0.33$; c) $k_s = 0.6$.

5. Reliability-based calibration of partial factor for precast RC-elements

In this section we describe the results of calibration of partial factor for self-weight. Within this analysis we consider a precast reinforced concrete structural element. Such elements characterized as heavy elements for which the self-weight could be of considerable proportion among other loads.

According to TCP-EN 1990 the design combinations of actions on a structural element *in persistent or transient design situations* may be expressed in general format as:

$$L_d = \max \begin{cases} \sum_j \left(\gamma_{G,j} \cdot G_{k,j}\right) + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} + \sum_{i>1} \left(\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right) \\ \sum_j \left(\xi \cdot \gamma_{G,j} \cdot G_{k,j}\right) + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \left(\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right) \end{cases} \quad (5)$$

where, the less favorable of the two expressions is to be chosen.

In case of only one permanent and one variable load acting, e.g. self-weight plus live load, the design combinations should be:

$$L_d = \max \begin{cases} \gamma_G \cdot G_k + \gamma_Q \cdot \psi_{0,Q} \cdot Q_k \\ \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot Q_k \end{cases} \quad (6)$$

In general case the following values of partial factors and combination factors are recommended in NA to TCP-EN 1990, as given in Table 5.

The probabilistic state function $g(\mathbf{X})$ of the structural element (*Ultimate Limit State*) can be expressed as:

$$g(\mathbf{X}) = z \cdot R - \Theta E \cdot \left[\chi \cdot G + (1-\chi) Q\right] \quad (7)$$

where: z is a cumulative design parameter, e.g. cross-sectional area, reinforcement area; $\chi = G_k / (G_k + Q_k)$ is a factor between 0 and 1, giving the relative importance of permanent load among other loads (*permanent load – variable loads*).

The probabilistic models of basic variables are given in Table 6. The models for the resistance R and live load Q are the same as described in the previous sections.

It is known that precast concrete plants should have conformity assessment for product geometry and strength of materials organized. It means that products with geometrical parameters being out of tolerances should be rejected. That is why self-weight of precast elements cannot exceed considerably its nominal values. Thus the difference between cast-in-situ and precast elements in terms of reliability theory may be expressed in changing probabilistic model for self-weight. In our case we assume that the coefficient of variation of self-weight for precast elements should not exceed 0.05. The model for permanent load G in Table 4 takes into account this assumption.

It is possible to estimate reliability level of precast structural elements by applying the approaches and methods as stated in the previous sections.

Figure 2 shows the reliability index β as a function of load parameter χ.

The reliability index $\beta_t = 3.8$ is stated as a target value in TCP-EN 1990 for the RC2 reliability class of structures and for the reference period $T = 50$ years.

One can see from the Figure 2 that there is certain excessive reliability in the area where contribution of permanent loads is significant ($\chi \leq 0.6$). It means that we may reduce the value of γ_G in such an extent that the reliability level for the considered area will not be lower than the required target level β_t.

The new reduced value of $\gamma_G = 1.15$ was determined for those elements corresponding to the area on the plot with significant self-weight loads ($\chi \leq 0.6$). The new reliability diagram is shown on Figure 3.

Table 5. The values for γ , ψ_0 , and ξ according to EN 1990.

Load type	Partial factor	Combination factor
Permanent – self weight G	$\gamma_G = 1.35$	$\xi = 0.85$
Variable – live load Q	$\gamma_Q = 1.5$	$\psi_{0,Q} = 0.7$

Table 6. Proposed probabilistic models of basic variables for precast elements

Basic variable	Characteristic value	Distrib.	μ	σ	V
Permanent load (G)					
- for any element	G_k	Normal	G_k	$0.10G_k$	0.05
- for precast element		Normal	G_k	$0.05G_k$	0.05
Live load (Q) (for residential building, reference period T = 50 yrs)	Q_k	Gumbel	$0.6Q_k$	$0.20Q_k$	0.33
Resistance (R)	R_d (design value)	LogNormal	$1.4R_d$	$0.15R_d$	0.11
Model uncertainty (Θ) for load effect	Θ_k	Normal	Θ_k	$0.05\Theta_k$	0.05

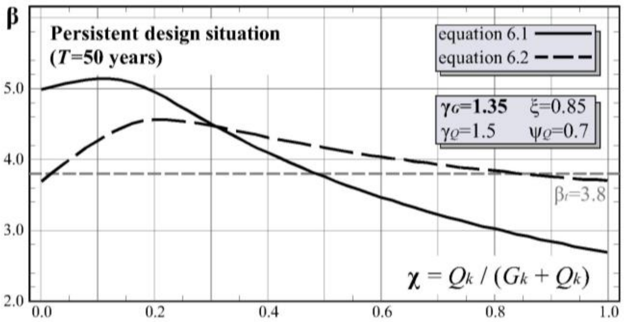

Figure 2. Reliability index β for structural elements as a function of load parameter χ for the reference period $T = 50$ years and $\gamma_G = 1.35$

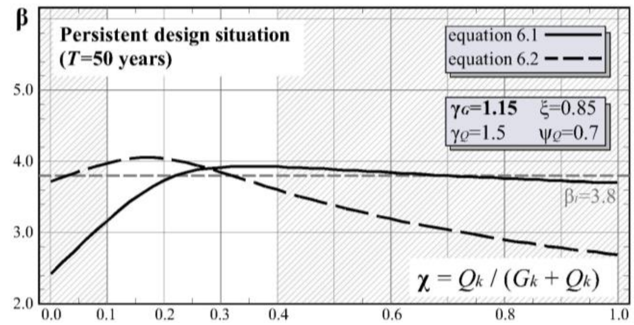

Figure 3. Reliability index β for structural elements as a function of load parameter χ for the reference period $T = 50$ years and $\gamma_G = 1.15$.

The Belarusian National Annex to TCP-EN 1990 allows using the reduced value of partial factor $\gamma_G = 1.15$ if the following conditions are provided:

- the certified quality control is organized at the plant;
- the coefficient of variation of self-weight of the structural element is not higher than 0.05;
- the ratio of the variable loads to the total load on the element including self-weight should be in the range:

$$0.1 \le \frac{\sum_{i \ge 1} Q_{k,i}}{\sum_{j \ge 1} G_{k,j} + \sum_{i \ge 1} Q_{k,i}} \le 0.4 \qquad (8)$$

It can be seen that assuming the mentioned conditions the value of the partial factor γ_G for self-weight loads can be reduced significantly. These results are expected to provide a great economical effect for precast concrete industry.

6. Conclusion

Probabilistic methods of reliability analysis of structural elements were used to compare these standards by a criterion of reliability index that is provided by the appropriate design rules for loads assessment. Probabilistic models of loads have been developed subject to the nature of these loads and to their expected duration.

Additionally the results of reliability-based calibration of partial are presented. The calibration resulted in the reducing the value of the partial factor for self-weight in case of the precast elements design from $\gamma_G = 1.35$ to $\gamma_G = 1.15$.

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