

UDK 624.024.046.5+006.036

**THEORY OF STRUCTURES AND DESIGN CODES****A.V. Perelmuter,**

Doctor of Technical Science, Senior Staff Scientist

*Kyiv National University of Construction and Architecture  
Povitroflotskyj av., 31, Kyiv, 03680**SCAD Soft Ltd  
Office 1, 3a Osvity street, Kyiv, 03037, Ukraine*

DOI: 10.32347/2410-2547.2022.108.3-16

Building structures should correspond to the reliability requirements which are implemented with the help of design codes. The latter are based on the method of limit states. In order to simplify the verifications, the design codes often deliberately deviate from the scientifically based theoretical provisions of such fundamental disciplines as the theory of elasticity and the theory of plasticity, replacing them with so-called working practices. The paper presents that there are inaccurately formulated recommendations in the design codes. The paper also specifies on some important problems that are not reflected in the design codes. This applies to the choice of failure probability values, the use of partial reliability factors, the calculation methodology in case of an emergency, the problems of using the results of nonlinear calculation, etc.

The paper presents some considerations on these issues, with the main attention being paid to the analysis of the existing design tradition and guidance to unresolved issues. The problems of recommended reliability parameters, clarification of the limit state concepts, analysis of accidental situations, the safety factors values and possible relationship between the safety factors for load and the safety factors for materials, loads and load effects, vulnerability assessment as well as reliability of protected systems have been considered.

The considerations presented by the paper give only a partial idea of the range of issues that arise when comparing working practices used in the design codes with the theoretical fundamentals they should correspond to. It should also be noted that the design codes do not provide any justifications for their recommendations. The presented paper can initiate a scientific discussion and be useful both for the developers of design codes and for the developers of software packages implemented the design codes.

**Keywords:** failure probability, partial safety factors, limit state, design codes.

**Introduction.** In order to simplify the analysis, design codes don't always strictly follow scientific approaches, replacing them with the so-called working practices. They are approximate not only in essence, but their justifications are often approximate as well, a detailed analysis of their origin and a comparison of the advantages and disadvantages of their application were once carried out by N.S. Streletsky [29]. However, more than sixty years have passed since the publication of [29], the design methods have changed and they are now based on computer modeling, and the class of design problems has expanded significantly.

Modern FEA software are based on such disciplines as the theory of elasticity, the theory of plasticity, structural mechanics, etc., while many of the working practices do not correspond, and sometimes even contradict to the fundamentals of these disciplines. This approach is used in programming because it can be applied to any problem, while working practices have been developed for certain special cases. However, since the working practices are

provided in the design codes, they suddenly become preferable over scientific approaches and more accurate solutions that do not appear in the codes only due to the complexity of the calculations. At the same time, certain technical, legal and economic problems arise due to the fact that the authors of design codes had not foreseen the possibility (and necessity!) of their software implementation [14].

This, however, does not mean that all the working practices have to be replaced with more justified, but also more time-consuming methods. The point is that a lot of working practices have been successfully used for such a long time that most practicing engineers are under the delusion that they accurately reflect the physics of the phenomenon, especially since their scope is not specified.

We believe there is an urgent need for a detailed description of those theories that are presented in the design codes by working practices, as well as to point out those important points of the theory of reliability that are absent in the codes.

Some thoughts on this issue are given below. These are just individual statements on the specified topics and not an exhaustive coverage of the problem as a whole. We believe that the presented text can initiate a scientific discussion and be useful both for the developers of design codes and for the creators of FEA software.

**Recommended Reliability Parameters.** The method of checking the reliability of structures, adopted in the design codes [1, 5, 10, 21], considers the probability of failure as a measure of reliability. It is based on comparing the failure probability  $P_f$  and the related reliability index  $\beta_f$ , with their allowable or target values  $P_{tag}$  and  $\beta_{tag}$ . The procedure for assessing the reliability of a structural system is reduced to the following inequalities:

$$P_f \leq P_{tag} \quad (1)$$

or

$$\beta_f \geq \beta_{tag}. \quad (2)$$

For example, three classes based on the consequences of failure are introduced in EN1990 [5], which are represented by target values of reliability indices  $\beta_{tag}$ . Ukrainian codes [21] provide these values for a 50-year period and they depend not only on the class of failure consequences, but also on the category of importance of the element, and on the load case.

The recommended values of the reliability indices  $\beta_{tag}$  in [5] are related to both the predicted consequences of failure and the relative cost of safety measures. ISO 2394:2015 [10] contains target reliability levels established on the basis of economic optimization using the life safety criterion, according to which the marginal cost of saving a life is estimated (Fig. 1).

They are based on the so-called compound social indicator – Life Quality Index (LQI). This compound indicator includes three important social parameters: the value of gross domestic product per capita; average life expectancy; the share of active working life.

The threshold (limit) value of  $P_{LQI}$  (see Figure 3), set using the LQI criterion, determines a certain range of acceptable values within which cost optimization should be performed.

