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HUNGARIAN EXPERIENCE IN STRUCTURAL DESIGN CODING (HISTORICAL ANTECEDENTS OF EUROCODE-2)

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Abstract

This paper gives review of the historical antecedents of Eurocode-2 in Hungary and East Europe. The method of permissible stresses, using uniform safety factor was first changed in 1950 in Hungary by the semi-probabilistic method using partial safety factors. This new method was accepted with some resistance on the part of the leading structural engineers. Nevertheless most of the East-European countries accepted the new method with some 'political overtones', to be follow the Soviet example.

The authors assert in the papaer that due to the economic necessities, Hungary and the other East European countries gained experience with the regulations affording less safety than the EC2, and this offers an interesting set of experience to the West European countries which have intoduced or are introducing the semi-probabilistic procedure. The most significant point all the experience is the recognition that only one part of the parameters in the structural analysis determining safety can be handled statistically.During design the statistically not significant data such as the error of the structural model must also be taken into consideration. Based on the experience, the authors propose an alternative design method.

Keywords: safety factor, design principles, allowed stress, semi-probabilistic design, alternative design.

Introduction

Following Wordl War II, from the beginning of the '50s, the political and economic separation of the East-European countries, led to separation in the fields of building research and coding as well. Thanks to the leading technical personalities of these countries, the East-West relations in the area of research and structural regulations did not end even in the 'most difficult' times. Achievements in the area of research and structural regulations, within the framework of CEB-FIP reached Hungary and the other East-European countries, where they creatively influenced the scientific research and structural regulations of those countries. Information in the opposite direction, however was considerably worse. As it is known, in the economic system of directed plans, research was financed by the national budget and in accordance with this, or even above and beyond it, the authorities prescribed and politically expected the inclusion of the result of structural research into the national structural standards which were considered to be legally binding.

In the national structural standards of Hungary and of the other East European countries, the rules based on CEB-FIP research results appeared decades earlier than they did in West European countries. Beside of this, the above regulations, in view of the often exaggerated material saving expectations represented higher risk [1], [2], in comparison to the later suggestions of CEB-FIP. Naturally the designers often compensated for this fact. Referring to the daring level of risk-taking, the researchers at the same time had an opportunity to examine and at least in part to answer the resultant problems which arose.

Thus the history of the Hungarian and the other East European structural coding and construction practice is a valuable collection of experience for other researchers if only for the reason that the history of East European structural coding is a history of changing the deterministic and probabilistic principles (*Fig. 1*) [3].

