MINISTRY OF EDUCATION AND SCIENCE OF UKRAINE
NATIONAL UNIVERSITY «YURI KONDRATYUK POLTAVA POLYTECHNIC»

**SERGII PICHUGIN, LINA KLOCHKO,
KATERYNA OKSENENKO**

METHODS OF LIMIT STATES AND LOAD STANDARTIZATION

Manual

for students of specialty 192 “Construction and civil engineering”

Poltava 2022

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Рецензенти:

В.А. Пашинський *(доктор технічних наук, професор, професор кафедри будівельних, дорожніх машин і будівництва Центральноукраїнського національного технічного університету);*

О.В. Семко *(доктор технічних наук, професор, завідувач кафедри будівництва та цивільної інженерії Національного університету «Полтавська політехніка імені Юрія Кондратюка»)*

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Пічугін С.Ф., Клочко Л.А., Оксененко К.О.
Методика граничних станів і нормування навантажень: навчальний посібник для студентів спеціальності 192 «Будівництво та цивільна інженерія» / С.Ф. Пічугін, Л.А. Клочко, К.О. Оксененко. – Полтава: Національний університет «Полтавська політехніка імені Юрія Кондратюка», 2022. – 241 с. (англ. мовою).

Навчальний посібник призначений студентам 5-го курсу (9-10 семестри), які навчаються англійською мовою за освітнім рівнем «магістр» за спеціальністю 192 «Будівництво та цивільна інженерія». Детально розглядаються основні положення і параметри методики граничних станів. наводяться роз'яснення щодо походження і обґрунтування основних засад і розрахункових коефіцієнтів методики. Послідовно викладаються дані щодо навантажень, що є основними для будівель і споруд: снігових, вітрових, кранових, ожеледно-вітрових. У посібнику використовуються результати досліджень навантажень, які на протязі багатьох років виконуються в університеті.

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CONTENTS

INTRODUCTION

Discipline "Methods of limit states and load standardization" is taught to 5th year students (9 - 10 semesters), who study at the educational level "Master" in specialty 192 "Construction and Civil Engineering" at the Department of Building Structures, National University "Poltava Polytechnic Yuri Kondratyuk".

The need to study of this discipline is due to the fact that the calculation method of limit states is applied to all building structures, it is the basis of codes for their design, from the 50s of last century to the present. Without confident mastery of this basic technique, it is impossible to become a full-fledged construction specialist, able to make competent and sound decisions about load-bearing capacity, rigidity, reliability and other necessary qualities of load-bearing and enclosing building structures.

The manual begins with the history of the development of structural strength calculations, proceeds to the method of allowable stresses, which prevailed in construction in the 18-19 centuries and in the first half of the 20th century, and examines in detail the main provisions and parameters of the limit states. In this case, the presentation of the material is not limited to the provisions of the design codes and includes an explanation of the origin and justification of the basic principles and calculation coefficients of the methodology.

The crucial role in the calculations of building structures is played by the correct consideration of the loads and loadings acting on them. The manual consistently presents data on the loads that are essential for buildings and structures: snow, wind, crane, ice-wind. The nature of loads, methods of their research, standardization and probabilistic justification are considered. The manual uses the results of load studies that have been performed at the University for many years.

The manual is quite voluminous with a focus on working with students in computer classes of the University and the active use of the electronic version of the manual.

The manual "Methods of limit states and load standardization" is also recommended for students of other departments and courses, as well as graduate students and civil engineers who want to expand their horizons in the field of basic principles of building structures calculation and the load theory.

Authors

LECTURE 1. METHOD OF ALLOWABLE STRESSES

1.1. Introduction

Calculation is one of the stages of structural design. Design is begun with the development of a structural scheme of the building. Only after the basic general sizes of a construction, a structural form of elements and means of their connection are established, it is possible to pass to calculation of structure.

The purpose of the calculation is to check the strength, stability and rigidity of the pre-planned structural scheme of the construction and further correction of the dimensions of the elements and their cross sections. It is necessary to solve two opposite problems:

- on the one hand, the structures must be economical, that means to have minimal material and labor costs for manufacture and installation;
- on the other hand, it is necessary to ensure the ***reliability*** of structures and their trouble-free operation for the entire working life with certain reserves. The reserves have to take into account random loads, unpredictable deviations in the properties of materials, the difference between the actual operation of structures from the theoretical model.

Structural calculation is complicated by the fact that working life of structures can be tens of years, or for unique structures it can be hundreds of years. With such long-term operation, it is difficult to predict their behavior and possible effects on them.

It has been established that, the calculation of structures and their elements is performed on the basis methods of materials resistance and structural mechanics. The calculation determines the internal forces and displacements that occur in structures under the influence of applied loads. At present, due to the development of computer technology, there are no fundamental difficulties in structural calculation of any complexity with the accuracy required for practice. However, the methods of resistance of materials and structural mechanics do not answer the following important questions:

- what loads must be taken to structural calculation;
- with what value should be compared the forces, stresses and displacement which were obtained by calculating

- how to take into account the deviations (inadequacy) of the idealized theoretical model and the actual structure;
- how to assess the reliability of the structure and ensure its smooth-running operation throughout its working life.

The answers to these and other questions should be given by the methodology of structural calculation.

1.2. A brief history of the development of the structural calculation

Despite thousands of years of construction experience, the problem of building strength has always existed, it is relevant today. For a long time, structural mechanics did not exist. That's why even in the most perfect ancient buildings, gross errors can be found, indicating their ignorance of the basics of resistance to materials and the theory of buildings. Builders determined the strength intuitively, by trial and error. They have learned from accidents and crashes. Superstitious fear of the unrecognized secret of the material forced the builders to seek the help of otherworldly forces with the involvement of prayers (which continues today), conspiracies and even sacrifices.

From ancient times, the profession of builder was considered very responsible. The possible construction errors had very serious consequences for those who assumed them. In particular, the Code of Hammurabi were compiled in 1750 BC. These laws regulated the responsibility of the builders of ancient Mesopotamia as follows:

«§ 229. If a builder built a house for someone, and does not construct it properly, and the house which he built fall in and kill its owner, then that builder shall be put to death.

§ 230. If it killed the son of the owner the son of that builder shall be put to death.

§ 231. If it killed a slave of the owner, then he shall pay slave for slave to the owner of the house.

§ 232. If it ruined goods, he shall make compensation for all that has been ruined, and inasmuch as he did not construct properly this house which he built and it fell, he shall re-erect the house from his own means».

Here is also a relatively recent example. In 1830, the famous Russian architect Karl Ivanovich Rossi, who began the construction of the Olexandrinsky Theatre in St. Petersburg, proposed the use of arched roof trusses made of metal. Several major experts at the time doubted the strength of these structures and halted construction. Offended Rossi immediately wrote to the minister of the court: «In the event that some kind of misfortune occurs in the said building due to the installation of a metal roof, then as an example for others, let them immediately hang me on one of the truss». This was the only argument, because the theory of calculating such farms did not exist then Trusting the intuition of the great architect, the trusses were built and are still in place today. There is such a tradition even now among bridge builders.

Collapses not only happened in the ancient and middle ages, they continued later and still happen today. Each collapse added new knowledge to the builders, set new tasks. When knowledge was lacking, a reserve factor was introduced (and it is being introduced now) in engineering calculations. For example, we determined the load that can withstand the element during operation, and selected its dimensions that could withstand loads greater than, say, 100 times the operating. This meant that the created element had a reserve factor which was equal to 100. Since no one knew what unpredictable, unknown phenomena this factor takes into account and whether it should be exactly that and not less, for example in 10 times, it was called the **coefficient of ignorance**.

The history of the development of construction science preserves such famous names as Archimedes, Leonardo da Vinci, Galileo Galilei, Robert Hooke, Thomas Jung, Leonard Euler and others, who developed the question of the reserve factor. But for the first time in the science of construction, the reserve factor was introduced by the famous French engineer and scientist Louis Marie Henri Navier (1785-1836). Already being an academician, Navier in 1826 published a course of lectures in which he laid the foundations of the theory of elasticity and introduced the concept of stress. Navier proposed to set the calculated allowable stresses at which the structure can work reliably, and to calculate these stresses. Obviously, the allowable stresses must be much less than destructive stresses. Navier wrote - "Resistance to destruction is not enough to design, because you need to know not the destructive force, but the one that can load the element without the changes that occur in it growing over time." If the steel beam breaks at a stress of 4000 kgf/cm^2, Navier suggests accepting a permissible stress of 1300 kgf/cm^2 when it bent. At the same time the sizes of a beam are accepted such that during operation it had stresses, not above admissible stresses.

With the development of construction science, the reserve factor that is the coefficient of ignorance, was changed, so in fact the whole history of the science of strength was the history of the struggle to reduce this coefficient of ignorance. Now this ratio has become relatively small (this will be discussed later), but it took centuries.

Thus, starting from the XIX century, the reserve factor was established on the basis of engineering intuition, experience in the design and operation of structures and was dominated in building mechanics until the 50's of the twentieth century.

1.3. Procedure of calculation by allowable stresses

The calculation of this method is as follows: the stresses arising in the structural elements should not exceed the allowable stress $[\sigma]$:

$$\sigma \leq [\sigma]. \tag{1.1}$$

Allowable stresses $[\sigma]$ make up a some part of the dangerous (limit) stresses. For plastic materials, such dangerous (limit) stresses are the yield strength σ_y. This is when deformations increase rapidly and interfere with the normal operation of the structure. In particular, for steel construction, the allowable stress is equal to the yield strength divided by the reserve factor k_1:

$$[\sigma] = \sigma_y / k_1. \tag{1.2}$$

For brittle materials, the dangerous stress is the limit strength σ_u at which the material breaks. So, for the allowable stress we have:

$$[\sigma] = \sigma_u / k_2, \tag{1.3}$$

The reserve factor relative to the yield strength k_1 is taken in the range from 1,2 to 2,5; the reserve factor relative to the strength limit for brittle materials k_2=3…5 (and sometimes higher, for example, for natural and artificial stones, it can be in the range of 10… 30). With the improvement of the quality of materials and the refinement of the calculation theory, the values of allowable stresses are obviously increase.

The calculation was performed for loads corresponding to normal operating conditions. The loads were not taken into account alone, but in subsequent combinations.

1. Basic combinations of actions there are combinations of such loads that regularly act on the structure and the resistance of which is the main purpose of the structure. The example of the simplest combination is a combination of permanent and imposed loads. For many structures, such as ceilings, atmospheric snow loads are added to these loads. Loads of basic combinations of actions are called *basic loads.*

2. Additional combinations of actions there are combinations of main loads with *additional* ones, which are relatively rare. Their perception is not included in the purpose of the structure. The most typical additional load was hurricane wind. It was believed that it is rare and its perception is not the purpose of the structure, which is designed to perceive the payload. In some cases, the additional load may be mounting cranes, which rarely work only during installation.

3. Special combinations of action there are combinations of main and additional loads (usually not all) with very rare special effects, often emergency. A typical example of special loads is seismic loads in seismic areas.

For steel structures as a yield strength was taken as its value, set in the technical conditions for the supply of metal. In the 50's of the last century, this value for the steel of St. 3 was equal to 2100 kgf/cm^2. The reserve factor was assumed to be equal $k_1 = 1{,}36$. Therefore, the allowable stress was equal to:

$$[\sigma] = 2100/1{,}36 = 1600 \text{ kgf/cm}^2.$$

This usual value was taken into account only for the main loads. Under the action of the main and additional loads, the reserve factor was taken lower, and the allowable stress was increased to $[\sigma] = 1800$ kgf/cm^2.

