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RELIABILITY OF REINFORCED CONCRETE STRUCTURES DESIGNED ACCORDING TO THE DESIGN CODES OF BELARUS AND THE UKRAINE

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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 and Ukraine. It is noted that such structures have different reliability levels as well as failure probabilities. Approaches to assessment loads and actions on structures which are stated in European, Belarusian, former USSR, and Ukrainian standards were analyzed. It is shown that in most cases former USSR and Ukrainian standards do not meet the modern requirements for safety of structures. Additionally the results of reliability-based calibration of partial factors are presented. The calibration resulted in the reduced value of partial factors for permanent loads on precast elements.

The process of reforming the Belarusian system of standards is not completed yet. It is confirmed by the fact that three generations of normative documents have legal validity. These documents have followers as well as opponents, at the same time they lead to the results that are sometimes differ significantly.

In this connection numerical criteria for unbiased comparing different design codes are needed as well as the tool and methods to estimate the criteria.

According to the concept of reliability of structures stated in the international standard ISO 2394 [1] 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 setting the characteristic values of all the variables together with the system of partial factors (also called safety factors) and load combination factors create the safety margin for structures. Ideally, it should correspond to the target reliability levels stated in structural codes. And of course, the expected reliability level of structures should be checked before making any corrections in design standards. But in reality it doesn't always happen.

There are lot of works devoted to reliability level assessment for different countries (Holický, Sýkora, Retief, Sørensen, Faber and many others [2 – 6]). The mentioned works were 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 Belarusian and Ukrainian conditions. It will show the level of reliability of new and existing structures.

It should be noted that the standardized approaches for assessing loads and 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 standards, as well by modern Belarusian and Ukrainian standards. The main challenge of such study consists in creating the base for comparing different standards. As it will be shown later the considered standards comprise completely different rules for deriving design combinations of loads on structures. Actual statistical data on actions (e.g. snow or wind loads) should be taken into account at that.

The aim of the present paper is to estimate the level of design reliability of structures (provided by using a system of safety 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 and Ukraine for the last decade. The following problems should be solved 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;
- 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.

Known methods of reliability theory will be used; among them are the methods based on the 1-st order approximation of the probabilistic state functions as well as approaches of extreme values theory (for assessment of stochastic processes of loading). The detailed explanation of the methods and models used will be given below.

The comparative analysis of the standards 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 EN 1990 – EN 1991 [7, 8] (*hereinafter referred to as Eurocodes*);
- Belarusian design code SNB 5.03.01-2000 «Concrete and reinforced concrete structures design» [9];
- the USSR design standard SNiP 2.01.07-85 «Loads and actions» [10].

Comparing the code SNB 5.03.01 [9] with the system of Eurocodes and ISO 2394 [1], it should be noted that these standards are 100 % harmonized in respect of loads assessment in design. However, the majority of provisions stated in SNiP 2.01.07-85 [10] contradict the ISO 2394 and EN 1990 [7] that are also valid in Belarus. For example, there are inconsistencies in the classification of loads and actions, in values of partial factors for loads, in combination rules for loads for *Ultimate* as well as *Serviceability Limit State* design of structures.

The Ukrainian standard DBN B.1.2-2-2006 «Loads and actions» [11] is also considered in the article. This document mainly repeats the concept of SNiP, but also contains some approaches similar to those used in EN 1991.

The Eurocodes and SNiP 2.01.07-85 [10] 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 Ukrainian standard DBN B.1.2-2-2006 «Loads and actions» [11] mainly repeats the concept of SNiP, but for determining characteristic values of snow and wind loads an approach similar to the one used in Eurocodes [7, 8] is applied. This approach is based on 50-years return periods for the extreme values of loads.

The rules for deriving design combinations of loads on structures in persistent design situations are presented in Table 1. The case when permanent, live, and snow loads are imposed, 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 are indicated in Table 2.