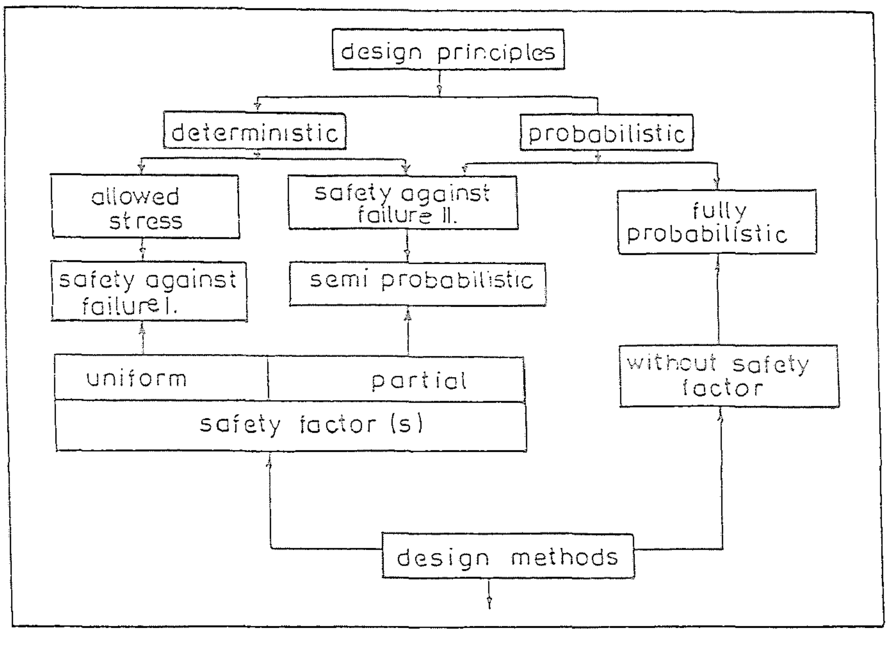

Fig. 1. History of design principles and methods

The authors of this articles think that certain aspects of the activities in structural coding of Hungary and of the East European countries are of interest for research history and as such it is worth offering them to the public.

2. The History of Structural Design Procedures in East-Europe

2.1. The Uniform Safety Factor in Design Methods

The method of permissible stresses based on the condition

$$\sigma_{\max}(Q_m, L_m) \leq R_{adm} \approx \frac{R_m}{\gamma_l} \tag{1}$$

was replaced in the Soviet Union in 1938 by the 'safety against failure I.' method, which also used the uniform safety factor. The change can be regarded mainly as a criticism against the design method of concrete structures using the $n = \frac{E_s}{E_c}$ ratio. [2], [3]. The basic unequality of the method was:

$$S(Q_m, L_m) \leq \frac{R(R_m, L_m)}{\gamma_2}, \tag{2}$$

where

S — is the external (or internal) design load

R — the expected value of the resistance determined according to the theory of failure or to the theory of plasticity

γ_2 — the ratio of the resistance and the design service load depending on the character of the failure and from the ratio of the permanent and variable loads. (*Table 1*) [4], [5].

In the calculation according to (2), the compression strength R_c of the concrete, was determined from the average 200 mm cubic strength R_m of the concrete, ($R_c = 0.7R_m$) and the strength of the reinforcing bars were in turn theoretically calculated from the expected value of the yield strength [4], [5].

The permissible load calculated from the assumed failure state was accepted by the structural engineers because it was analogous to the earlier method used and it created a good opportunity to judge the safety against failure.

In the rest of the East European countries and thus in Hungary as well, this procedure in such a form was not applied.

Table 1
Values of uniform safety factor

Load combination	Ratio of $\frac{\text{permanent}}{\text{variable}}$ loads	Cause of failure concrete in compression and steel in tension columns, arches, supports		other structural elements	concrete in tension (principal stress
basic	≤ 2.0	2.0	(1.85)	1.8	2.2
	> 2.0	2.2	(2.0)	2.0	2.4
basiḍ and	< 2.0	1.8	1.6	2.0	
additional	> 2.0	2.0	1.8	2.2	
with accidental actions		1.6	1.5	1.8	

Note: The values in parameters refer to the structures with high ($\rho \geq 0.05$) reinforcement ratio

2.2. The Partial Safety Factors in Design Methods

'The safety against failure II.' method based on the partial safety factors was later called in the Soviet Union perhaps not quite accurately the limit state method [4], [5].

The limit state method which was essentially the early or first version of the semi-probabilistic procedure, was first introduced in Hungary as a national regulation in December 1950 and it can be regarded as one of the important antecedents of Eurocode-2.

The 1950 Hungarian code for reinforced concrete structures meant a qualitative change, and it was met nearly generally with reservations in spite of the fact, that structural research all over the world attested to its validity. In professional circles, MAX MAYER's book [6] was well-known as were the DENISH A. J. MOE's work [7] of the divided safety factors system and G. KAZINCZY's [8] and others' experiments on the plastic reserves of structures. In spite of this, in November 1949 — with high-level content — the 'National Building Design Regulations II. Reinforced Concrete Regulations' based on the system of allowed stresses was published in Budapest, [9]. Following this, within one year, the supplement was prepared with a surprising quickness to the leading structural engineers, entitled, 'Instructions for the Design of Reinforced Concrete'. The Instructions used non-linear calculations based on the theory of failure of reinforced concrete structures and used partial safety factors [9], [17]. In this, the structural requirements were — for the ultimate limit state

$$S_d\left(\sum \gamma_{Qi} \cdot Q_{im}, L_m\right) \leq R_d\left(\frac{R_{im}}{\gamma_{Ri}}, L_m, H_a\right) \tag{3a}$$

— for the serviceablity limit state

$$L \leq L_{adm} , \tag{3b}$$

where

S_d and R_d — design load determined from loads and other actions or from the actions or from the action effects and design resistance determined from the resistance parameters, supposing the III. stress state (in the concrete rigid-plastic and in the steel elastic-plastic material properties).