1.4. Disadvantages of the method of allowable stresses

In principle, the reserve factor k should take into account adverse factors affecting the operation of structures and are not taken into account directly in theoretical calculations. Here we can name such factors as:

- conditions of construction and operation of the structure;
- working life of the structure;
- type of effort;
- loading condition and loads are not taken into account;
- inevitable fluctuations in material quality;
- proximity of calculation.

Therefore, we can say that the reserve factor is a generalizing indicator that ensures the safety of structures. Obviously, such a load on a single factor is excessive. It should be noted here that the reserve factor is the main factor. It takes into account only general factors and only at static load. Taking into account the dynamics of the load and various additional factors (stress concentration, variability of loads, longitudinal and off-center bending, etc.) is carried out by introducing additional coefficients by which the basic coefficient is multiplied.

In the middle of the twentieth century, the method of allowable stresses seriously constrained the development of building structures due to a number of uncertainties, of which we can highlight the following:

- loads, which were considered in the calculations, corresponded to normal operating conditions, without regard to the possibility of exceeding them;
- the possibility of using material with reduced characteristics in comparison with the technical conditions was not taken into account in the structure;

- it was assumed that the actual operating conditions of the structure will correspond to the idealized conditions accepted in the calculation;
- the reserve factor, which had to take into account these factors, for all structures of this material remained unchanged, regardless of the specific operating conditions of structures and the degree of their responsibility.

As a result of this approach, different structures had ***different reliability.*** This was pointed out in the pre-war years by the preeminent Soviet scientist-builder N.S. Streletsky. It is known that the own weight of structures can not really fluctuate more than 10%. Temporary loads - snow, wind, weight of people and equipment – can be varied much more during operation. Therefore, at different ratios, the real factor of safety of structures differs significantly from the theoretical. With the reserve factor approximately egal 2 and a temporary to permanent load ratio of 0,25:1,0, even a threefold increase in temporary load gives only a 40% increase in total stresses compared to the normative value, which does not pose a threat to the structure. At the same time, with the value of this ratio 4:1, a twofold increase in the temporary load leads to 80% overload of the structure, which while reducing the strength of the material by 20% against the norm can lead to an accident. We will have a similar picture in all cases of calculation of structures in the presence of several independent force influences, each of which can change to a different degree.

In addition, the work of structures, in particular steel structures, was considered only in the ***elastic state***, without taking into account the plastic properties of the material, which reduced their efficiency. The criterion for calculating the structures also remained unclear: what will happen if condition (1.1) is violated. Therefore, a single reserve factor is unacceptable in cases where the hazardous situation is far beyond the proportionality between stresses and forces.

At a certain stage in the development of methods for calculating structures (early twentieth century) there was a reasonable impression that the method of allowable stresses does not give an idea of the true factor of safety, includes too large reserves and leads to overuse of valuable materials from which the structure is made. Considering this, in the first half of the twentieth century, the allowable stresses for steel were increased by about half, for concrete - and a half times. While the allowable stresses were increasing, there were no strong arguments for criticizing this method.

However, as the calculations were clarified, it turned out that there are zones in the structures where the stresses are equal to the limit and even exceed them, but the structures are not destroyed, but safely operated. As an example, we will cite steel trusses, which are usually calculated as hinged systems, although in fact their nodes are rigid. Taking into account the influence of rigidity of the nodes and the study of stress distribution in the gusset plate showed that steel trusses, which operate well even under dynamic loads, have zones in the gusset plate where the calculation reveals stresses exceeding the limit. One of the most striking experiments was also performed in 1908: the ends of the steel beam were rigidly clamped in concrete massifs, and the beam was loaded with a uniform load. Knowing the span of the beam, its profile and the moment of resistance, it was easy to calculate the uniform load at which the beam was to collapse. Experience has shown, however, that the destruction of the beam occurred at a load that is 30… 40% higher than the design load.

In some cases, the calculation of allowable stresses indicated the need to reinforce the structures that have existed successfully for many years.

All this was the basis for the development of a new method of calculating structures by limit states, which was adopted in the USSR in 1955 with the approval of the main design guiding document - "Building Norms and Rules" (SNiP).

The method of allowable stresses is still used, in particular, in mechanical engineering. It can be assumed that this is due to the fact that, unlike building structures, machine structures are operated in relatively stable, fairly accurately predicted conditions, for a limited period of time. Under these conditions, a single stock ratio probably reliably covers possible adverse factors in the operation of machines and mechanisms.

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Control questions

1. What is the main purpose of structural design?
2. How is the calculation of structures for allowable loads performed?
3. What must be considered in the factor of safety?
4. What are the disadvantages of the allowable loads method?

LECTURE 2. STATISTICAL JUSTIFICATION OF THE RESERVE FACTOR

2.1. The structure of the reserve factor
2.2. Statistical approach to determining the reserve factor
2.3. Guarantee of the non-destructiveness
2.4. Scientific feat N.S. Streletsky
2.5. Life path of N.S. Streletsky

2.1. The structure of the reserve factor

The basic principle of engineering and engineering calculation is ***the condition of indestructibility***, according to which the applied force acting in the structure during its working life should be less or, at least, equal to the lowest possible during this time the limit resistance of the structure material:

$$max\, S_{structure.}^{fact.lim\,it.} \leq min\, S_{material of structure}^{fact.lim\,it.}. \tag{2.1}$$

Accordingly, the main issue of engineering calculation is to determine these efforts. Undoubtedly, this task is extremely difficult, because we are dealing with hypothetical efforts.

Streletsky Nikolay Stanislavovich was the first to note that the fulfillment of this inequality can be predicted only with a certain degree of probability. In his small but extremely meaningful work "Fundamentals of statistical accounting of the reserve factor of structures" [1], he argued that following a statistical path, studying and comparing the facts of a homogeneous group of structures and materials in structures, we can establish the law of these factors and extrapolate this law to the future, if there are sufficient grounds for it.

The condition of indestructibility can be rewritten in a more detailed form in conjunction with the method of allowable stresses:

$$max\, S_{structure.}^{fact.lim\,it} = k \cdot S_{design.force.} \leq \mathrm{c} \cdot S_{normative.}^{material.}\, min\, S_{material of structure}^{fact.lim\,it}, \tag{2.2}$$

where k is the reserve factor relative to the effort (stress);

c is the transition factor from the actual stress of the material to the normative stress, which has the same nature as the reserve factor k, which was sometimes called the material quality factor. The normative limit stress can be taken as a defective minimum of material or some other, more convenient, value.

If we transfer the coefficient c to the left, we obtain the following expression:

$$k\frac{1}{c}S_{design.force} = k_0 \cdot S_{design.force.} \leq S_{normative.}^{material.}. \quad (2.3)$$

where k_0 is the calculated reserve factor relative to the normative resistance of the material.

N.S. Streletsky was first, who presented the reserve factor as the product of its constituent components:

$$k = k_1 k_2 \dots k_n = \prod k_i. \quad (2.4)$$

The reserve factor structure in the form of a supply of multipliers is called ***the canonical reserve factor structure***. This structure is convenient because the number of multipliers in it can always be set depending on the course of the survey. This structure corresponds most closely to the practice of calculation, according to which specific cases of structure's work and material are traditionally evaluated by coefficients, which are included in the calculation as multipliers. Thus, if the risk of losing the stability of the flexible rod must be taken into account, the buckling coefficient φ is introduced. If it is necessary to include the risk of fatigue, loss of strength, prolonged support, etc., a corresponding factor of reduction of load-bearing capacity in case of fatigue, loss of strength, etc. must be applied. All these factors are counted as multipliers of the overall reserve factor.

Each coefficient k_i is a ratio of a certain variable (actual load, actual state, etc.) to a certain value that was taken as a unit of comparison, which is the estimated load, estimated state, etc. Each coefficient k_i, in turn, can be broken down into elements. However, it should not be necessary to excessively increase the number of coefficients, since the efficiency of such increase is strongly extinguished by errors in each of them. However, that the more general coefficients, which depend on a large number of factors, are more stable.

Furthermore, this methodology does not require a large number of coefficients. It is recommended to combine the coefficients into three groups:

1) load mode group (coefficient k_1);
2) the group of conditioning of the calculation (coefficient k_2);
3) the group of building state (coefficient k_3).

A distinctive feature of these groups is that they can be considered ***independent*** of each other. Actually, if it is assumed that the state of the building is regarded as normal and the building is operated without any particular restrictions, the conditions of operation (load conditions), the method of calculation and the actual condition of the structure are completely independent of one another and may be considered as circumstances that have no correlation relationships (in terms of mathematical statistics or the theory of probability).

2.2. Statistical approach to determining the reserve factor

N.S. Striletsky correctly showed that each of the coefficients, which characterises some specific feature of the structure's work, depends on a large number of causes and circumstances that may occur during the operational life of the structure, and thus it can be described best with the help of the ***statistical method***.

For this purpose, as a result of statistical processing of observations, it is established how often during the operation of the buiding appears one or another value of the phenomenon under study. The results of these observations are presented in the form of graphs (frequency curves), the abscissa axis of which shows possible values of the researched argument (phenomena), and the ordinate axis shows the number of occurrences of each value or frequency of their appearance (the construction of frequency curves will be described in detail below).

Having received on the basis of observations experimental or calculated values of coefficients k_i and having constructed for them frequency curves, it is possible to found the maximum values of coefficients k_i. The values of the general reserve factor can be obtained by these coefficients:

$$k = max\, k_1 \cdot max\, k_2 \cdots max\, k_n. \tag{2.5}$$

However, this definition has two disadvantages.

Firstly, accurate determination of the maxk_i values is difficult, because the maximal values of coefficients are unreliable, as they are determined by a small number of observations, and the limit (the highest) value - may be determined by even one observation. In addition, there is no reason to claim that there cannot be higher values in the future, than which is taken as limit value on the basis of past experience.

Secondly, as the k_i coefficients are independent of each other, we cannot assume that the maximum values of these coefficients can both occur at equation at the same time. Therefore, it cannot be argued that with a practical degree of probability, equality can be:

$$k = max(max\, k_1\, , max\, k_2\, , \cdots max\, k_n) = \prod max\, k_i. \tag{2.6}$$

To overcome these difficulties, it is necessary:

1) Increase the probability of the curve extremities by using all of the reliable middle part of the curves, which is based on a large number of observations;

2) set the limits of the curves on the basis of a very objective criterion.

For this aim it is necessary to replace the investigated curves by theoretical distribution curves, which are the closest to them and which present a perspective representation of the investigated data with a sufficiently large number of observations. These curves, as noted, can be considered equidistant at all points.

In order to overcome inequality

$$max \prod k_i = max(k_i, k_2, \dots, k_n) \neq max\, k_1 \cdot max\, k_2 \cdots max\, k_n \qquad (2.7)$$

due to the incompatibility of the maximum values of the coefficients k_i, it is necessary to construct a distribution curve for the value Πk_i of the product of the values k_i. Such construction can be done basing on the apparatus of the probability theory [3]. It should be noted that the distribution of the product of random variables goes to the lognormal distribution.

The coefficient c can be represented by at least a sum of two coefficients:

- coefficient c_1, which is the ratio of the actual material resistance of the sample to the normative resistance of the material

$$c_1 = S^{actual}_{materialofstructure.} / S^{normative}_{material}; \qquad (2.8)$$

- coefficient c_2, which is the ratio of the actual resistance of the material in the structure to the actual resistance of the sample material

$$c_2 = S^{actual}_{materialofstructure.} / S^{actual}_{materialofsample}. \qquad (2.9)$$

The coefficient c_1 is determined by the sample material's properties (under laboratory conditions). For a large number of materials the curves $S^{actual}_{materialofsample}$ and c_1 appear as normal Gaussian curves, which are well understood.