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**
EN 1990:2002 [7] SNB 5.03.01-2002 [9]	$\max\begin{cases}\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{0,Q} \cdot Q_k + \gamma_S \cdot \psi_{0,S} \cdot S_k \\ \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot Q_k + \gamma_S \cdot \psi_{0,S} \cdot S_k \\ \xi \cdot \gamma_G \cdot G_k + \psi_{0,Q} \cdot \gamma_Q \cdot Q_k + \gamma_S \cdot S_k\end{cases}$
SNiP 2.01.07-85 [10]	$\max\begin{cases}\left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,\text{reduced}} \cdot Q_k^{(\text{reduced})} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n \\ \left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,\text{full}} \cdot Q_k^{(\text{full})} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n\end{cases}$
DBN B.1.2-2-2006 [11]	$\max\begin{cases}\left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,\text{reduced}} \cdot Q_k^{(\text{reduced})} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n \\ \left(\gamma_G \cdot G_k + \gamma_Q \cdot \psi_{Q,\text{full}} \cdot Q_k^{(\text{full})} + \gamma_S \cdot \psi_S \cdot S_k\right) \cdot \gamma_n\end{cases}$

Note: detailed symbol definitions may be found in Table 2

One can see from Table 2 that there is a significant difference between the approaches to setting characteristic values of loads. The safety factor *for permanent loads* γ_G in Eurocodes has a greater value, but it should be used 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 [8] the characteristic value is the value which on average is exceeded once in 50 year. An analogous approach is accepted in Ukrainian standard DBN [11]. Meanwhile, within the bounds of SNiP [10], the characteristic value of a snow load is the mean value of 1-year maximums.

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

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

	Eurocodes, SNB 5.03.01	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	–	$\gamma_n = 0.95$	$\gamma_n = 0.95$

Notes:
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

According to SNiP [10] and DBN [11], in contrast to Eurocodes, *variable live loads* are divided into full and reduced values. The ratio of full and reduced values in Table 2 is estimated using the characteristic values of live loads on floor slabs in residential buildings (given in SNiP and DBN).

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

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

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

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

where Φ[…] is the cumulative distribution function of the standardized Normal distribution. The relation between β and P_f is given in Table 3.

Table 3 – Relation between β and P_f

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

Originally 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.

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 1-st 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 [12] and Turkstra's rule [13] are used for probabilistic modeling 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 EN 1990 [7]. Normal distribution is adopted for modeling permanent loads, Gumbel distribution – for modeling variable loads, Normal distribution – for load effect uncertainties, LogNormal distribution – for modeling resistance of structural elements.

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

$$g(\boldsymbol{X}) = z{\cdot}R - \Theta{\cdot}[(1-\eta)\,G + \eta((1-k_s)Q + k_s S)], \tag{4}$$

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

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)					
Eurocodes ($Q_k = 1.5$kN/m^2)	Q_k	Gumbel	$0.2Q_k$	$0.19Q_k$	0.95
SNiP 2.01.07-1985 ($Q_k^{(full)} = 1.5$kN/m^2) ($Q_k^{(reduced)} = 0.3$kN/m^2)	$Q_k^{(full)} = Q_k$ $Q_k^{(reduced)} = 0.2Q_k$	Gumbel	$0.2Q_k$	$0.19Q_k$	0.95
DBN B.1.2-2-2006 ($Q_k^{(full)} = 1.5$kN/m^2) ($Q_k^{(reduced)} = 0.35$kN/m^2)	$Q_k^{(full)} = Q_k$ $Q_k^{(reduced)} = 0.23Q_k$	Gumbel	$0.2Q_k$	$0.19Q_k$	0.95
Snow load (S)					
Eurocodes	S_k	Gumbel	$0.38S_k$	$0.21S_k$	0.55
SNiP 2.01.07-85	S_k	Gumbel	$0.58S_k$	$0.32S_k$	0.55
DBN B.1.2-2-2006	S_k	Gumbel	$0.38S_k$	$0.21S_k$	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

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 [14]. It should be noted that JCSS guidelines correspond to the results of investigation published in USSR by Raizer, Bulychew et al. in 1980s [15–17].