H_a — those cases of the ultimate limit state (turning over, slipping etc.) in which the strength does not play a role in the change which impedes the useability of the structure.

In the Hungarian structural Code of 1950 the design value of strength was prescribed with reference to the Soviet experience [9], the average value of R_m of the concrete's cubic compression strength was determined by

$$R_c \approx \frac{0.7 R_m}{1+3s} k_b \tag{4a}$$

and the strength of the steel reinforcement was determined by:

$$R_s \approx \frac{R_{sym}}{1+3s} k_s . \tag{4b}$$

The $(1+3s)$ denominators were accepted based on Mayer's [6] suggestion. In relation to the k_b and k_s values it is worth mentioning that in the case of concrete specimens less than 300 mm, the $k_b = 0.8$, and in the case of welded fabric made of cold-drawn wires $k_s = 0.65$ [5].

The design procedure based on the plastic behaviour of materials and the partial safety factors was established by structural research in the '20s and '30s based on Mayer's book [6], but the world-wide introduction of a structural code on such a basis was delayed because the respectable structural engineers did not recognize the advantages of the procedure and there was no political pressure or expectation in theis regard.

2.3. The Design Standard Based on Equal Safety

Using the materials presented on the safety concepts at the 1949 IABSE Congress in Liege, in Hungary in 1951, new railway bridge regulations were introduced in which the 'Safety Against Failure II' procedure's further developments, the concept of equal safety [7] was implemented. The procedure follows the concept that the measure of safety means that the exceeded prescribed variable load is exceeded. The purpose of the structural design here is that the various parts of the bridge structure should be designed with uniform safety. To attain this, the safety of a structural element is acceptable if

$$\gamma_Q \approx \frac{R_d - \sum \gamma_{Gi} \cdot G_{im}}{\sum Q} \geq \gamma_{Qd}$$

condition is met.

This procedure was used during decades in the design practice according to the Hungarian Railway's Regulations of Bridges.

Finally this regulation was withdrawn because it became clear that safety must be defined in a wider sense.

2.4. Politics and Design Methods

2.4.1. The Political Background to the Introduction of the Hungarian Design Code

The introduction of the new design methods into codes in Hungary and generally in East Europe was furthered by the political situation. The new methods which created a new era, were iniciated by the brilliant structural engineer I. Menyhárd who used the advantages of referring to the Soviet experience and thus on the decision of the important Hungarian National Economic Council, the new code was introduced.

In analyzing the political factors, it is interesting to note that I. Menyhárd was arrested in 1947 after Churchill's speech in Fulton and was held in captivity for a few weeks, because he was, in fact, one of the creators of the Foundation with the help of which in 1943 a delegation load by Nobel prize winner A. Szentgyörgyi went on a secret mission to Istambul where they discussed a Hungarian cease-fire with the British representatives. On the other hand structural engineer E. Hilvert, had travelled on a contract work to the Soviet Union in the '30s and when he returned to Hungary, after the war he took the temporary Soviet instructions based on the safety against failure [16]. I. Menyhárd in that time was at the Budapest Scientific Institute of Building Research (ÉTI) and recognized the professional and political advantages of the situation. He managed to get this method introduced, which was professionally established, and which iniciated a professional development. It was emphasized at the 1949 IABSE Congress in Liege as well, only by referring to the Soviet example in spite of the opposition within the profession.