The coefficients c_2 depend on the type and structure of the building. In structures that are structurally simple, such as stone pillars, walls, etc., the value of the coefficient c_2 depends primarily on the circumstances of construction and, obviously, can be obtained from experience by direct testing of structures; the coefficient can be divided into a number of intermediate coefficients, depending on the circumstances of construction. In structures that are structurally complex, such as farms, frames, in addition to the issues of construction and manufacture, the influence of structural characteristics of the structure must be significant. As a first approximation, the structure of the building can be characterized by the fact that it consists of many elements that can be destroyed separately. In this regard, structures can be statically defined, in which the destruction of one (each) element is already the destruction of the system, and can be statically indeterminate, the destruction of which requires the destruction of several elements, a number equal to the number of static connectivity.

2.3. Guarantee of the non-destructiveness

The condition of non-destructiveness requires a combination of extreme values of the curves $kS_{calculated}$ and $cS_{normative}$. However, since the curves k and $kS_{calculated}$ and the curves c and $cS_{normative}$ are asymptotic, the exact fulfillment of this condition is not possible, because we do not know the extreme values of the curves. Thus, the condition of non-destructiveness is possible only with a certain accuracy. For this purpose it is necessary to conditionally break the specified curves at a certain point and to connect the cut off curves. The measure of the accuracy of such combination, obviously, is the rejected areas of the curves at the actual point of intersection or the product of these areas.

Rejecting the areas of the curves, we take them to be practically zero and connecting the curves in this way, we claim that the condition of non-destructiveness is practically fulfilled and our structures are practically non-destructive.

Thus, the product of rejected areas $\omega_1 \cdot \omega_2$ can be considered as a measure of inaccuracy of the statement that the structure is non-destructive, and the value

$$G = 1 - \omega_1 \cdot \omega_2, \tag{2.10}$$

can be considered as a measure of the accuracy of the statement that the structure is non-destructive.

Therefore, this value G (2.10) was called by M.S. Striletskii as a ***guarantee of the non-destructiveness of the structure.*** He emphasizes that the value of the guarantee of non-destructiveness is quite conditional value associated with the fulfillment of the condition of non-destructiveness (*Fig. 2.1*).

For the first time N.S. Streletsky determined the numerical values of the guarantee of non-destructiveness in 1938. Steel trusses under a cold reinforced concrete roof for the Moscow region were considered. Statistics on snow and wind loads for 35 years (1885… 1930) were taken into account. The following structural variants were analysed.

1.The steel trusses are made of steel St0 with the allowable stress $[\sigma]$ = 1200 kgf/cm^2 and the statistical characteristics $\overline{\sigma}$ = 2420 kgf/cm^2 and $\hat{\sigma}$ =150 kgf/cm^2;

2.The steel trusses are made of steel St3 with the allowable stress $[\sigma]$ = 1400 kgf/cm^2 and the statistical characteristics $\overline{\sigma}$ = 2700 kgf/cm^2 and $\hat{\sigma}$ =148 kgf/cm^2;

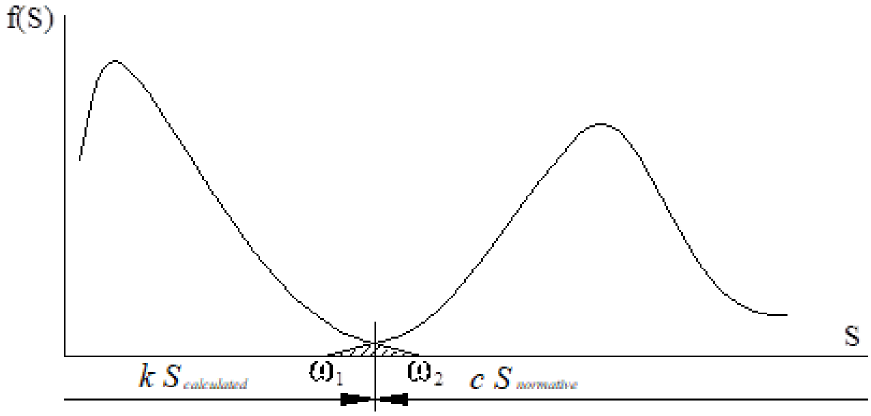


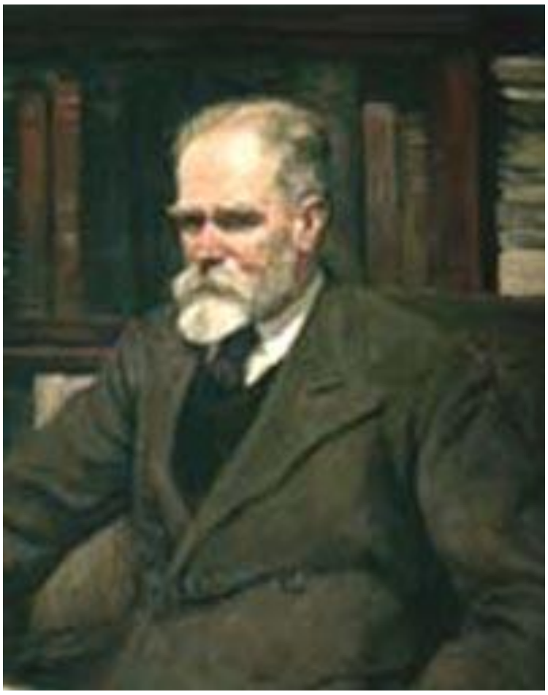
Fig. 2.1. The condition of non-destructiveness

For the 1st variant trusses, the areas of the tails of the curves (*Fig. 2.1*) are $\omega_1 = 2{,}36 \cdot 10^{-4}$; $\omega_2 = 2{,}3 \cdot 10^{-4}$, which resulted in a guarantee of non-destructiveness G =1 – 5,5·10^{-8}. For the 2nd variant, $\omega_1 = 2{,}5 \cdot 10^{-4}$; $\omega_2 = 3 \cdot 10^{-4}$ G = 1 – 8,5·10^{-8} values were obtained accordingly. For steel trusses which were designed in accordance with the 1934 code, a guarantee of non-destructiveness was very high.

2.4. Scientific feat N.S. Streletsky

The value of the guarantee of non-destructiveness for metal structures was calculated under the rules of 1934 on the basis of load distribution curves and yield strength. The guarantee of non-destructiveness had values that were far within the asymptotic part of the non-destructiveness guarantee curve. Assuming the output data to be correct, it could be concluded that there are realistic considerations regarding the possibility of increasing the loads allowed for metal structures in relation to the 1934 code. Of course, the available research data were not exhaustive and served only as primary material, which revealed a general picture of the phenomenon. However, this material showed that the values of the guarantees of non-vulnerability under the codes of 1934 turned out to be quite close to one. This circumstance should have remained in force and at change of initial data, as on asymptotic part of a curve even considerable changes of argument affect rather little.

These considerations were substantiated by N.S. Streletsky and were taken into account by the Narkombud of the USSR in the midst of the Second World War in 1942. As a result, the allowable stresses for steel structures were increased by 2 kGs / mm^2 and were accepted for structures made of steel Oc equal to 14 kGs / mm^2 (by 15%!) and for structures made of steel St3 equal to 16 kGs / mm^2 (by 12.5%!) while maintaining without changing the mechanical characteristics of steels (normalized minimum yield strength for steel Oc 19 kGs / mm^2 and for steel St3 22 kGs / mm^2).



N.S. Streletsky

These changes reduced the reserve factor from 1,58 to 1,36 (under the main loads). Despite the quite low value of the reserve factor, which was record-breaking, such increasing stresses were obviously feasible, as it was shown by the corresponding analysis. The value of the guarantee of non-destructiveness at a reserve factor of 1,36 was according to various estimates $G = 1 - 6 \cdot 10^{-7}$; 1- 3^{-6}; $1\text{-}5 \cdot 10^{-6}$. All values of *G* remained quite close to unity, they are less only for one millionth of a unit for lightweight metal structures.

2.5. Life path of N.S. Streletsky

Nikolay Stanislavovych Streletsky (1885 - 1967) was a master of domestic metal structures and the founder of the science of reliability in construction. He is the author of the first textbook on metal structures in the USSR [4], which was repeatedly reviewed and was used during 50 years by several generations of construction engineers, including the authors of this manual. Students and future builders should know about the life path of such eminent person [2].

1885 p. - N.S. Streletsky was born into the family of a military fortification engineer, who built defence structures in the western and northern outposts of Russia.

1904 p. - he graduated from the Vladivostok Gymnasium.

1904 - 1911 - he studied at the St. Petersburg Institute of Railway Engineers.

1911 - 1913 – he had a business trip abroad to Germany, studied at Charlottenburg University of Technology.

Since 1914 - he had been designing and testing steel bridges.

1917 - 1928 – had worked as a teacher at the Moscow Institute of Railway Transport Engineers.

1927 - 1937 – he worked and headed of GIS (later CNIPS).

1931 – he was a corresponding member of the USSR Academy of Sciences.

1932 - 1967 - he was a head of the department of metal structures MISI

1941 - 1943 - he was a head of the MISS in the evacuation in Novosibirsk.

1966 - Hero of Socialist Labor.

1967 - N.S. Streletsky died and was buried in the New Maiden Cemetery in Moscow

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Control questions

1. The structure of the reserve factor.
2. Statistical nature of the reserve factor.
3. The essence and definition of the guarantee of non-destructiveness.
4. How was it possible to increase the allowable stresses for steel St3 from 14 kGs/mm^2 to 16 kGs/mm^2?

LECTURE 3. FUNDAMENTALS OF THE METHOD OF LIMIT STATES

3.1. Method Implementation
3.2. Definition of limit states
3.3. Classification of limit states
3.4. Limit inequality for limit states of the first group
3.5. Limit inequality for limit states of the second group
3.6. Limit state method input effect

3.1. Method Implementation

The significant shortcomings of the allowable stress method presented in previous lectures were noticed by Soviet engineers and scientists quite a long time ago. As a result, at the highest state level, the task to develop a unified calculation method and improve the system of safety factors was set.

Even before the end of the Great Patriotic War (The Eastern Front of World War II), in 1943, under the Technical Council of the People's Construction Commissariat of the USSR, a commission was organized for the unification of methods for calculating building structures, which later came under the jurisdiction of the Central Research Institute of Industrial Structures (TsNIPS). V.A. Baldin (metal structures), A.A. Gvozdev (reinforced concrete structures and structural mechanics), I.I. Goldenblat (theory of elasticity), Y.M. Ivanov, V.M. Kochenov (wooden structures), V.M. Keldysh (chairman of the commission, reinforced concrete structures), L.I. Onishchik (stone structures). M.S. Strelkovy (metal structures and statistical calculation methods), K.E. Tal (reinforced concrete structures). In 1944, the commission made proposals to replace the unified safety factor with a system of differentiated coefficients of variability. On the basis of these proposals and subsequent studies and discussions, a little later in 1955, the calculation of all structures was transferred to a more modern and reasonable method of limit states.

The first codes for the design of steel structures for limit states are NTU 121-55 "Codes and technical conditions for the design of steel structures" and SNiP II-B. 1-54 “Construction codes and rules. Loads and impacts". Further, both documents were repeatedly revised and republished on a permanent general basis of the limit state method. In recent years, Ukraine has developed (with the participation of the author of this lecture notes) significantly changed the load standards (DBN V.1.2-2:2006 " Loads and loading. Codes of projection") and the design standards for steel structures (DBN V.2.6-163:2010 "Steel structures").