The probabilistic models of *snow load* are based on the own results of the current statistical investigation of long-term data collected from 18 weather stations which are spread proportionally on the territory of Belarus [18]. Moreover the zoning of the territory by characteristic values of snow load according to the Belarusian National Annex to EN 1991-1-3 [8] and SNiP 2.01.07-85 [10] is also taken into account. While considering the Ukrainian standard DBN B.1.2-2-2006 [11] 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 [18].

Generally speaking we consider a reinforced concrete structural element designed with the following assumptions:

- resistance of the element is calculated according 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 EN 1992 [19];
- 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 condition 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.

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 EN 1990 [7] 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 [10] *do not meet* modern reliability and safety of structures requirements.

One can see on diagrams from Figure 1 that using Eurocode results in structures with the value of reliability index at average greater by 1 than for structures designed in accordance with SNiP 2.01.07 [10]. It means that the probability of failure for the latter can 10-100 times exceed the maximum permissible values.

In respect of the Ukrainian standard DBN [11] 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 [10]) to deriving design combinations of loads.

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

According to EN 1990 [7] 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:

$$L_d = \max \begin{cases} \gamma_G \cdot G_k + \gamma_Q \cdot \psi_{0,Q} \cdot Q_k & (6.1) \\ \xi \cdot \gamma_G \cdot G_k + \gamma_Q \cdot Q_k & (6.2) \end{cases}$$