2.4.2. The Political Background of the Introduction of the New Limit State Method into the Soviet Code

Hungarian graduate students who were studying in Russia at the Moscow University of Construction Engineering (MISI) attended in 1952 as observers the meeting of its Scientific Senate. Previous to that meeting, Professor Levanov, Head of the MISI Chair for History of Science, wrote a letter to the Central Committee of Communist Party arguing against the introduction of the new draft Code. Based on The letter, N.S. Hruschov, who at that time was the party secreatary of Moscow and was also responsible for the construction industry, asked the Scientific Senate of MISI for advice. There was sharp arguments on the Scientific Senate between the proponents and the adversaries of the Code. The strongest argument of the opposition was the fact that the new design procedure would cause confusion among the structural engineers and that it might led to economic damages. The scientist who developed the procedure, A. A. Gvozdev, pointed to the two Hungarian graduate students present and referred to the fact that in Hungary used this procedure had been used for two years without any difficulty. As a result of the debate, the new Code was introduced in 1955 in the Soviet Union, too.

2.4.3. The CMEA Code and its Antecedents

Following the practice of Hungary and of the Soviet Union, by the end of the '50s Poland, Bulgaria and Romania had changed to the limit state method. It was also attempted in Czechoslovakia but the profession there did not accept the procedure.

The first conference about a possible CMEA Structural Design Codes (especially the Basic Principles and the Part dealing with structural concrete), was held in Moscow in 1960. One of the authors of this article, K. Szalai took part in that discussion where Professor G. Brendel of Dresden, thinking of DIN, passionately defended the method of permissible stresses against the opinion of Professor Gvozdev.

Gvozdev again referred to the Hungarian experience. Finally, after the attempts of a number of years, the detailed Design Handbook, based on the Soviet Code was prepared with the purpose that the member states would introduce them as their national building codes. Hungary, in the absence of political pressure, did not accept it, because the code built on semi-probabilistic bases contained in its details such experimentally worked out procedures which could not be theoretically supported. The rest of the East European countries introduced the Handbook as a national code, which incidentally created serious political problems within the circle of structural engineers, for example in Poland. Istead of the detailed Design Handbook, the CMEA Code was developed in 1978 with active Hungarian participation as a compromise solution based on the semi-probabilistic procedure, but only contained the basic principles. This Code met the CEB-FOP '78 Recommendations.

3. Safety-Related Research

3.1. Optimal Safety

The trial and comparison calculations based on the CEB-FIP '78 Recommendations, proved the fact that the national codes of Hungary and of some other East European countries, generally took greater risks than put forward in the Recommendations [1], [2]. The problem of the low safety level has engaged the East European researcheres in the past several decades.

Starting from the CEB '64 Recommendation [10], in Hungary several researchers were concerned with investigations on optimal (or sufficient) risk p_0 corresponding to the 'Minimum of Complex Allocations', which according to T. KÁRMÁN [11]

$$p_0 = \frac{1}{k_0} = \frac{1}{80\gamma} \tag{6a}$$

and according to E. MISTÉTH [12]

$$k_0 = \frac{1}{p_0} = \frac{2.3}{b}(\gamma + 1.5)\,. \tag{6b}$$

In the Soviet Univon, the research activities in relation to the uniform safety factor [13] led to the result

$$\gamma_0 = \frac{R_m}{S_m} = 1 - \frac{0.5772}{\alpha_0} + \ln(\alpha_0 \cdot T \cdot S)\,, \tag{7}$$

where

$$\alpha_0 = \frac{\pi}{\nu_{Rs}\sqrt{6}}\,, \qquad T = \frac{1}{E} \approx 6.667\,.$$

The Hungarian researchers [11], [12] proved that in the case of apartment houses and office buildings, the δ damage coefficient and thus the p risk would be