3.2. Definition of limit states

The necessity of transition to the method of limit states can be argued as follows. Raise a question: what is the main task for an engineer in the building or structure design? At first sight, the answer is obvious - to ensure the structures bearing capacity. This question was answered with a certain inaccuracy by the allowable stress method. But such houses have a number of serious disadvantages. For instance, such houses are very durable, but it is inconvenient and uncomfortable to live there, as the walls sag and crack. The premises of such houses have poor sound insulation, and it is also cold due to insufficient thermal insulation. An example of such buildings can be the Poltava Local Lore Museum. Talking about industrial buildings were designed in this way, their disadvantages are obvious. Is it impossible to work in the powerful industrial workshops (Poltava diamond plant with fluid roofs, Kremenchug Amper plant with vibrating ceilings)? Obviously, the main requirement for buildings, structures and their constructions is normal operation.

Consequently, the purpose of the building structures calculation is to ***providing the necessary operating conditions*** for a building or structure and their sufficient strength at the lowest present value, i.e.

Limit states are states in which structures cease to meet the requirements for construction and operation.

Note that the limit states have different significance. In particular, when one of them occurs, only interference with normal operation occurs. Here are some examples:

• The deflection of the crane beam, exceeding 1/400 of the span, makes it difficult to move the overhead crane and requires additional energy consumption.

• Deflection of floor beams within 1/150…1/200 causes discomfort for the working staff.

• Significant movement of the building frame can damage the wall protection.

• Damage to the anti-corrosion coating contributes to the development of steel corrosion and reduces the durability of structures.

Another situation develops if the limit state makes the structure completely unacceptable for operation. The typical more dangerous examples are presented below:

• A transverse crack in the tensioned chord of the beam, which can lead to its failure.

• Loss of firmness of the compressed brace, which leads to the fall of the truss.

• Formation of a plasticity hinge in a single-span beam, turning it into a replaceable system.

3.3. Classification of limit states

It is important to pay attention to the difference in the terminology of limit states in Ukraine and Europe.

Limit state design requires the structure to satisfy two principal criteria: the ultimate limit state (ULS) and the serviceability limit state (SLS).

Any design process involves a number of assumptions. The loads to which a structure will be subjected must be estimated, sizes of members to check must be chosen and design criteria must be selected. All engineering design criteria have a common goal: that of ensuring a safe structure and ensuring the functionality of the structure.

Hereby, limit states in Ukrainian building codes are divided into two groups:

The limit states of the first group lead to the exhaustion of the bearing capacity of structures and (or) cause their complete unsuitability for further operation. The requirements of this group are absolute, since if they are violated, the structure, in particular the metal one, ceases to exist, turning into scrap metal. The states of this group are usually called "final limit states".

The limiting states of the second group cause unsuitability of structures for normal operation or reduce their durability as a result of significant deformations. Operation is considered normal if it is carried out in accordance with technological or living conditions without restrictions stipulated by the standards or the design assignment. The requirements of this group are less categorical, since they concern only the mode of operation of structures that can be controlled. The states of this group are usually called "limiting deformation states".

The limit states of the first group include:

• viscous, brittle, tedious or other nature of destruction caused by force impacts;

• destruction from the simultaneous action of force factors and adverse effects of the environment;

• general loss of form stability (for example, loss of stability of a beam or truss brace);

• loss of position stability (for example, destruction of a retaining wall or chimney, falling of the floating crane “Zakharia” in Kiev, *Fig. 3.1*);

• qualitative change of the configuration, transformation of the structure into a geometrically variable system;

• resonant oscillations (destruction of the Tacoma bridge, *Fig. 3.2*);

• conditions in which there is a need to stop operation due to excessive fluidity of the material, displacement in joints, creep, presence of cracks in metal structures.

The limit states of the second group include:

• movement of structures that create obstacles to the operation of the structure;

• excessive subsidence, angles of rotation of structures;

• fluctuations that violate the operation of equipment or sanitary and hygienic standards for working personnel;

• opening of cracks in reinforced concrete structures (in the first versions of the calculation standards, this referred to a separate third limit state, which was subsequently removed from the standards)

• other violations leading to temporary shutdown of operation and repair.

3.4. Limit inequality for limit states of the first group

Structural analysis should ensure that the limit state of this group occurs no more than once during the entire service life or does not occur at all. The fulfillment of this condition is ensured by satisfying the following inequality, called the "non-destructive condition":

$$N \leq \Phi\,, \tag{3.1}$$

where N is the greatest possible force (limiting) in the structural element for the entire period of operation (function of loads and impacts);

Φ is the lowest possible bearing capacity of an element, in other words - the ultimate force that the element can withstand (a function of the material properties and element dimensions).

It should be emphasized that such an approach to the calculation of structures with simultaneous consideration of the maximum forces and minimum resistance of the elements provides a sufficiently high level of structural reliability, as evidenced by many years of accident-free experience in the operation of building structures.

In the first codes for designing structures in terms of limit states, condition (3.1) was interpreted as follows:

$$\sum \alpha_i P_i^H n_i \leq m\Phi R^H k, \tag{3.2}$$

where α_i is coefficient of transition from the i-th load to the force in the element (in other words, the numerical impact);

P_i^H is normative load;

n_i is overload coefficient;

m is working conditions coefficient;

Φ is geometric characteristic of the section (area, modulus, etc.);

R^H is characteristic material strength;

k is coefficient of uniformity.

Let's explain the content of each of the limit inequality coefficients.

Normative values of loads P_i^H are the main loads characteristics, their maximum values, which do not correspond to the normal operation of the structure and are established by design standards. The overload coefficient n_i

takes into account the shift of loads and the possibility of exceeding the existing loads of standard values. Design load values are defined as:

$$P = P^H n. \tag{3.3}$$

Design loads are the largest loads possible during the life of the structure, which can lead to its bearing capacity depletion. The methodology for substantiating the standard values and overload factors is different for different loads. These issues are discussed in detail in the following chapters of this course.

The characteristic material strength R^H is the main strength of the material, established by the standards and controlled during the manufacture and material acceptance. For steel, the normative strength is equal to the insufficient minimum of the yield strength or tensile strength of steel according to the current GOSTs for the corresponding grades and classes of steel. The uniformity coefficient k takes into account the possibility of getting into the steel structure with a yield strength value below the missing minimum established by the standard, due to the selectivity of the steel rejection testing procedure (several test samples per steel batch of tens of tons). The design material strength is defined as

$$R = kR^H. \tag{3.4}$$

Therefore, the design resistance of a material, in particular steel, is the lowest possible value of the yield strength.

The coefficient of working conditions m takes into account the structures operation features that increase or reduce their work reliability.

The content and structure of limits inequality and the calculated coefficients are applied without significant changes to this day. Some additions and changes to the designation were made to the codes of SNiP in the 80s of the last century, and instead of formula (3.2), the following expression was written:

$$N = \gamma_n \sum_{i=1}^{m} F_{ni}\gamma_{fi}\psi_i\,\alpha_i \leq \frac{AR_n\gamma_c}{\gamma_m} = \Phi. \tag{3.5}$$

Where F_{ni} is normative load;

γ_{fi} is load safety coefficient similar to overload coefficient;

R_n is characteristic material strength;

γ_m is reliability coefficient for the materials, similar to the uniformity factor (reciprocal);

γ_c is coefficient of working conditions;

ψ_i is conjugation coefficient, which is introduced when considering several loads together; this coefficient was implicit in previous editions of the standards;

γ_n is reliability coefficient for the purpose, taking into account the different capital and responsibility of construction projects;

A is geometric characteristic of the section;

α_i is influence number.

Design loads are still determined, $F = \gamma_{fi} F_n$, and the calculated strength is slightly different: $R = R^n/\gamma_m$.

In the national codes DBN V.1.2-2:2006 "Loads and loading" [7], in the development of which the author of the manual took part, the definition of loads in general terms (3.5) has been changed:

• instead of the normative load, the characteristic load value F_0 was introduced, the definition of which differs significantly for some loads (atmospheric, crane loads);

• for the calculation of the limit states of the first group, the following are used: limits design value $F_m = \gamma_{fm} F_0$, quasi-constant design value $F_p = \gamma_{fp} F_0$ (taking into account long-term processes, creep, etc.), cyclic design value $F_c = \gamma_{fc} F_0$ (for vibration loads).

3.5. Limit inequality for limit states of the second group

In the first codes editions for designing structures for limit states, this inequality looked like this:

$$\sum P_i^H \Delta_i \leq f, \tag{3.6}$$

where Δ_i is displacement or deformation from a single load;

f is the maximum allowable amount of displacement or deformation.

Inequality (3.6) in subsequent editions of the codes slightly changed its form:

$$\gamma \sum_{i=1}^{n} F_{ni}\, \psi_i \delta_i \leq \Delta, \tag{3.7}$$

where δ_i is the number of impact, that is, the displacement of the structure from a single load;

Δ is the limiting value of the displacement, which determines the possibility of normal operation and is established by the standards or the design assignment.

In the codes of DBN V.1.2-2:2006 " Loads and loading " [7], instead of the normative values of loads, inequality (3.7) is substituted with operational calculated loads values $F_e = \gamma_{fe} F_0$.

3.6. Limit state method input effect

The limit state method immediately showed its significant advantages over the allowable stress method.

1. All components of limits inequalities have a clear physical content that characterizes a possible change in loads, materials or operative conditions. In this

regard, possible to conclude that in the main inequality there is ***no longer a coefficient of ignorance.***

2. The method of limit states essentially divides the single safety factor into its component parts, replaces it with several design coefficients. The resulting ***reliability coefficient became variable*** due to the effect of conjugation of the indicated coefficients, depending on the loads values, the purpose of the structure, the materials strength and the circumstances of the structure's operation.

Let's illustrate this by slightly changing formula (3.2):

$$\sigma = \frac{\sum P^H}{\Phi} \leq \frac{R^H}{\frac{n}{mk}} = \frac{R^H}{K} = [\sigma]\,. \tag{3.8}$$

It can be seen that a single safety factor can be represented as a function of 3 arguments:

$$K = \frac{n}{mk}\,. \tag{3.9}$$

Taking into account that for most cases the coefficient of operating conditions m =1.0, and for low-carbon steel in the codes of the 50s, the uniformity coefficient was equal to k = 0.9. Then the safety factor according to formula (3.9) will be equal to K = 1.22 with an overload factor n = 1.1 (the predominant action of a constant load), K = 1.33 with an overload factor n = 1.2, K = 1.44 with overload factor n =1.3 (dominant effect of variable loads).

3. Since the introduced calculation coefficients have a clear content and are independent, the possibility of a separate study and each of them refinement has opened up, mainly on the basis of statistical and probabilistic methods. This gave a powerful impetus to research in this area, and a number of scientists obtained important scientific results, defended dissertations, and significantly developed the theory of structural analysis:

- B.N. Koshutin, S.F. Pichugin, A.V. Figarovski, V.N. Val, Y.S. Kunin, B.Y. Uvarov, N.Ya. Kuzin, A.T. Yakovenko (MISI);
- V.A. Otstavnov, M.F. Barshtein, A.A. Bat (TsNIISK);
- Y.A. Zdanevich (GIBI, Dnipro).