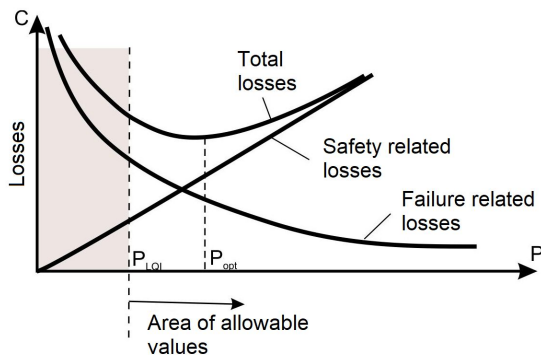


Fig. 1. Accounting for LQI in the optimization analysis

However, it is not explicitly stated whether the recommended value of  $\beta_{rag}$  applies to the design section, to a separate structural member or to the structural system as a whole, and how these probabilities correlate. By indirect evidence, most likely they are related to the considered design section.

As the analysis shows, the failure of an individual structural member usually has significantly less negative consequences than the complete destruction of the entire structural system or a significant part of it. This should be taken into account in optimization reliability analyses [3, 13, 21], while it should be expected that the target values of annual failure probabilities will differ for a structural member and for a structural system.

It is stated in [18] that, in general, the target reliability index of a structural element should be higher than the target reliability index for a structural system, except for systems with a high degree of static uncertainty. However, for series systems, the target reliability index is used as for a separate element, which leads to a decrease in the overall reliability index of the structural system as a whole (if the individual elements are to a certain extent independent).

**Clarification of Concepts of Limit States.** The ultimate limit state analysis usually considers the local bearing capacity of a design section. And the method of checking the bearing capacity adopted in the design codes, which is based on the idea of a consistent and independent assessment of the reliability of design sections, assumes that the elements of the system are connected in series, so that the failure of any of them leads to the failure of the entire system. Such an approach is quite justified for statically determinate systems [26], but it's not so certain whether it can be applied to statically indeterminate systems. Indeed, the failure of an element here does not automatically lead to the failure of the system, because a redistribution of forces is possible. Using this method we will only get higher reliability of course, but the price for such reliability remains unknown.

The concept of “bearing capacity” has a broader meaning, if we talk about the structure as a whole. It is known that violation of the strength condition in the section does not always lead to catastrophic consequences. However, it has

not yet been possible to formulate a sufficiently general limit state criterion for the entire structure, since each structure will have its own limit state, and, possibly, not the only one. This circumstance indicates another way of regulation, which is formulated in terms of the functional purpose of a building or structure, certain restrictions of which determine the limit state.

A typical example of this approach can be found in the US Federal Emergency Management Agency (FEMA) manuals, which contain 4 monitored performance levels [6]:

- (OL) – Operational Level. Backup utility services maintain functions; very little damage.
- (IOL) – Immediate Occupancy Level. The building remains safe to occupy; any repairs are minor.
- (LSL) – Life Safety Level. Structure remains stable and has significant reserve capacity; hazardous nonstructural damage is controlled.
- (CPL) – Collapse Prevention Level. The building remains standing, but only barely; any other damage or loss is acceptable.

Focusing on the performance characteristics of the structural behavior, among other things, allows you to use the idea of the possible implementation of several limit states during the life of a structure, since they are essentially reduced to interruptions in operation.

It should also be noted that one of the main ideas that form the basis of the limit state design method is the thesis that of all possible technical states of an operated structure, only its limit states are selected for the analysis. It is assumed that the behavior of the system before or after the limit state does not affect its operability (ultimate limit states) or the probability of difficulties in the process of its operation (serviceability limit states). And the linear analysis, which was used as the basis for developing the limit state design method, is not aimed at analyzing the post-critical behavior of the system.

However, the analysis of the system response for any fixed states is not

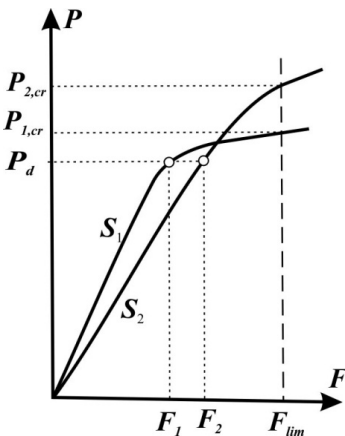


Fig. 2. Comparison of equilibrium curves

always sufficient to assess the reliability of the system. This fact becomes especially noticeable after a nonlinear analysis which takes into account the redistribution of forces in the system and reveals the actual limit state of the structure. The simplest example is given by a comparison of two systems  $S_1$  and  $S_2$ , a graphical illustration of which is shown in Fig. 2 as a relationship between the reaction  $F$  and the intensity of action  $P$ .

Comparison of their safety margins at the design value of the load  $P_d$  shows that the  $S_1$  system is preferable, but even a slight increase in  $P$  in the  $S_1$  system leads to a sharp increase in the reaction,

up to its critical value, which is not observed in the  $S_2$  system. Hence, a proposal appeared to consider not only the concept of the limit state but also the system behavior characteristic, which is determined by the gradient of the system response relative to the external action  $g = dF/dP$  [22].

It should be noted that this approach is almost always used in experimental studies of the structural operation, in which the experiment stops when, for example, a rapid increase in deflections begins.