— for the ultimate limit state

$$\delta_0 = 125\,, \qquad p_0 = 10^{-4}\,,$$

— and for the serviceability limit state

$$\delta_{\text{ser}} = 2.5\,, \qquad p_{\text{ser}} = 5 \cdot 10^{-3}\,.$$

3.2. Design Values of Loads and Other Actions and Resistances in the Hungarian Code

Starting from the acceptable $p_0 = 10^{-4}$ risk, according to [12] the calculation, for a two-parameter case was found (*Fig. 2*) that fractiles on the load S_d side would be 99% ($p_s = 1\%$), while on the resistance R_d sides would be $1^\circ/_{\circ\circ}$ ($p_R = 0.1\%$) [2]. At the same time, if at the design stage we consider the statistically non-treatable circumstances, and the structure's planned economical life time (*Fig. 3*), then, according to [14]

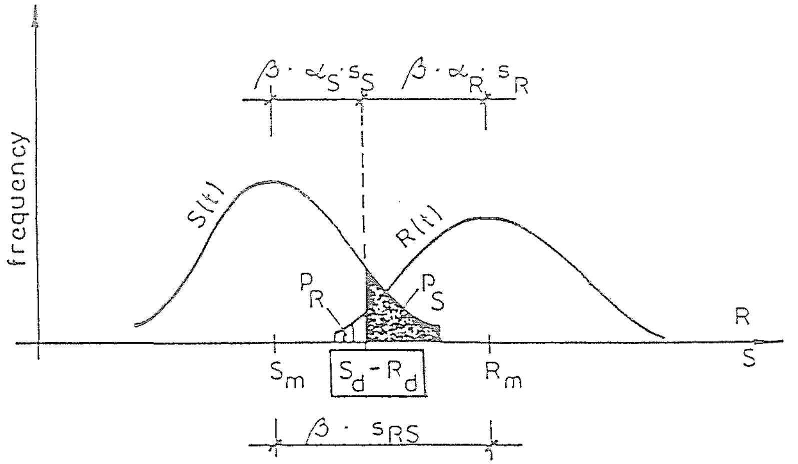

Fig. 2. Fractiles in the semiprobalistic method

– the value of the loads and action effects (of resistances) will be:

$$S_d = \left[\sum \gamma_{Gi} \cdot G_{im} + \psi_1 \cdot \gamma_{Q1} \cdot Q_l m(t) + \sum_{i=2}^{\rho} \psi_i \cdot \gamma_{Qi} \cdot Q_{im}(t)\right] \gamma_{Sn} \cdot \gamma_{Sd}\,, \tag{8}$$

– and the calculated value of the load-bearing capacity in symbolic form will be

$$R_d = \left[R\left(\frac{R_{ik}}{\gamma_{Ri}}, \frac{L_{im}}{\gamma_{Li}}\right)\right] \gamma_{Rn} \cdot \gamma_{RT} \cdot \gamma_{Rd}\,, \tag{9}$$

where

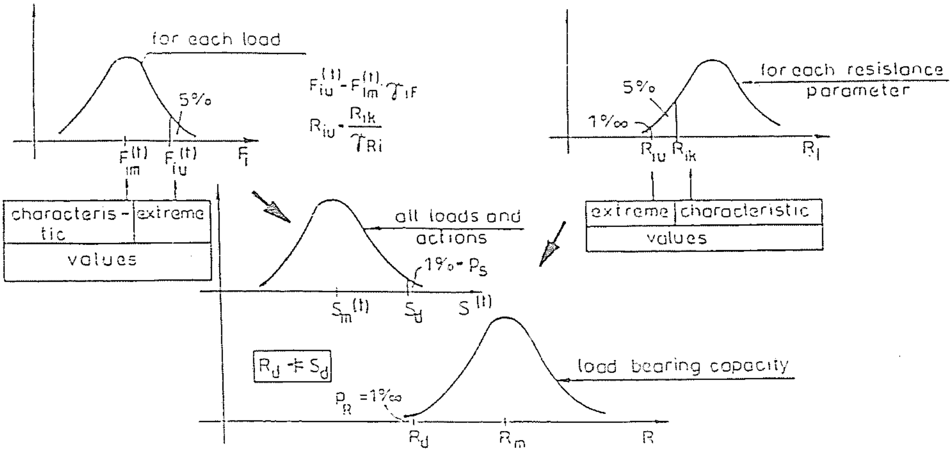

Fig. 3. The S_d and R_d values in Hungary

$Q_1m(t)$ and $Q_{im}(t)$ — are the maximal expected values of the most important and the other variable loads respectively, during the t economic life time.