4. Already the first experience of introducing a new method provided ***material savings*** in structures, in particular for metal structures in the range of 3 ... 10% (*Tables 3.1* and *3.2*). It was found that structural elements that are predominantly affected by constant overloads with minimal overload factors turned out to be lighter. As a result, the greatest savings were obtained for trussed and subrafter trusses, while the crane girders remained almost unchanged. The calculation of columns as part of transverse frames has led to a slight increase in the cross sections of the upper parts, which is explained by the development of

new standards for the calculation of off-center elements. At the same time, a decrease in the sections of the lower parts of the columns was noted. In general, the columns either remained unchanged or even became somewhat heavier *(Table 3.1)*. The designs of blast furnaces became 3.8…12.6% lighter *(Table 3.2)*. In subsequent years, the design coefficients of the methodology gradually decreased, as a result of which the efficiency of structures increased.

Table 3.1

Steel savings obtained in the industrial buildings structures calculation for limit states

Name structures	Steel savings in structures, %	
	Columns	Trusses
Open-hearth shop	-1,5	10,0
Blooming	-2,4	11
Rolling shop	-1,7	10,5
GRES building	2,6	3,2
Machine building shop	0	10,0
Metal structures shop	0	4,0
Part of the equipment	0	8,0

Table 3.2.

Steel savings in the design of a blast furnace in the calculation by limit states

Blast Furnace Structures	Steel savings, %
Casing	3,8
Koper	8,6
Top platform	11,2
Mine and forge columns	12,6

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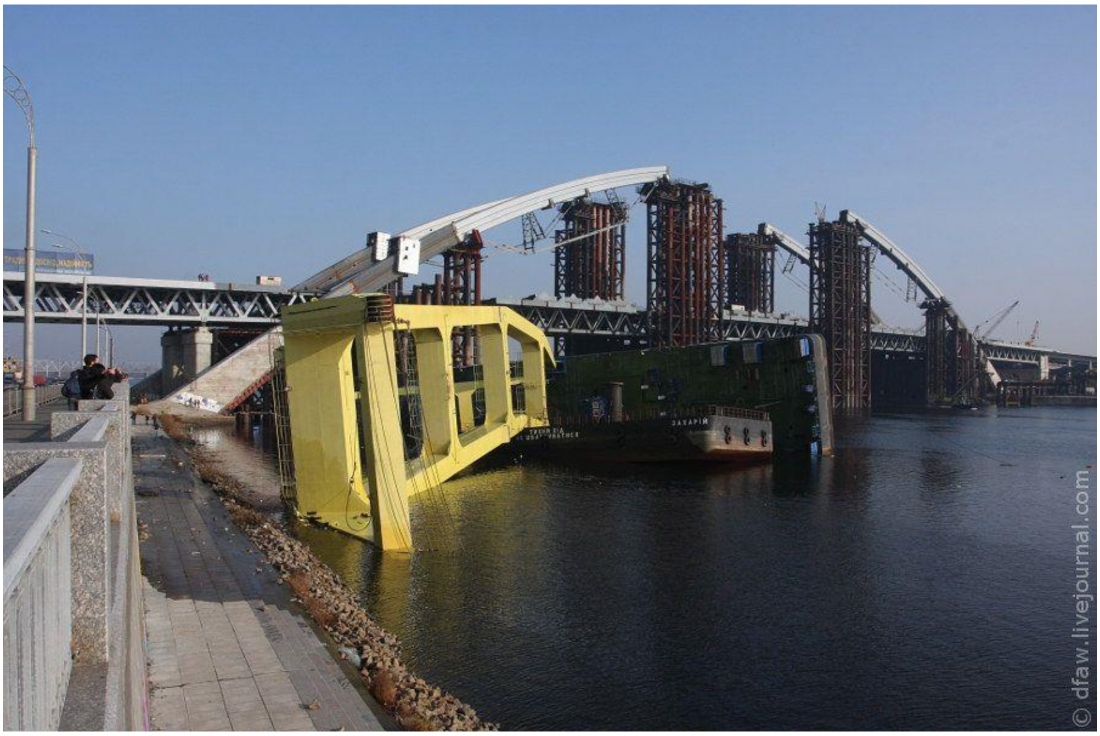
8. Control questions

1. Give a definition and examples of the limit states of building structures.
2. What refers to the limit states of the first group?
3. What is related to the limit states of the second group?
4. What form does the limit inequality have in the codes of SNiP for the first group of limit states?
5. What changes have been made to the limiting non-equation by the DBN codes?
6. What form does the limit inequality have for the second group of limit states?
7. Did the introduction of the limit state method have an economic effect?

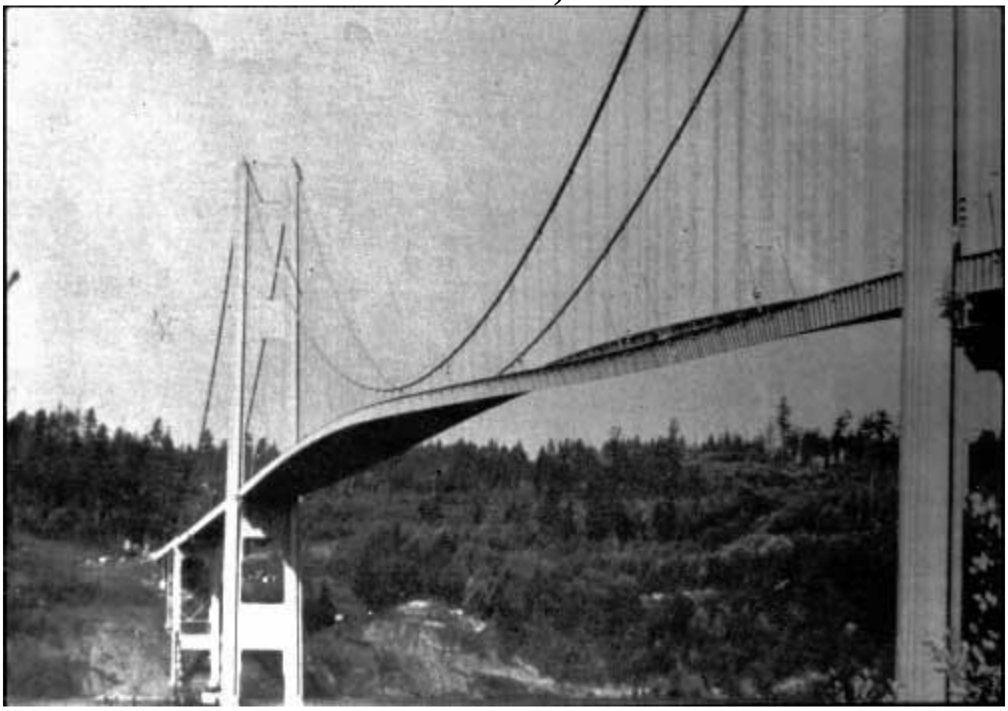


a).

b)

Fig. 3.1. Floating 600-ton crane "Zachariy"
a - the crane after the collapce (November 18, 2011)
b - crane in operation

a)



b)

Fig. 3.2. Destruction of the Tacoma Bridge:
a - the beginning of loosening; b - the nature of the bridge destruction

LECTURE 4. SPECIFIED AND DESIGN MATERIAL STRENGTH. MATERIAL SAFETY FACTOR

4.1. Random variables distribution curves

Insofar as the mechanical characteristics of materials and loads on structures have a random spreading, their study requires the use of probabilistic methods. Therefore, let’s begin consideration of the raised question in this chapter with a presentation of some thesis of the probability theory and mathematical statistics concerning random variables.

Key definitions. *A random value (RV)* is a variable, as a result of the test it can take one or another value, and it is not known in advance which one.

Examples of random variables:

- geometric dimensions of structural elements;
- actual value of structural material strength;
- mechanical properties of materials.
- structural loads.

Designations: $\tilde{x}$ is a random value; x is its possible value.

Event probability ***A or RV*** called a numerical measure of the degree of objective possibility of this event or *RV*, notation *P(A)*, *P(x)*.

The concept of probability is closely related to the concept of *frequency*.

If in a series with *n* tests, event *A* occurs in *m* cases, the frequency is defined as

$$P^*(A) = \frac{m}{n}. \quad (4.1)$$

If the number of tests increases unlimitedly, the frequency tends asymptotically to probability, according to Bernoulli's theorem

$$P^*(A) \to P(A),\ n \to \infty. \quad (4.2)$$

Distribution curves RV. To characterize the *RV* probability, a function is introduced

$$F(x) = P(\tilde{x} < x). \quad (4.3)$$

This function is equal to the probability that the random variable $\tilde{x}$ will be less than some of its values *x*; this function is called an integral distribution function of a random variable, or simply a ***distribution function***. In the case of a positive continuous *RV*, the distribution function has the character that is illustrated in *Fig. 4.1*.

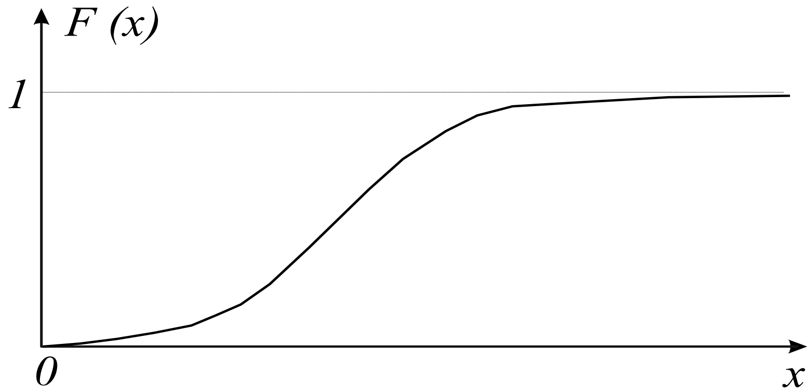


Fig. 4.1. Random distribution function

Derivative function *F(x)*

$$f(x) = \frac{dF(x)}{dx} \tag{4.4}$$

is called the differential distribution function or ***the distribution density*** of a random variable $\tilde{x}$. The function graph $f(x)$ is called ***the distribution curve*** *(Fig. 4.2)*.

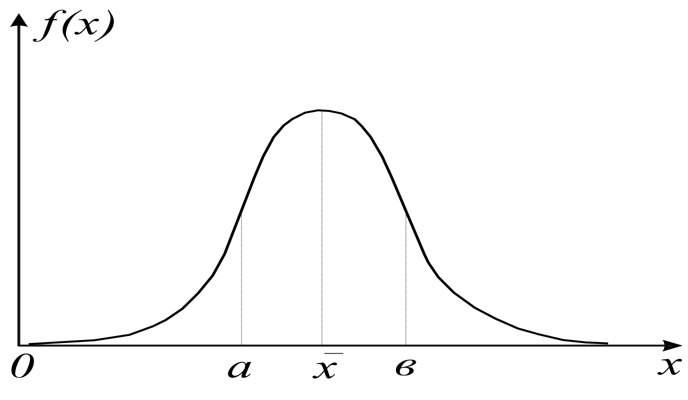


Fig. 4.2. Random distribution curve

The following relationships are important based on the *RV* distribution curve.

1. Transition from a differential function $f(x)$ to an integral distribution function *RV* $F(x)$:

$$F(x = a) = F(\tilde{x} < [x = a]) = \int_{-\infty}^{a} f(x)dx; \qquad (4.5)$$

2. Determining the probability of falling *RV* into the interval

$$F(a < \tilde{x} < b) = \int_{a}^{b} f(x)dx; \qquad (4.6)$$

3. The normalization condition, according to which the area under the distribution curve is equal to unity

$$\int_{-\infty}^{\infty} f(x)dx = 1.$$

4.2. Numerical characteristics of the random variables distribution

Mathematical expectation

$$\bar{X} = \int_{-\infty}^{\infty} xf(x)dx. \qquad (4.7)$$

Mathematical expectation determines *the distribution position* on the abscissa axis, geometrically it is interpreted as the center of gravity of the area bounded by the distribution curve and the abscissa axis (*Fig. 4.3*).