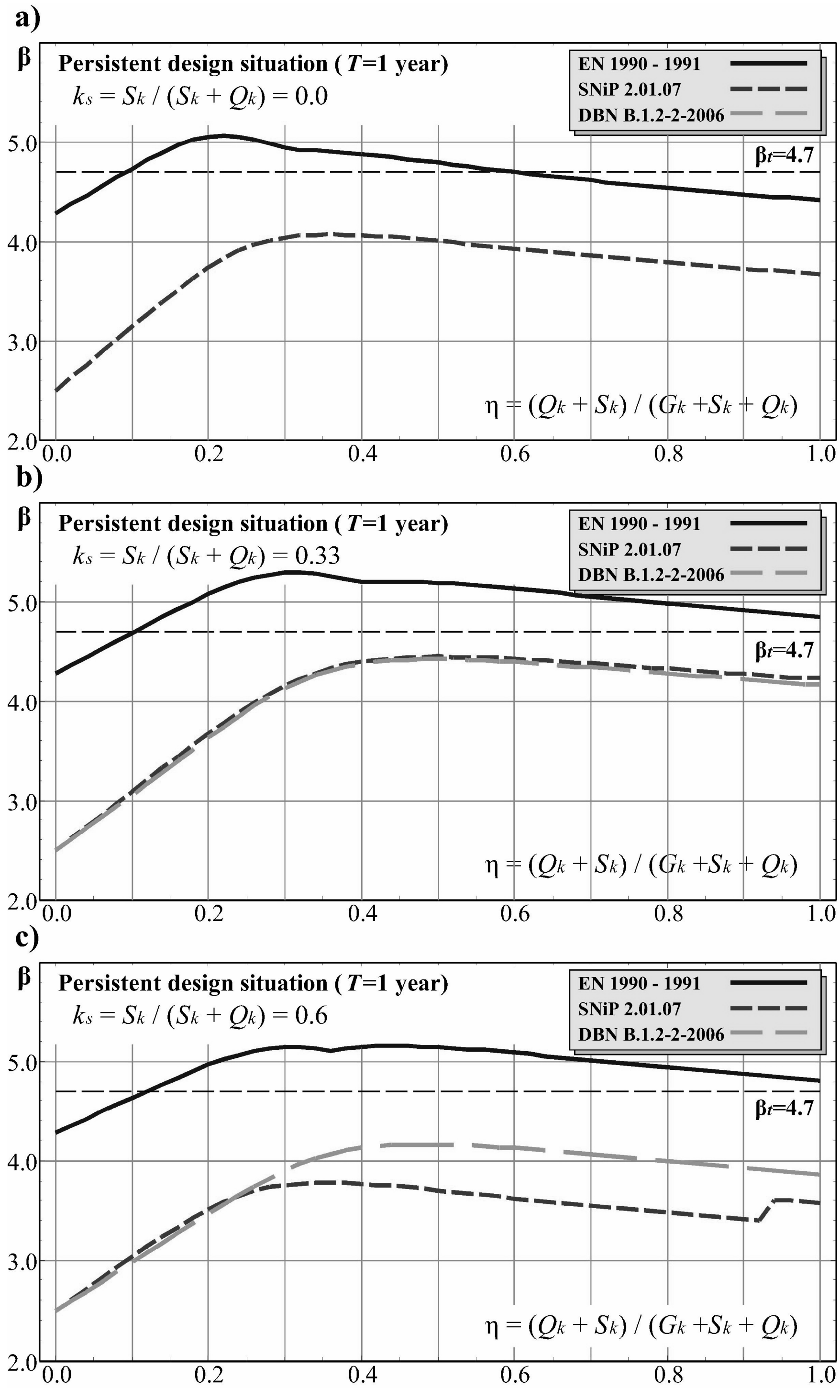

Fig. 1. Reliability index β for structural elements as a function of load parameter η with:
a) k_s = 0.0; **b)** k_s = 0.33; **c)** k_s = 0.6

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

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*	$\gamma_Q = 1.5$	$\psi_{0,Q} = 0.7$

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

$$g(\boldsymbol{X}) = z \cdot R - \Theta_E\,[\chi\, G + (1-\chi)Q], \quad (7)$$

where z = is a cumulative design parameter, e.g. cross-sectional area, reinforcement area; $\chi = G_k / (G_k + Q_k)$ = 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.

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

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 loading. 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 6 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 EN 1990 [7] for the RC2 reliability class of structures and for the reference period T = 50 years.

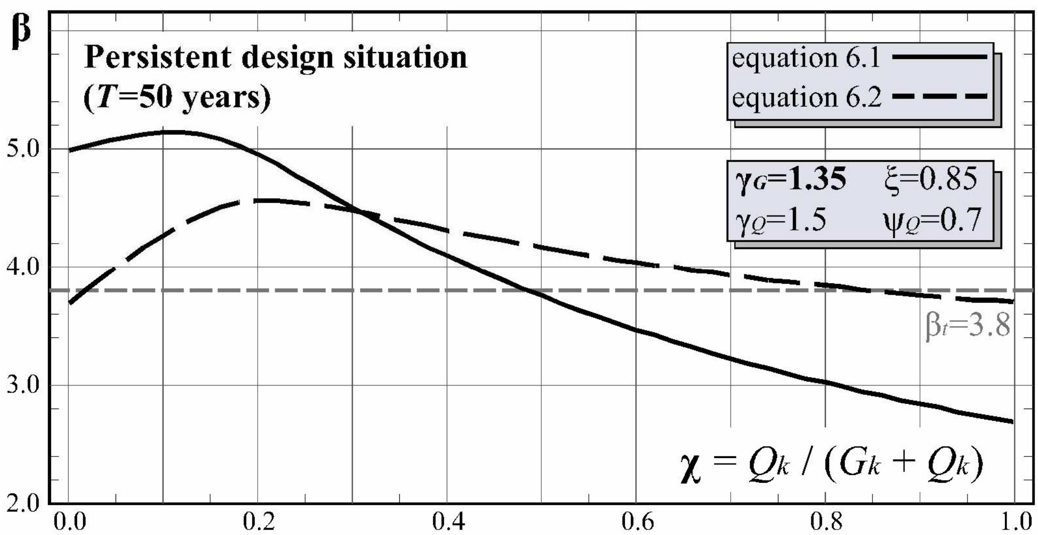

Fig. 2. Reliability index β for structural elements as a function of load parameter χ for the reference period T = 50 years and $\gamma_G = 1.35$

One can see from the Figure 2 that there is certain excessive reliability in the area where contribution of permanent loads is significant ($\chi \le 0.6$). 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 ($\chi \le 0.6$). The new reliability diagram is shown on Figure 3.