**Analysis of Accidental Situations.** Ukrainian and Russian standards have ignored the analysis of accidental situations for a long time. This approach was based on the idea that limit states correspond not to accidental, but to pre-accidental situations. However, even within this concept of failure-free operation during a given service life, an external accident can still occur.

In other words, an accidental design situation is a phenomenon that represents exceptional conditions for the operation of a structure under accidental actions that have a low probability of occurrence and a short duration, but can usually lead to severe consequences if special measures are not taken.

After the analysis of these situations the list of limit states has been expanded: foreign codes began to consider accidental situations [5, 17, 9], and special limit states appeared which were considered as the third group of limit states [27, 28].

The codes [5] emphasize that the specified reliability requirements related to the limit state analysis do not take into account gross human errors. Therefore, the failure probabilities given in the codes are not applicable to the analysis of the special limit state (robustness, progressive collapse), which is largely dependent on human error effects.

Robustness analysis and analysis of the structural response to possible catastrophic impacts have now become an almost mandatory stage of the design process. Scientists started doubting some of the main ideas of the traditional approach to analysis, in particular, its focus on the statistical properties of loads and materials. Catastrophic events that entail severe consequences are extremely rare and there is not enough statistical data for them. Therefore, the main approach is to shift the focus from external actions to possible damages of the building. There are practically no probabilistic justifications for such an approach, although some attempts have been made in this direction [11, 19]. It was proposed, for example, to normalize the level of resistance of the structure to collapse by acceptable risk values [4, 16].

These publications took into account that the probability of collapse is determined by the probability of an accidental situation  $P[H]$ , the conditional probability of local damage of the considered element  $P[D | H]$ , the probability of its failure  $P[\text{Failure} | D]$ . Then the probability of the collapse of the system is determined by the following relationship:

$$P[\text{Collapse}] = P[\text{Failure} | D] \times P[D | H] \times P[H] \quad (3)$$

while accepting the condition  $P[H]=1$ .

For accidental situations that are the result of gross human errors, it is logical to assume that the probability of encountering them increases with the number of elements in the system  $n$ , although more slowly than linearly, since

the degree of control usually increases along with the complication of the system. Here, the relationship  $P[H]=C \ln(n)$  is suitable. In this case, the constant  $C$  must be sufficiently small, since we are talking about rare events. As for the probability of damaging a specific element, the elements are equivalent in this respect and we can assume  $P[D | H] = 1/n$ .

When assessing  $P[\text{Failure} | D]$  it should be taken into account that the usual approaches provided by the current design codes are not fully applicable to the problem of identifying the conditions of the total structural collapse. In particular, you should keep in mind that the values of the partial safety factors were taken based on the statistical properties of “usual” design situations, but if we consider special limit states that correspond to extreme damage values, we should focus on other socially acceptable values of the allowable collapse probability. Here we are talking about a situation characterized by a low probability of an event occurring with high socio-economic consequences of an accident.

Paying attention to this circumstance, a number of publications [2, 12] proposed to add an increment  $\Delta\beta_{tag} = 0,4$  to the reliability index in order to take into account the consequences of a total collapse.

**Values of the Safety Factors.** One of the fundamental ideas of this method was to take into account the statistical properties of those design parameters that cannot be precisely established. But the idea of taking into account only two statistically variable parameters (load and strength), which is the basis of bearing capacity analysis, turns out to be unreasonable in many cases. The thing is that the property of variability is also inherent in a number of other parameters, the values of which significantly affect the result of the analysis, but are not taken into account in the current codes. Let's point out some of them.

(a) The load effect (force, stress, etc.), which is compared with the bearing capacity, is by no means always related to the load by a linear deterministic relationship, which (and only it) allows the safety factor for load  $\gamma_f$  to be assigned to the load effect. The transformation from load to load effect can be performed using some parameters with random values. In this case, the aforementioned assignment of  $\gamma_f$  can lead to a gross error [23].

(b) The resistance parameter is directly related to the mechanical characteristics of the material and the safety factor for the material is determined only by the variability of the mechanical properties in the case of the strength analysis. When performing the stability analysis of a compressed bar, its bearing capacity is determined by its random initial imperfection and random eccentricity [26].

Indeed, in the case of a strength analysis of a centrally compressed bar, for example, a random value of the safety margin  $\tilde{S}_R$ , expressed in stresses, is presented as the difference between random values of ultimate stresses  $\tilde{\sigma}_{us}$  and compressive stress  $\tilde{\sigma}_0$ , the reliability index is determined by the formula:

$$\beta = \frac{\bar{\sigma}_{us} - \bar{\sigma}_0}{\sqrt{\hat{\sigma}_{us}^2 + \hat{\sigma}_0^2}}, \quad (4)$$

and in the stability analysis the safety margin is equal to

$$\tilde{S}_{stab} = \tilde{\sigma}_T - \tilde{\sigma}_0 - (\tilde{e} + \lambda^2 \tilde{f}) \frac{\pi^2 E \tilde{\sigma}_0}{\pi^2 E - \tilde{\sigma}_0 \lambda^2}, \quad (5)$$

its variability

$$A_{S_{stab}} = \frac{1}{1 - \psi} \sqrt{A_{\tilde{\sigma}_T}^2 + A_{\tilde{\sigma}_0}^2 + \frac{c^2 \psi^2}{(c - \lambda^2 \psi)^2} (\hat{e} + \lambda^2 \hat{f})}. \quad (6)$$

where  $c = \pi^2 E / \bar{\sigma}_{us}$ ,  $c = \pi^2 E / \bar{\sigma}_{us}$ ,  $\psi = \bar{\sigma}_0 / \bar{\sigma}_{us}$ ,  $\hat{e}$  and  $\hat{f}$  are variance of random values of eccentricity and initial imperfection, respectively.