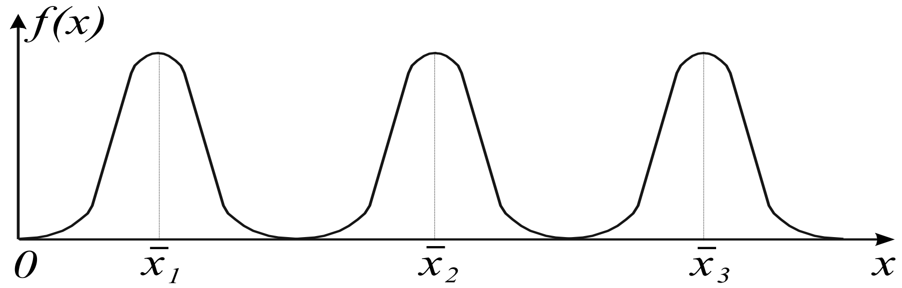


Fig. 4.3. Distribution curves with different mathematical expectation: $(\bar{x}_3 > \bar{x}_2 > \bar{x}_1)$

Dispersion, standart, variation coefficient

Dispersion is the mathematical expectation of the squared deviation *RV* $\tilde{x}$ from its center $\bar{x}$.

$$\hat{x} = \int_{-\infty}^{\infty} (x - \bar{x})^2 f(x)dx. \qquad (4.8)$$

Geometrically, the dispersion can be considered as the central moment of inertia of the area bounded by the distribution curve.

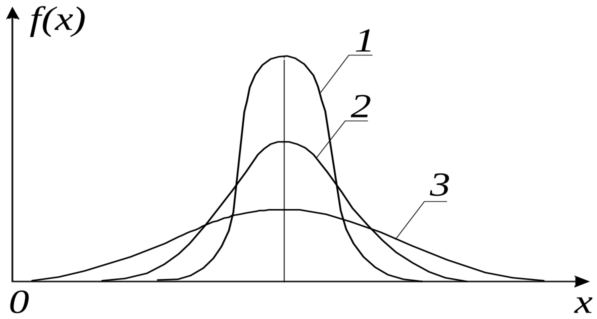


Fig. 4.4. Distribution curves with different standards: $(\hat{x}_1 < \hat{x}_2 < \hat{x}_3)$

The standard deviation (standard) $\hat{x}$ and *the variation coefficient V* characterize the values scatter of the random variable (*Fig. 4.4*):

$$\hat{x} = \sqrt{\hat{x}}; \;\; V = \frac{\hat{x}}{\bar{x}}. \tag{4.9}$$

The asymmetry coefficient *A_x* determines *the slanting* distribution of a random variable (*Fig. 4.5, a*):

$$\mathrm{A}_x = \frac{\mu_3}{\hat{x}^3}, \tag{4.10}$$

where μ_3 is the central moment of the third order, it is equal to

$$\mu_3(x) = \int_{-\infty}^{\infty} (x - \bar{x})^3 f(x) dx.$$

Kurtosis *E_x* estimates the pointedness (flatness) of the distribution of a random variable (*Fig. 5.5, b*):

$$E_x = \frac{\mu_4}{\hat{x}^4} - 3. \tag{4.11}$$

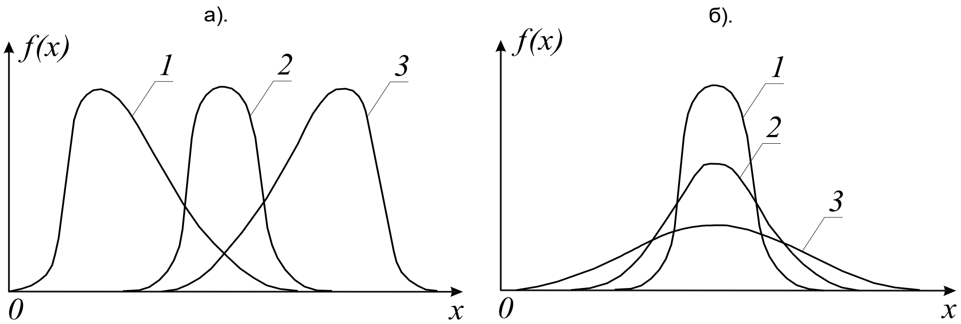


Fig. 4.5. Distribution curve options:
а) with different asymmetries: $A_1 > 0$, $A_2 \approx 0$, $A_3 < 0$;
б) with different kurtosis: $E_1 > 0$, $E_2 \approx 0$, $E_3 < 0$

4.3. The normal random variables distribution law

The density of the normal distribution (Gaussian) is described by the following expression:

$$f(x) = \frac{1}{\hat{X}\sqrt{2\pi}} exp\left[-\frac{(X-\bar{X})^2}{2\hat{X}^2}\right], \tag{4.12}$$

where X is a random argument; $\bar{X}$ and $\hat{X}$, respectively, the expected value and standard (standard deviation) of the argument X.

This is a symmetrical bell-shaped distribution defined by two parameters: $\bar{x}$ and $\hat{x}$ *(Fig. 4.6).*

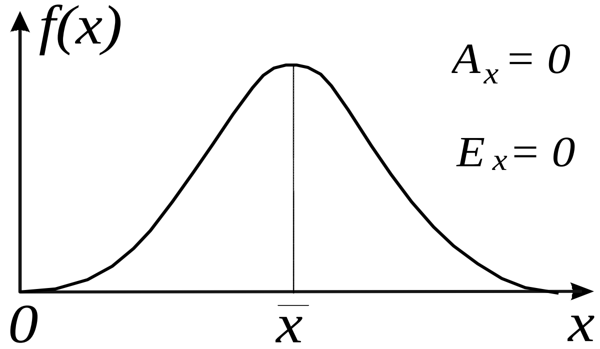


Fig. 4.6. Normal distribution of a random variable

This is a more common law in theory and practice, presented in the form of tables given in any manual on probability theory. This is due to its simplicity, theoretical validity (it is followed by the sum of independent RV with any distributions under the condition of increasing in the number of these RV), common in practice: to assess the errors in experiments on measurement accuracy, manufacturing quality and etc.

The ordinates of the normalized normal curve at $\bar{x} = 0$ and $\hat{x} = 1$

$$\varphi(x) = \frac{1}{\sqrt{2\pi}} e^{-0,5x^2} \tag{4.13}$$

are given in the table of paragraph 5-1 of the manual [1].

The normal distribution function is determined by integrating the density (4.13) and can be easily calculated using the tabulated Laplace functions *(Table paragraph 5-2 of the manual [1])*:

$$F(x) = \int_{-\infty}^{x} f(x)dx = \frac{1}{\hat{x}\sqrt{2\pi}} \cdot \int_{-\infty}^{x} e^{-\frac{(x-\bar{x})^2}{2\hat{x}^2}} dx = 0,5 \pm \Phi\left(\frac{x-\bar{x}}{\hat{x}}\right). \tag{4.14}$$

The plus sign corresponds to the positive value of the normalized deviation, thus the minus sign is to the negative value.

$$F(a < x < b) = \Phi\left(\frac{b-\bar{x}}{\hat{x}}\right) - \Phi\left(\frac{a-\bar{x}}{\hat{x}}\right). \qquad (4.15)$$

Using the Laplace function, the probability of falling into the following normalized intervals is easily determined:

- $\bar{x} \pm \hat{x} = 2 \cdot \Phi(1) = 0{,}34 \cdot 2 = 0{,}68;$
- $\bar{x} \pm 2 \cdot \hat{x} = 2 \cdot \Phi(2) = 0{,}477 \cdot 2 = 0{,}954;$
- $\bar{x} \pm 3 \cdot \hat{x} = 2 \cdot \Phi(3) = 0{,}49865 \cdot 2 = 0{,}9973.$

Thus, the output of a random variable out of limits has a probability of *0.27%*, that means, it is practically impossible (the "three sigma" rule).

4.4. Statistical data of the steel yield strength

As is well known, the process of steel smelting is rather complicated and not ideally controlled (high temperature, time of the smelting process, the content of alloying impurities, etc.). Subsequently, during rolling, the metal is compressed, the grains are crushed and their different orientation along and across the rolled product affects the mechanical properties of the metal. The properties of the steel are also influenced by the rolling temperature and subsequent cooling. In addition, with the rolled metal thickness increasing, the metal mechanical characteristics decrease. In the existence of such numerous factors affecting the steel strength, it is quite natural that the strength indicators have a certain statistical dispersion. A visual representation of the steel quality indicators variability is provided by statistical distribution curves of various steel characteristics.

The research results in the 30s - 40s of the last century. In the prewar and first postwar years, on the initiative of N.S. Streletsky, for the first time, large-scale statistical studies of the mechanical steels characteristics were launched, especially the most important parameter of the method for calculating metal structures - the steel yield strength [2].

In particular, the distribution curve of the yield strength for steel St3, according to V.V. Kuraev, obtained on the basis of 704 pre-war exams (1937-38), is shown in *Fig. 4.7*.

As seen in *Fig. 4.7*, the experimental polygon is quite close in shape to the normal distribution, it has a center (mathematical expectation)

$\overline{\sigma}_y = 270 \pm 14{,}8$ MPa. The normative minimum value of the yield strength 220 MPa lies from the center of the curve by 3,4 standards. If to cut the experimental curve off at the abscissa of 220 MPa, it gets to the ordinate of 0,00093, which corresponds to the abscissa of the Gaussian curve at a distance of 3,48 standards from the center. Replacing the cut off part of the experimental curve with the Gaussian distribution tail, we obtain its area equal to $2{,}5 \cdot 10^{-4}$.

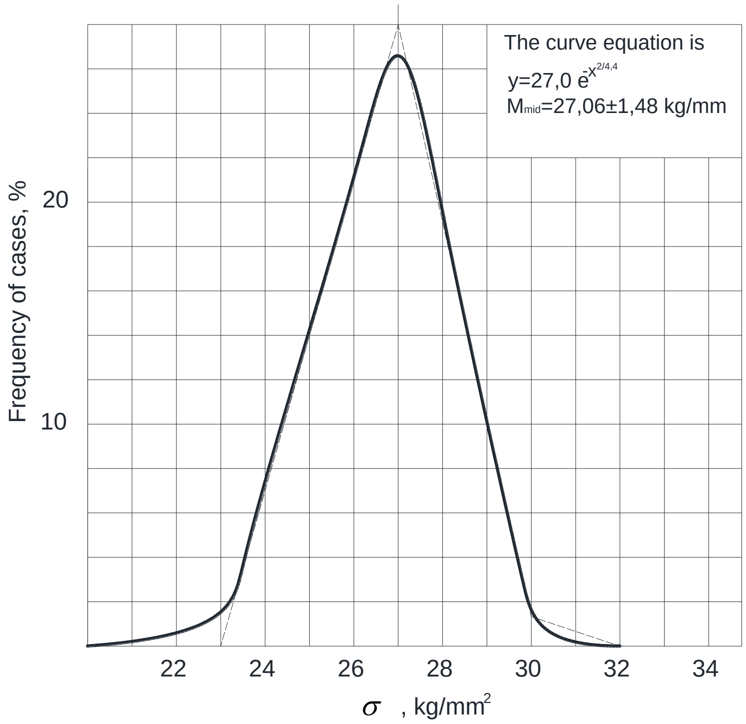


Fig. 4.7. Statistical curves of the yield strength distribution of steel grade St3

The materials published in 1941 by engineer V.V. Kuraev show that the yield strength of St3 steel was quite stable in all plants in the pre-war period, on average it was characterized by an average value of $\overline{\sigma}_y = 260 \pm 14$ MPa.