Note: In the case of overwhelming self weight or overwhelming variable load, the S_d according to (8), does not represent a 99% fractile. In such cases, instead of the 99% value, the calculation must be made according to the 99% level of the given load's exceptional value.

4. Summary of the Experience Gained with the Semi-Probabilistic Design Method

In Hungary and in the most of the East-European countries, the structures, and in particular be reinforced concrete structures were designed with the semi-probabilistic method (simnilar to the Eurocode 2) since the '50s. The experience of four decades' engineering practice is of value to the other (e. g. Western European) countries.

Of these the most important pieces of experience are:

a. The profession resists the acceptance of the procedure and is only willing to do so under economic or political pressure.

b. Time has proved the load bearing capacity of the East European structures which were designed on the basis of structural codes representing lower safety level. The degree of reliability 10^{-4} for ultimate limit state and $5 \cdot 10^{-3}$ for serviceability limit state proved to be acceptable if the parameters which cannot be statictically handled do not play an important role in the structure's safety.
c. These parameters should be taken into account using modifying factors in structural design.
d. Considering the above experience, such an alternative design procedure could be considered (see point 5.) which uses a uniform safety factor method instead of partial safety factors, but pays partial attention to those statistically untreatable parameters.

5. The Alternative Design Method

It seems reasonable to propose a new alternative design method called 'permissible actions or action effects' [14] according to *Eq.* 2, modeling the method of permissible stresses and the method 'safety against failure I' and taking into account the experience gained with the semi-probabilistic procedure. From the point of view of the statistically defineable parameters, the uniform safety factor could be expressed according to (7) or with a global safety factor according to [15]:

$$\gamma_0 = \exp\left(\beta_0 \cdot \alpha_R^{(+)} \cdot \nu_R\right)$$

$$\cdot \left[\frac{1}{1+\mu}\left(1 - \beta_0 \cdot \alpha_G^{(-)} \cdot \nu_G\right) + \frac{\mu}{1+\mu}\left(1 - \beta_0 \cdot \alpha_Q^{(-)} \cdot \nu_Q\right)\right] \tag{10}$$

which could be considered more exact.

Here β_0 is the safety index for the acceptable p_0 risk, (e. g. if $p_0 = 10^{-4}$ then $\beta_0 = 3.719$).

Considering the statistically underfineable parameters by the modifying coefficients γ_R and γ_S, detailed in (8) and (9), the safety of the structure against failure is sufficient, if the condition

$$S_m \leq \frac{R(R_m, L_m)}{\gamma_m} \tag{11}$$

is fulfilled where

$$\gamma_m = \gamma_0 \frac{\gamma_{Sn} \cdot \gamma_{Sd}}{\gamma_{Rn} \cdot \gamma_{Rt} \cdot \gamma_{Rd}}. \tag{12}$$

The γ_0 value can be interpreted in relation to the damage coefficient using (6a) from the point of view both of the ultimate and serviceability limit states.

For the exhaustion of the load-bearing capacity (ultimate limit state), the damage coefficient will be $\delta = 125$ which, according to (6/a), will have an accepted risk of $p_0 = 10^{-4}$. The calculated values of γ_0 are shown in *Table 2*.