Unlike (4) the reliability index is equal to

$$\beta = \frac{1}{A_{S_{stab}}} = \frac{\bar{\sigma}_{us} - \bar{\sigma}_0}{\sqrt{\frac{\bar{\sigma}_{us}^2}{\bar{\sigma}_0^2} \bar{\sigma}_0^2 + \bar{\sigma}_{us}^2 + \frac{c^2 \bar{\sigma}_0^2 \bar{\sigma}_{us}^2}{(c \bar{\sigma}_{us} - \lambda^2 \bar{\sigma}_0)^2} (\hat{e} + \lambda^2 \hat{f})}}. \quad (7)$$

In this case the analysis should obviously use not only the safety factors for the material and for the load.

The values of the partial safety factors are usually determined by a linear probabilistic analysis. The criteria for the limit state analysis, formulated in terms of limit forces, may not be applicable when there is no proportionality between the loads on the system and the internal forces and moments. At the same time, the question remains unanswered about using the results of the nonlinear analysis of forces, whether to apply the same factors that are used based on the results of the linear analysis or to introduce others (but which?), etc.

It is important that the verification of compliance with the requirements of any of the considered limit states uses both the safety factor for load  $\gamma_f$  and the safety factor for material  $\gamma_m$  and, therefore, the reliability of the structure is determined by both of these values. These factors are usually based not only on probabilistic and statistical data, but also on some additional considerations (control methods, data incompleteness, etc.). Therefore, the level of reliability is to a certain extent regulated by those additional margins that appear both on the left and on the right hand side of the limit inequality and depends on their consistency. But the established practice is such that the normalization of the values of  $\gamma_f$  and  $\gamma_m$  is carried out independently by different research teams, and the procedure for their coordination is not defined in any way.

**Possible Relationship between the Safety Factors for Load and Safety Factors for Material.** The safety factor for load and the safety factor for material are defined in such a way that these factors allow for the possible unfavorable deviations separately.

In most cases, in particular for all linear systems, this is true and the main inequality of the limit state design method is as follows:

$$\psi \gamma_n \gamma_f F_n \leq \gamma_c \gamma_m R_n, \quad (8)$$

here  $\psi$  is the combination factor,  $\gamma_n$  is the importance factor,  $\gamma_c$  is the service factor.

However, this is not always feasible in physically nonlinear problems, where the uncertainties of the action and resistance models can be closely related, for example, by using the same physical relationship  $\sigma=f(\varepsilon)$  both in the stress-strain analysis (i.e., load effect), and in the bearing capacity analysis.

And a real example, when the action effects and resistance parameters are not separated, is analyzed in detail in Eurocode-7. The Guide [30] says: “In contrast to the checking of structural designs, geotechnical actions from and resistances of the ground cannot be separated: geotechnical actions sometimes depend on the ground resistance, e.g. active earth pressure, and ground resistance sometimes depends on actions...”.

**Load and Load Effect.** The variability of loads and actions, allowed for by the factor  $\gamma_f$ , can be taken into account in the analysis in various ways. The thing is that not the values of the design loads  $F_d$ , but the values of the effects of these loads  $S_d$  (forces, stresses, displacements, etc.) are used in the design checks. However, the action effect is not only a function of the action itself, but of the characteristics of the design model as well, so its variability may differ from the characteristics of the action variability.

In practice, the probabilistic characteristics  $S_d$  are usually identified with the probabilistic characteristics of the load  $F_d$ , using the safety factor  $\gamma_f$  for  $S_d$ , the value of which is determined by the properties of the load.

This is always true when  $S$  is linearly dependent on  $F$ . Indeed, if  $S = cF$  ( $c$  is the influence coefficient) and  $F$  is a random variable with a mean  $\bar{F}$  and standard  $\hat{F}$  value, then the random variable  $S$  has the following mean and standard values:

$$\bar{S} = c\bar{F}, \quad \hat{S} = c\hat{F}, \quad (9)$$

and the coefficient of variation of the load effect is equal to the coefficient of variation of the load.

It will not be true for a nonlinear relationship  $S = f(F)$  though, and two approaches are possible when the partial factor  $\gamma_f$  is applied:

- either to the standard load values and then  $S_d = f(\gamma_f F_n)$ ;
- or to the action effect itself and then  $S_d = \gamma_s f(F_n)$ , where the safety factor  $\gamma_s$  has a value different from  $\gamma_f$ .

Such situations are typical for the analysis of geometrically nonlinear systems, where internal forces and moments can increase slower or faster than the load. In the first case we are dealing with geometrically hardening systems (most of the suspended structures), and in the second case – with geometrically degrading systems. Variability of the load effect for geometrically hardening systems (Fig. 3 (a)) is less than the variability of the load and, therefore  $\gamma_s \leq \gamma_f$ , and greater for geometrically degrading systems (Fig. 3 (b)), so  $\gamma_s \geq \gamma_f$ .