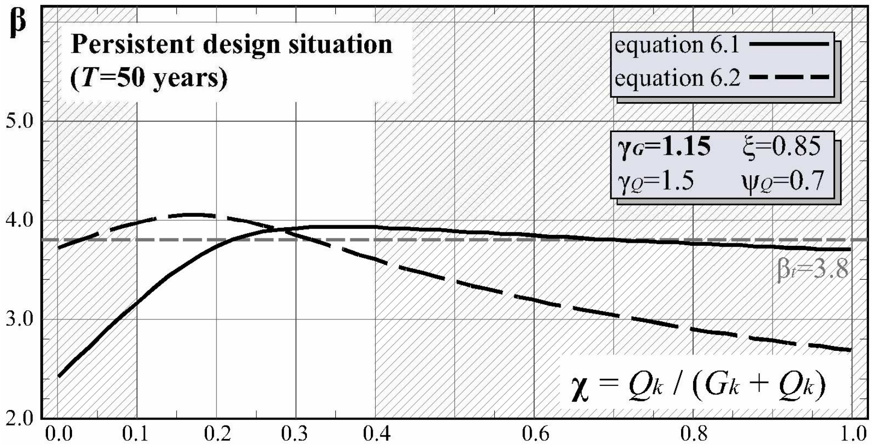

Fig. 3. Reliability index β for structural elements as a function
of load parameter χ for the reference period T = 50 years and $\gamma_G = 1.15$

The Belarusian National Annex to EN 1990 [7] allows using the reduced value of partial factor $\gamma_G = 1.15$ if the following conditions are provided:

- the system of 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 full load on the element including self weight should be in the range:

$$0.1 \le \frac{\sum_{i\ge1} Q_{k,i}}{\sum_{j\ge1} G_{k,j} + \sum_{i\ge1} Q_{k,i}} \le 0.4\ . \tag{8}$$

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.

The existing combination rules for loads for *Ultimate Limit State* design of structures have been described according to the three design codes that have been valid in the Republic of Belarus as well as in Ukraine for the last decade.

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.

It has been shown that the levels of reliability of structures designed according to former USSR and Ukrainian standards are significantly lower than the required level, and that the probability of failure for such structures can exceed maximum permissible values up to 100 times.

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 loads on precast elements from $\gamma_G = 1.35$ to $\gamma_G = 1.15$.

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SELF-COMPACTING CONCRETE – A MATERIAL OF A NEW GENERATION

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We consider the technological, technical and economic advantages of self-compacting concrete (SCC) compared with traditional concrete of vibrational compacting. We analyze the state of normative base for the application of self-compacting concrete in construction practice, as well as composition of SCC. We investigate the possibility for reducing the cost of self-compacting concrete.

One of the dominant trends in concrete technology during the last ten years has been growing interest in the self-compacting concrete.

In the literature we can find many definitions of the self-compacting concrete, but they characterize it in the same way. It is concrete, that is able without impact on it additional external energy to flow under its own weight, retaining its homogeneity, and also ensuring a complete compaction, filling formwork and encapsulation of rebar and embedded parts.

The advantages of self-compacting concrete in comparison with other traditional types of concrete are as follows:

– creation of building structures, having high strength and no defects caused by errors when compacting the concrete mix;

– the ability to create any geometry of concrete structure;

– the use of a simpler and less massive construction formwork (due to the lack of the concrete vibration process, on the formwork is not affected by additional static and dynamic load);

– the possibility of placing per shift larger volume of concrete;

– no necessity of concrete compacting and hence eliminating errors, which might arise during its compacting;

– work of the personnel in a safe conditions during concreting;

– the absence of noise and vibration, which have a negative impact on both the staff and the residents living near the construction site;

– shortening the duration of construction.