However, subsequent studies of the TsNIPS in the 40s of the last century (by the engineer Oyher) established that during the war the properties of St3 steel in relation to the yield strength changed quite noticeably; steel turned out to be much less homogeneity and at the same time more rigid *(Fig. 4.8).*

Average researches results of engineer Oyher give the expression $\overline{\sigma}_y = 300 \pm 25$ MPa for the yield strength of St3 steel (based on 6800 studies). Despite these changes, with new characteristics, just like with old ones, the normative yield strength lags behind its average value by more than three standards.

Also, there are interesting the results of a statistical study of the mechanical properties of steel St0s, which is the result of the conditioned steel

St3 rejection. This substandard steel was found to be a very in homogeneous material, illustrated by the experimental curves in *Fig. 4.9*.

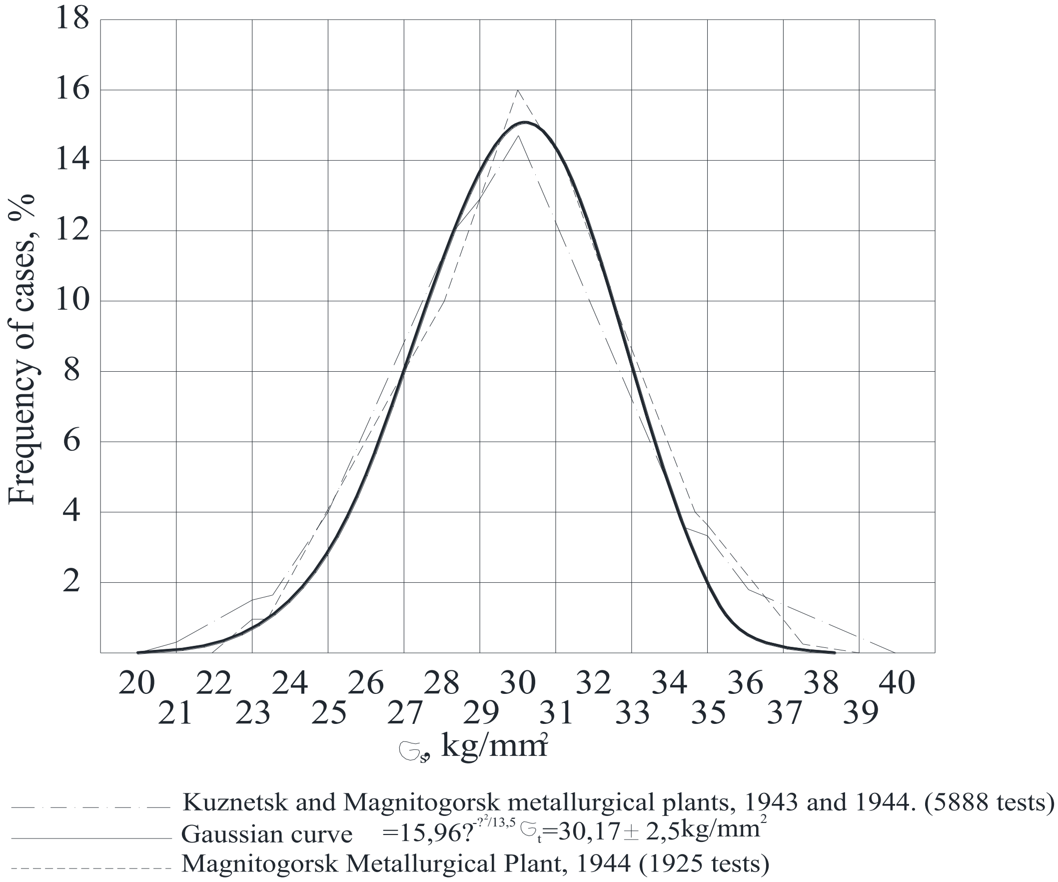


Fig.4.8. Steel yield distribution curve of steel St.3

A characteristic feature of the steel curves St0s from different plants, as well as the average curve for all plants, is their negative asymmetry, directed towards increased yield strength values, which indicates the heterogeneity of steelmaking conditions. The modal value of the steel yield strength St0s is quite high (280 MPa), and in 97 cases out of 100 it exceeds the standard minimum of conditioned steel St3 (220 MPa). This indicates that the main criterion for the St3 steel rejection (and the resulting St0s steel as a maintenance of St3 steel) is not the yield strength, but elongation and tensile strength.

Modern statistics on the mechanical steel characteristics. The yield strength and other mechanical characteristics of modern steels have a statistical spread, which is also well described by the normal law *(Fig. 4.10)*. The main statistical characteristics of the yield strength (average value and variation coefficient) for common modifications of low-carbon steel grade St3 are given in *Tables 4.1* and *4.2* [3].

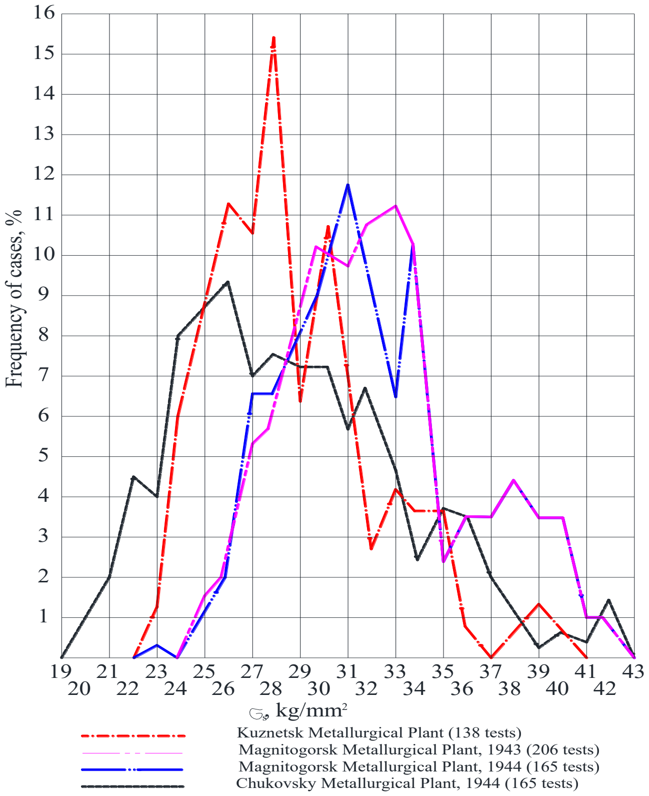


Fig. 4.9. The yield strength distribution curves of steel Os

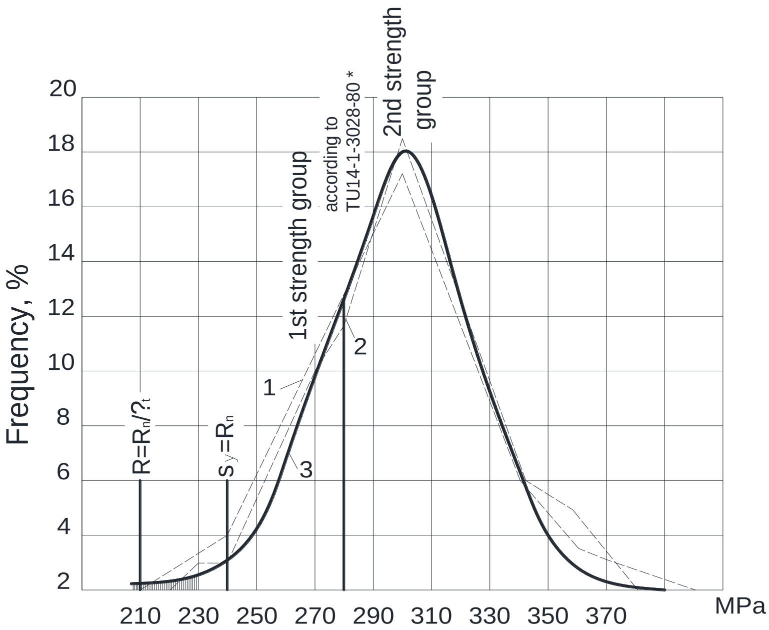


Fig. 4.10. Statistical curves of the yield strength distribution of steel grade St3:
1,2 - according to data from different factories; 3 - theoretical Gaussian curve

$$y = 15{,}96e^{-x^2/73{,}5}$$

Table 4.1.

Statistical steel supplied characteristics in accordance with GOST 380-71*

#	Sections	Steel grade	The average value of the yield strength $\bar{\sigma}_y$, МПа	The coefficient of variation V_m
1	rolled section, sheet thickness more than 3 mm	ВСт3пс3,ВСт3пс2 (VSt3ps3, VSt3ps2)	305	0,08
2		ВСт3пс3, ВСт3пс2 (VSt3ps3, VSt3ps2)	285	0,08
3		ВСт3пс3, ВСт3пс2 (VSt3ps3, VSt3ps2)	295	0,08
4		ВСт3пс3, ВСт3пс2 (VSt3ps3, VSt3ps2)	275	0,08
5	formed section, sheet thickness up to 5 mm	ВСт3Гсп2(VSt3Gsp2)	315	0,07
6		ВСт3пс2 (VSt3ps2)	295	0,08
7		ВСт3кп2 (VSt3cp2)	275	0,09
8	The same, more than 5 mm	Steel marks pp. 5,6,7	270	0,08

Table 4.2.

Statistical characteristics of steel strength, supplied in accordance with GOST 380-71*

Steel grade and type of rolled products	The thickness of a sheet or profile shelf, mm	Steel group I		Steel group II	
		The average value of the yield strength $\bar{\sigma}_y$,	The coefficient of variation V_m	Середнє значення межі текучості $\bar{\sigma}_y$, MPa	Коефіцієнт варіації V_m
ВСт3сп (VSt3sp), sheet	4 – 6	285	0,049	321	0,064
	8 – 10	283	0,050	315	0,060
	12 – 16	273	0,052	303	0,060
ВСт3пс (VSt3ps), sheet	4 – 6	280	0,055	313	0,058
	8 – 10	277	0,056	309	0,055
	12 – 16	270	0,053	298	0,055
ВСт3сп (VSt3sp), steel-cut	4 – 6	293	0,080	330	0,062
	8 – 10	292	0,080	325	0,058
	12 – 16	282	0,051	311	0,056
ВСт3пс (VSt3ps), steel-cut	4 – 6	284	0,050	318	0,062
	8 – 10	282	0,050	313	0,053
	12 – 16	280	0,051	308	0,053

4.5. Method of determination the coefficient homogeneity

In the first publishing of the design standards for limit states, the possibility of getting into the steel structure with a yield strength value below the insufficient minimum established by the standard was taken into account by ***the coefficient homogeneity***, which was taken as the ratio of the minimum probable yield strength to the standard resistance $k = R/R^{H} \leq 1{,}0$.

In subsequent editions of the standards is included the reciprocal value called ***the reliability coefficient for the material*** $\gamma_m = R_n/R \geq 1{,}0$.

These coefficients take into account:

• inevitable variability and dispersion of steel properties, estimated on the basis of statistical processing of steel factory tests numerous results (see distribution curves in the previous paragraph 4.2);

• deviations due to the dependencies inaccuracy between the material of construction and selective laboratory samples, according to the test data of which the standard values of the steel strength characteristics are established;

• possible deviations of the cross-sectional area from the nominal value within the tolerances for rolled profiles established by the standards (so as not to introduce additional coefficients).