An even more complicated situation arises when the transition from  $F$  to  $S$  is such that the influence coefficient  $c$  turns out to be a random variable. Here, the design combination of loads (and the characteristics of the scatter of their values)



or the design combination of internal reactions of the system (forces, stresses, displacements) are different situations which depend on such random parameters as the position of the crane bridge on the crane beam and the position of the trolley on the crane bridge. The characteristics of the scatter of load values obviously do not coincide with the similar characteristics of the load effects for the crane beam and for the column. This fact was confirmed by statistical testing [23].

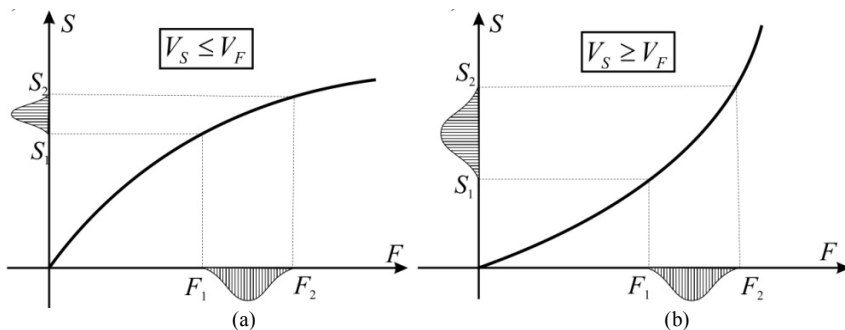


Fig. 3.  $S$ - $F$  diagram options

One of the results of statistical simulation is presented in the form of polygons of normalized values in Fig. 4. The normalization was carried out with respect to the data of the deterministic analysis. A relative value of 0.294 corresponds to the standard value of the bending moment in the crane beam caused by the load from two cranes with 95% reliability, i.e. the difference was threefold.

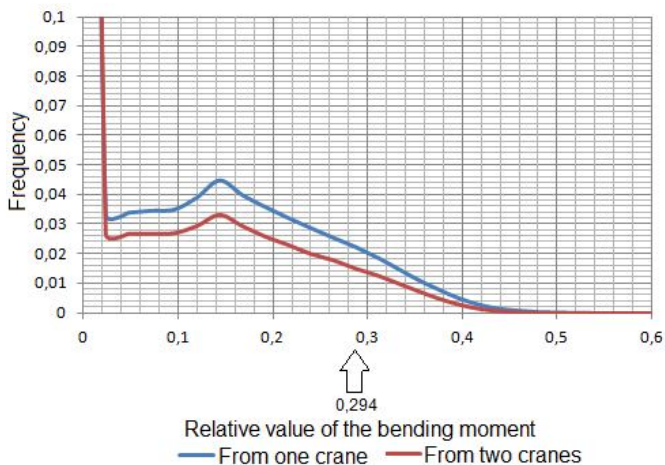


Fig. 4. Polygon of bending moments in a beam

**Vulnerability Assessment.** The limit state design method tacitly assumes that the design considers and takes into account all the loads and actions that

may occur during the life cycle of the designed structure. But in addition to clearly predictable loads and actions, there is always a possibility of a random action on the design structure that is not provided for neither by design codes nor by the designer's prediction. From the point of view of these surprise events vulnerability of the design object is an important characteristic.

Vulnerability characterizes a possibility of causing damages of any nature to the considered system by some external means or factors. Vulnerability is closely related to a well-known characteristic of "robustness" and to an additional characteristic — "mobilization" recently suggested in [24]. The robustness is considered as a spatial characteristic which shows how a local perturbation spreads throughout the space of the system and whether this local destruction can get a disproportionately large development "in breadth".

While mobilization shows the readiness and ability of the system to react to a local in time (pulse) unexpected perturbation. In both cases, the perturbation may be too strong to ignore its consequences, but its nature makes it impossible to predict the time and place of its occurrence, as well as other quantitative characteristics. Noticeable absence of the structural mobilization, as well as insufficient robustness, should serve as a reason for the increased attention and use of some protective measures.

**Reliability of Protected Systems.** Issues of analyzing load-bearing structures equipped with protection systems (seismic protection, fire protection, overload protection, etc.) are becoming increasingly common in the design practice. These systems change the nature of actions on the bearing structures, their intensity and, sometimes, statistical properties.

It is necessary to distinguish between protection devices that are included in the system as additional elastic, plastic or damping parts and change the static and kinematic properties of the protected system (for example, all seismic isolation systems), and protection devices that break when overloaded and remove the load from the protected structure (for example, protection against explosions in the form of easily removable structures).

There are no new fundamental issues in the first case, only the properties of the considered structure change, and its reliability increases due to these changes.

In the second case, the protection has an ambiguous effect on reliability. On the one hand, it reduces the probability of accidents, since an accident can occur only when the protection fails. If the protection is absolutely reliable, the crash failures do not occur at all. On the other hand, the probability of hang failures increases, since some of the crash failures are transformed into hang failures.