Table 2
The γ_0 global safety factor

		$\nu_R = 0.05$		$\nu_R = 0.10$		$\nu_R = 0.15$	
$\mu = \frac{Q_m}{G_m}$	ν_G	$\nu_Q = 0.10$	$\nu_Q = 0.20$	$\nu_Q = 0.10$	$\nu_Q = 0.20$	$\nu_Q = 0.10$	$\nu_Q = 0.20$
	0.05	1.2808	1.2808	1.5285	1.5286	1.9614	1.9614
0.01	0.10	1.4481	1.4481	1.6448	1.6148	1.0577	2.0577
	0.15	1.6233	1.6233	1.8059	1.8089	2.1016	2.1017
	0.05	1.2831	1.3663	1.5212	1.5818	1.0577	2.0063
0.05	0.10	1.3663	1.4324	1.5818	1.6308	2.0063	2.0474
	0.15	1.4703	1.5222	1.6644	1.7128	2.0718	2.1050
	0.05	1.3082	1.4622	1.5409	1.6571	1.0718	2.0667
1.00	0.10	1.3542	1.4026	1.5729	1.6851	1.9088	2.0861
	0.15	1.4170	1.5392	1.6201	1.7290	2.0385	2.1188
	0.05	1.3304	1.5272	1.5563	1.7174	1.9847	2.1087
1.50	0.10	1.3586	1.5444	1.5761	1.7334	2.0015	2.1233
	0.15	1.4004	1.5718	1.6066	1.7595	2.0274	2.1467
	0.05	1.3964	1.6838	1.6039	1.8736	2.0251	2.2506
5.00	0.10	1.4006	1.6921	1.6069	1.8760	2.0277	2.2531
	0.15	1.4074	1.6958	1.6121	1.8796	2.0319	2.2565
	0.05	1.4480	1.8039	1.6448	1.9864	2.0577	2.3546
100.00	0.10	1.4481	1.8039	1.6448	1.9864	2.0577	2.3546
	0.15	1.4481	1.8039	1.6448	1.9864	2.0577	2.3546

The simoultanious expected load value S_m in (11) will be

$$S_m = \sum_{i=1}^{n} G_{im} + \sum_{j=1}^{k} \Psi_j \cdot Q_{fm}(t) . \tag{13}$$

To verify the requirements of the serviceability limit states, in the expression (10), γ_R is the coefficient of variation of the examined change (deformation, cracking) of the structure.

6. Notations

6.1. Greek Letters

$\alpha_R, \alpha_G, \alpha_Q$ — sensitivity factor of the resistance, permanent and variable loads respectively
β — the safety index
γ — uniform safety factor
γ_0 — global safety factor
γ_G, γ_Q — safety factor of the permanent and variable loads
γ_i — safety factor of the geometrical data
γ_{Sn}, γ_{Rn} — the safety modification factor of the whole structure or a structural part with statistically underfineable factors, or with not ordinary economic importance (according to the designer's judgement)
γ_{Rt} — safety factor for resistance of such structures or structural parts which are designed for different than usual life duration (e. g. due to corrosion)
γ_{Sd}, γ_{Rd} — safety factor which takes into account the errors in the accepted structural or material models (according to the designer's judgement)
γ_{Qd} — equal safety factor of the variable loads
$\delta = \frac{D}{C}$ — damage coefficient
$\mu = \frac{Q_m}{G_m}$ — ratio of the permanent and variable load
ν_R, ν_G, ν_Q — coefficient of variation of the resistance, the permanent and the variable loads respectively
ν_{RS} — resulting coefficient of variation
σ_{max} — the greatest stress calculated according to linear elasticity
ρ — reinforcement ratio
Ψ — the frequency coefficient for loads.

6.2. Latin Letters

b — factor which depends on the strength standard deviation of structural materials ($b = 0.03 - 0.10$);
C — the original cost of the structure;
D — damage costs due to the unfavourable state;
E — the building industry's effective investment factor ($E \approx 0.15$);
E_s, E_c — modul of deformation of the reinforcement and the

		concrete, respectively
G_m	—	the expected value of the permanent load;
k_b, k_s	—	the correction factors of the design model which cannot be statistically treated;
L, L_{adm}	—	the design and the admissible change e. g. deformation, crack width;
$\sum Q_m$	—	the expected value of the simultaneous variable loads;
$R(R_m, L_m)$	—	the expected value of the resistance of the structure or the structural element;
R_{adm}	—	the allowed resistance;
R_H	—	the ultimate resistance of the structural element;
R_k	—	the characteristic value of the resistance;
R_m	—	the expected resistance;
s_R	—	the standard deviation of the resistance.

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