The issues arise here of checking the bearing capacity of both the protected structure [20, 15] (what is the safety factor for load), and the protection system which should have a guaranteed operability margin and, therefore, be guided by some values of partial factors. You should keep in mind here that an excessive increase in the breaking load by the protection leads to the fact that its "protective function" is reduced, and a decrease in this load leads to an increase in the number of hang failures.

**Conclusion.** The above considerations give only a partial idea of the range of issues that arise when comparing working practices with the theoretical

fundamentals they should correspond to. It should also be noted that the design codes do not provide any justifications for their recommendations. There is no such information in the textbooks as well. As a result, practicing engineers with standard education treat the design codes as the main source of knowledge.

We apparently need some supplements to the design codes like Background documents issued by the authors of Eurocodes. The guides to the Ukrainian and Russian codes have a different purpose (detailing, examples of application, etc.) and do not serve this function.

#### REFERENCES

1. ASCE SE17-16. Minimum Design Loads for Buildings and other Structures. – Reston, VA, USA, American Society of Civil Engineering, 2017.
2. *Diamantidis D., Sykora M.* Reliability Differentiation and Uniform Risk in Standards: a Critical Review and Practical Appraisal // *Future Tends in Civil Engineering – Zagreb, Croatia, 2019.* – P. 59–78.
3. *Ditlevsen O.* The Structural System reliability Problem, Qualitative Considerations of Reliability and Risk Analysis // *Civil Engineering, NC Lind (ed.), Institute for risk research, University of Waterloo., 1987, P. 1–11*
4. *Ellingwood B.* Acceptable risk bases for design of structures // *Progress in Struct. Eng. and Mat. 3(2).* – 2001. – P. 170 - 179.
5. EN 1990. Basis of Structural Design. – Eurocode European Committee for Standardization, 2002.
6. FEMA 273. Federal Emergency Management Agency. NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Washington, D.C. – October, 1997.
7. *Frank R., Bauduin C., Driscoll R et al.* Designer's guide to EN 1997-1, Eurocode 7: Geotechnical design — London: ICE Publishing, 2004 — 232 p.
8. *Gavrilenko I., Girenko S., Perelmuter A., Perelmuter M., Yurchenko V.* Load-bearing capacity as an interactive analysis tool in SCAD Office // *Proceeding of the METNET Seminar 2017 in Cottbus — Hämeelinna: HAMK, 2017— P. 112-127.*
9. GSA «Alternate path analysis & design guidelines for progressive collapse resistance», October 2016.
10. ISO 2394:2015. General principles on reliability for structures – International Organization for Standardization, 2015.
11. *Lin H., Yang L., Chen G.M. et al.* A novel methodology for structural robustness assessment of offshore platforms in progressive collapse // *Journal of Loss Prevention in the Process Industries, 2019, Vol. 62, Paper 103966.*
12. *Lu D., Song P.Yu.* Global seismic Reliability Analysis of Building Structures based on System / D. Lu, // *Level Limit States / Oct. 12- 17, 2008.* – Bijing, China.
13. *Madsen H.O.* Model Updating in Reliability Theory // «Reliability and Risk Analysis in Civil Engineering», Proceedings of ICASP 5, Lind N.C. (Ed.), Institute for risk research, University of Waterloo, Vol. 1, 1987, P. 564–577.
14. *Perelmuter A.V.* Strength analysis in regulatory design documents and computational software // *Strength of Materials and Theory of Structures, 2020, № 104 — P. 90-103.*
15. *Perelmuter A.V., Veriuzhska T.Y.* Optimization of the overload-protection degree // *Engineering Optimization IV — London: Taylor & Francis Group, 2014. — P. 529–532.*
16. *Stewart M.G., Melchers R.E.* Probabilistic risk assessment of engineering systems // *Chapman & Hall, London. – 1997.*
17. UFC 4-023-03 (Including Change 3, 2016) «Unified facilities criteria. Design of buildings to resist progressive collapse».
18. *Vrouwenvelder, A.C.* Developments towards full probabilistic design codes / *Struct. Safety, 24, 2002.* – P. 417–432.
19. *Yi Li, Xinzheng Lu, Hong Guan, Peiqi Ren, Lanping Qian.* Probability-based progressive collapse-resistant assessment for reinforced concrete frame structures // *Advances in Structural Engineering, 2016, Vol.: 19 issue: 11 — P.: 1723-1735.*

20. *Anilovich V.Ya., Lupandina A.P.* Protection Theory Used to Ensure the Reliability of Machines, // *Reliability and Durability of Machines and Structures*, Issue. 13. – Kyiv: Naukova Dumka, 1988. — P. 1-8.
21. DBN V.1.2-14:2018. General Principles of Reliability and Structural Safety of Buildings and Structures — Kyiv: Ministry of Regional Development, Building and Housing of Ukraine, 2018.— 30 p.
22. *Perelmuter A.V.* Using the Criterion of Resistibility to Assess the Structural Limit State. // *Vestnik MGSU*, Volume 16, Issue 12 — C.1419-1426. DOI: 10.22227/1997-0935.2021.12.0000-0000.
23. *Perelmuter A.V.* Statistical Modeling of Crane Loads and Design Combinations of Forces // *International Journal for Computational Civil and Structural Engineering*, 2017. Vol. 13, No 2.— P. 136-144.
24. *Perelmuter A.V., Pichugin S.F.* On One Safety Characteristic of Buildings // *Steel Structures*, 2014, Volume 20, No. 2 — P. 109–117.
25. *Perelmuter A.V., Tur V.V.* Are We Ready to Move on to Nonlinear Analysis in Design? // *International Journal for Computational Civil and Structural Engineering*, 2017. Vol. 13, No. 3 — P. 86-102.
26. *Rzhaniysyn A.R.* Theory of Reliability Analysis of Structures - Moscow: Stroyizdat, 1978. - 239 p.
27. SP 296.1325800.2017. Buildings and Structures. Accidental Actions - Moscow: Standartinform, 2017 — 23 p.
28. SP 385.1325800.2018. Protection of Buildings and Structures Against Progressive Collapse. Design Code. Basic Statements — Moscow: Standartinform, 2018 — 19 p.
29. *Streletsky N.S.* Importance of Working Practices in the Structural Analysis // *Izvestiya Vuzov. Construction and Architecture*, 1960, No. 5.
30. *Frank R., Bauduin C., Driscoll R., Kavvadas M., Krebs Ovesen N., Orr T., Schuppener B.* Designers' Guide to EN 1997-1 Eurocode 7: Geotechnical Design General Rules — Thomas Telford, London, 2005. — 213 p.

*Стаття надійшла 16.04.2022*

*Перельмутер А.В.*

### **ТЕОРІЯ СПОРУД І НОРМИ ПРОЕКТУВАННЯ**

Норми будівельного проектування ґрунтуються на методі граничних станів, за допомогою якого реалізуються вимоги надійності, які висуваються до будівельних конструкцій. З метою спростити їх використання норми достатньо часто свідомо відступають від науково обґрунтованих теоретичних положень таких фундаментальних дисциплін, як теорія пружності та теорія пластичності, натомість застосовуючи так звані робочі методи. У статті показано, що наявні неточно сформульовані рекомендації норм проектування, а також зазначається, що деякі важливі проблеми будівельного проектування і зовсім не відображені у нормах. Сказане відноситься до вибору значень ймовірності відмови, до використання часткових коефіцієнтів надійності, методики розрахунку у випадку аварійної ситуації, проблем використання результатів нелінійного розрахунку тощо.

У статті представлені деякі міркування щодо зазначених питань, при цьому головна увага приділена аналізу проектної традиції, що склалась, та вказівкам на нерозв'язані проблеми. Розглянуті проблеми рекомендованих параметрів безпеки, уточнення понять граничних станів, аналізу аварійних ситуацій, значень коефіцієнтів надійності та можливого зв'язку коефіцієнтів надійності за навантаженням та за матеріалом, навантажень та навантажувальних ефектів, оцінки уразливості та надійності захищеної системи.

Наведені у статті міркування дають лише часткове уявлення про коло питань, що виникає при зіставленні робочих методів норм проектування з теоретичними положеннями, яким вони повинні відповідати. При цьому слід зауважити, що практика викладення нормативних документів, яка склалась, ніяк не проголосувє зв'язок рекомендацій норм із дослідженнями, які їх обґрунтовують. Представлена стаття може покласти початок для наукової дискусії, а також бути корисною як для розробників нормативних документів, так і для розробників програмних систем, орієнтованих на розрахунок будівельних конструкцій.

**Ключові слова:** ймовірність відмови, часткові коефіцієнти надійності, граничні стани, будівельні норми

*Perelmuter A.V.*

### **THEORY OF STRUCTURES AND DESIGN CODES**

Building structures should correspond to the reliability requirements which are implemented with the help of design codes. The latter are based on the method of limit states. In order to simplify the verifications, the design codes often deliberately deviate from the scientifically based theoretical provisions of such fundamental disciplines as the theory of elasticity and the theory of plasticity, replacing them with the so-called working practices. The paper presents that there are inaccurately formulated recommendations in the design codes. The paper also specifies on some important problems that are not reflected in the design codes. This applies to the choice of failure probability values, the use of partial reliability factors, the calculation methodology in case of an emergency, the problems of using the results of nonlinear calculation, etc.

The paper presents some considerations on these issues, with the main attention being paid to the analysis of the existing design tradition and guidance to unresolved issues. The problems of recommended reliability parameters, clarification of the limit state concepts, analysis of accidental situations, the safety factors values and possible relationship between the safety factors for load and the safety factors for materials, loads and load effects, vulnerability assessment as well as reliability of protected systems have been considered.

The considerations presented by the paper give only a partial idea of the range of issues that arise when comparing working practices used in the design codes with the theoretical fundamentals they should correspond to. It should also be noted that the design codes do not provide any justifications for their recommendations. The presented paper can initiate a scientific discussion and be useful both for the developers of design codes and for the developers of software packages implemented the design codes.

**Keywords:** failure probability, partial safety factors, limit state, design codes

*Перельмутер А.В.*

### **ТЕОРИЯ СООРУЖЕНИЙ И НОРМЫ ПРОЕКТИРОВАНИЯ**

Нормы строительного проектирования основываются на методе предельных состояний, с помощью которого реализуются требования надежности, выдвигающиеся к строительным конструкциям. С целью упростить их использование нормы зачастую сознательно отступают от научно обоснованных теоретических положений таких фундаментальных дисциплин, как теория упругости и теория пластичности, подменяя их так называемыми рабочими методами. В статье показано, что имеются неточно сформулированные рекомендации норм проектирования, а также указывается, что некоторые важные проблемы строительного проектирования и вовсе не отражены в нормах. Сказанное относится к выбору значений вероятности отказа, к использованию частных коэффициентов надежности, методике расчет в случае аварийной ситуации, проблемам использования результатов нелинейного расчета и др.

В статье представлены некоторые соображения по указанным вопросам, при этом главное внимание уделяется анализу сложившейся проектной традиции и указаниям на нерешенные проблемы. Рассмотрены проблемы рекомендуемых параметров безопасности, уточнения понятий о предельных состояниях, анализа аварийных ситуаций, значений коэффициентов надежности и возможной связи коэффициентов надежности по нагрузке и по материалу, нагрузок и нагрузочных эффектов, оценки уязвимости и надежности защищенной системы.

Приведенные в статье соображения дают лишь частичное представление о том круге вопросов, который возникает при сопоставлении рабочих методов норм проектирования с теоретическими положениями, которым они должны соответствовать. При этом следует отметить, что сложившаяся практика изложения нормативных документов связь своих рекомендаций с обосновывающими их исследованиями никак не оглашает. Представленная статья может положить начало для научной дискуссии, а также быть полезной как для разработчиков нормативных документов, так и для создателей программных систем, ориентированных на расчет строительных конструкций.

**Ключевые слова:** вероятность отказа, частные коэффициенты надежности, предельное состояние, строительные нормы

УДК 624.024.046.5+006.036

*Перельмутер А.В. Теорія споруд і норми проектування* // Опір матеріалів і теорія споруд: наук.-тех. збірн. – К.: КНУБА, 2022. – Вип. 108. – С. 3 – 16.

*Норми будівельного проектування ґрунтуються на методі граничних станів, за допомогою якого реалізуються вимоги надійності. Однак ці вимоги, що представлені в нормах, не завжди відповідають основним теоретичним положенням. Є неточно сформульовані рекомендації, деякі важливі проблеми не відображені в нормах. Це відноситься до вибору значень ймовірності відмови, до використання часткових коефіцієнтів надійності, методики розрахунок у разі аварійної ситуації, проблем використання результатів нелінійного розрахунку та ін. У цій роботі представлені деякі міркування з зазначених питань, при цьому головна увага приділяється аналізу проектної традиції, що склалася, і вказівкам на невирішені проблеми.*

Іл. 4. Табл. 0. Бібліог. 30 назв.

УДК 624.024.046.5+006.036

*Perelmuter A. V. Theory of structures and design codes* // *Strength of Materials and Theory of Structures: Scientific-and-technical collected articles* – Kyiv: KNUBA, 2022. – Issue 108. – P. 3–16.

*Building design standards are based on the method of limiting states, with the help of which reliability requirements are implemented. However, these requirements are presented in the norms do not always correspond to the fundamental theoretical provisions. There are inaccurately formulated recommendations, some important problems are not reflected in the norms. This applies to the choice of failure probability values, the use of partial reliability factors, the calculation methodology in case of an emergency, the problems of using the results of nonlinear calculation, etc. This paper presents some considerations on these issues, with the main attention being paid to the analysis of the existing design tradition and guidance to unresolved issues.*

Figs. 4. Tabs. 0. Refs. 30.

УДК 624.024.046.5+006.036

*Перельмутер А.В. Теория сооружений и нормы проектирования* // Сопротивление материалов и теория сооружений: науч.-тех. сборн. – К.: КНУСА, 2022. – Вип. 108. – С. 3-16.

*Нормы строительного проектирования основываются на методе предельных состояний, с помощью которого реализуются требования надежности. Однако эти требования представлены в нормах не всегда соответствуют основополагающим теоретическим положениям. Имеются неточно сформулированные рекомендации, некоторые важные проблемы никак не отражены в нормах. Это относится к выбору значений вероятности отказа, к использованию частных коэффициентов надежности, методике расчет в случае аварийной ситуации, проблемам использования результатов нелинейного расчета и др. В этой работе представлены некоторые соображения по указанным вопросам, при этом главное внимание уделяется анализу сложившейся проектной традиции и указаниям на нерешенные проблемы.*

Ил. 4. Табл. 0. Библиог. 30 назв.

**Автор:** доктор технічних наук, старший науковий співробітник Перельмутер Анатолій Вікторович

**Адреса робоча:** 03037 Україна, м. Київ, вул. Освіти 3А, оф. 2

**Робочий тел.:** +38(044)249-71-91

**Мобільний тел.:** +38(050)382-16-25

**E-mail:** [avp@scadsoft.com](mailto:avp@scadsoft.com)

**SCOPUS ID:** 6602444316

**ORCID ID:** <https://orcid.org/0000-0001-9537-2728>