The procedure for determining the homogeneity coefficient was developed by V.A. Baldin [5] by constructing a curve of possible conjugations of the yield strength values and the elements cross-sectional area. This curve replaces two scattering curves and is approximately described by a Gaussian distribution with the following parameters:

• scattering center (mathematical expectation)

$$a_0 = a_{1SC} a_{2SC}; \tag{4.16}$$

• standard deviation (standard)

$$S_0 = \sqrt{S_1^2 a_{2SC}^2 + S_2^2 a_{1SC}^2}, \tag{4.17}$$

where $a_{1SC} = \frac{\bar{\sigma}_y}{R_n}$; $a_{2SC} = \frac{\bar{A}}{A}$; $S_1 = \frac{\hat{\sigma}_y}{R_n}$; $S_2 = \frac{\hat{A}}{A}$ are scattering centers and square deviations in dimensionless form, respectively;

$\bar{\sigma}_y$ is average statistical value of steel yield strength;

$\bar{A}$ is average statistical value of the rolled cross-sectional area;

A is nominal value of the rolled cross-sectional area (by assortment);

$\hat{\sigma}_y$ is середнє квадратичне відхилення межі текучості сталі;

$\hat{A}$ is standard deviation for the rolled cross-sectional area.

The minimum value of the conditional curve ordinate is determined at a distance of three square deviations from the scattering center (the "three sigma" rule):

$$a_{0MIN} = a_0 - 3S_0. \quad (4.18)$$

Accordingly, the minimum probable value of the yield strength will be equal to:

$$\sigma_{y,MIN} = R_n a_{0MIN}. \quad (4.19)$$

The steel homogeneity coefficient is defined as the ratio of the minimum probable value of the steel yield strength to the standard resistance:

$$k = \frac{\sigma_{y,MIN}}{R_n}. \quad (4.20)$$

Some experts do not agree with this approach, believing that it is more logical to take this factor into account as a coefficient of working conditions.

4.6. Material safety factor calculation

According to the current standards, the probabilistic security (the probability of deviations downward) of the characteristic (previously standard) steel resistance should be equal to P = 0.95 *(Fig.4.10).* This value of coverage is estimated based on the Gaussian distribution as:

$$P = 0{,}5 + \Phi(\beta), \quad (4.21)$$

where β = 1,64 is the argument value corresponds to the Laplace function value Φ = 0,45.

The corresponding value of the characteristic steel resistance in terms of yield strength is equal to:

$$R_{yn} = \bar{\sigma}_y(1 - 1{,}64 v_m), \quad (4.22)$$

where $\overline{\sigma}_y$ is mathematical expectation (average value) of the yield steel strength; v_m is variation coefficient of the yield steel strength.

According to numerous statistical studies, for most building steels, the provision of the characteristic resistance beyond the yield strength R_{yn} is somewhat higher - 0.95 ... 0.99. This is a consequence of that R_{yn} is equal to an insufficient minimum in terms of supply standards adopted and in force for many years, which do not have a consistent statistical justification.

The design steel resistance is the minimum likely resistance of steel, which can be determined based on the statistical distribution of the yield strength *(Fig. 4.10)*, for example, based on the three sigmas rule. As indicated above, the transition from the normative to the design resistance was previously carried out in the form $R_y = kR_{yn}$ using homogeneity coefficients set equal to: k = 0,90 for low-carbon steel St3 and κ = 0,85 for low-alloy steel NL2. In modern standards for the steel structures design in the formula the safety factors for the material are introduced into the formula $R_y = R_{yn}/\gamma_m$ are introduced slightly lower values of reliability coefficient for the material, that is equal to γ_m= 1,025 for steels according to GOST 27772-88 (except for high-strength steels S590 and S590K); γ_m= 1,05 for steels according to GOST 380-71* and γ_m= 1,10 and 1,15 for steels supplied to other standards.

The values of characteristic and design steel resistances are given in the design standards for steel structures [5]. Based on the *Tables 4.1* and *4.2* of the statistical data for the steel yield strength St3, it is possible to determine the probabilistic availability of the indicated resistances and evaluate the compliance of the above values of the reliability coefficient for the materials.

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Control questions

1. How are the distribution curves of random variables constructed and what features do they have?
2. What are the numerical characteristics of the random variables' distribution?
3. What do the mechanical characteristics distributions of structural steel look like?
4. What are the probabilistic parameters (average value, standard, variation coefficient) of structural steel strength?
5. How was the coefficient homogeneity determined according to SNiP?
6. How is the reliability coefficient for the material justified?

LECTURE 5. COEFFICIENTS OF WORKING CONDITIONS

5.1. Coefficients of working conditions in design codes

The coefficient of working conditions γ_c (pre-designation m) must take into account all the features of the structure's work and operation that are not explicitly taken into account by other coefficients of the limit states method. Therefore, it is the most loaded by purpose and the least determined in content and coefficient value. The value $\gamma_c < 1$ takes into account the unfavorable working conditions of the structure, the value $\gamma_c > 1$ is favorable working conditions.

It can be assumed that this coefficient covers all the inaccuracies of the calculation model that arise due to its simplification and idealization, so that the calculation can be performed with reasonable labor costs. It is known that in any calculation simplifying provisions are introduced, in particular the main hypotheses of material resistance and structural mechanics. Here are examples of assumptions about the operation of the material (for example, Prandtl diagram), elements (hypothesis of flat sections), structures (pin joints of steel trusses), structural systems (simplified schemes of frames SIB (single-storey industrial buildings).

In order to take into account (or cover!) these errors and ensure the necessary reliability of the designed structure, ***the coefficient of working conditions*** is introduced. It can be considered that it has a statistical nature, in some cases it is studied and substantiated in detail. However, in most cases, its values are established speculatively based on the design experience and operation. The values of the coefficients of working conditions of steel structures in accordance with SNiP II-23-81 * "Steel structures" [1] are given in the Table. 5.1. Almost unchanged, this table was transferred to the Codes of Ukraine DBN B.2.6-163: 2010 "Steel structures. Standards of design, manufacture and installation "[2].

Here are explanations for some paragraphs in this table.

• Paragraph 1 of the table - less than one factor of working conditions for structures under the action of mostly constant load (with low load factor) is justified by the fact that these responsible structures can be destroyed by any minor accidental loading (for example, the destruction of a furniture store in Spain).

Table 5.1

Working conditions coefficients of steel structures [1]

№ n/n	*Design elements*	*Coefficients of working conditions γ_c*
1	*Solid cross-section beams and compressed elements of floor trusses under the halls of theaters, clubs, cinemas, under grandstands, under shops, bookstores and archives, etc. at the weight of the floors, equal to or more than the live loads*	0,90
2	*Columns of public houses and of water supports towers*	0,95
3	*Compressed main elements (except support) of a lattice of the folded T-section from corners of welded roof trusses and overlappings with flexibility $\lambda \geq 60$*	0,80
4	*Solid beams in the calculation of the total stability with $\varphi_b < 1{,}0$*	0,95
5	*Tightening, rods, braces, suspension brackets made of rolled steel*	0,90
6	*Elements of rod structures roofs and overlapping* *a) compressed (except for closed tubular sections) in the calculations for stability* *b) stretched in welded structures* *c) stretched, compressed, as well as butt plates in bolted structures bearing static load, in the calculation of strength*	 0,95 0,95 1,05
7	*Solid composite beams, columns, as well as butt joints that accept static load and are made with bolted joints, in the calculation of strength*	1,10
8	*Cross-sections of rolled and welded elements, as well as overlays at the joints made on bolts bearing static load, based on strength:* *a) solid beams and columns* *b) roof and overlap rod structures*	 1,10 1,05
9	*Compressed elements from single corners fastened by one shelf*	0,75
10	*Support plates made of steel up to 285 MPa, accepting static load, thickness, mm:* *a) up to 40* *b) more than 40 to 60* *c) more than 60 to 80*	 1,20 1,15 1,10

- Paragraph 3 of the table - compressed elements of considerable flexibility can easily acquire bends during transportation, installation and operation, which creates a risk of losing the trusses stability at all; the reduced coefficient is introduced after the appearance of real trusses accidents.
- Paragraph 5 of the table - takes into account the presence of threads in the elements when calculating the main cross section.

- Paragraphs 6, 7, 8 - coefficients, more than 1, reflect the operating conditions of bolted joints under static loads, more favorable compared to welded joints.
- Paragraph 9 of the table - center-compressed rods made of single corners, in the places of welded joints on one of the shelves transmit forces with eccentricity, which reduces their bearing capacity, which is taken into account by a factor $\gamma_c = 0.75$.

5.2. Structural correction as an assessment of the actual structures operation

Structural correction k_σ is the ratio of the actual stress or deflection from the selected load to the conditional design stress (or deflection) from the same load:

$$k_\sigma = \frac{\sigma_{exp}}{\sigma_{teor}}; k_f = \frac{f_{exp}}{f_{teor}}. \quad (5.1)$$

The magnitude of the structural correction is a characteristic of the approximation of the accepted design assumptions to the actual operating conditions of the structure and shows how the conditional design scheme is close to its actual scheme. This interpretation of the structural correction practically coincides with the above definition of the content of the working conditions coefficient of, so we can assume that ***the structural correction is an experimental assessment of the working conditions coefficient***.

Considering the structure of the marginal inequality for the limit states of the first group (lecture 3), the work conditions coefficient a quantity inverse to the constructive amendment, namely:

$$\gamma_c = 1/k_\sigma; k_\sigma = 1/\gamma_c. \quad (5.2)$$

Therefore, the structural correction, less 1, correspond to the values of the working conditions coefficient, more than one, which indicates the favorable features of the work and the possible reserves of load-bearing capacity of the structure. On the contrary, structural correction, more than 1, indicates an underestimation of the selected design models of actual stresses, which requires the introduction of working conditions coefficients, less 1 and the corresponding reinforcement of the structure.

The experience of field tests of real structures, in particular steel, convincingly shows that structural corretions in most cases are not equal to one. DSc. G.O. Shapiro, who conducted extensive research on the actual operation of steel structures of industrial plants in 1936… 1951 [3], as follows, developed an approximate structure of the structural orrectiont of steel structures:

$$k = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5\alpha_6\alpha_7\alpha_8, \tag{5.3}$$

where α_1 is general correction to the calculation scheme;

α_2 is correction to the geometric scheme;

α_3 is correction for structural elements;

α_4 is correction for cross sections of working elements;

α_5 is correction for spatial work;

α_6 is sediment correction and rotation of supports;

α_7 is load correction: its magnitude, relative position and change;

α_8 is correction for the rigidity of the nodes.

As you can see, this still incomplete list of components clearly shows the rather complex structure and content of the constructive amendment and, accordingly, the working conditions coefficient.

5.3. Structural correction of steel trusses.

In the pre-war years, TsNIPS and Ginstalmost tested 300 light rafter trusses [4], which allowed to build an experimental curve of the distribution of correction for stresses with numerical characteristics: average value $\bar{k} = 0{,}90$, standard $\hat{k} = 0{,}11$ *(Fig. 5.1)*. The experimental curve was successfully approximated by V.V. Kuraev by the normal distribution with the following equation:

$$y = 34{,}2e^{-\frac{x^2}{0{,}024}}. \tag{5.4}$$

To understand the nature of the structural correction of steel trusses, recall that in determining the effort, they are an idealized system with rectilinear rods that converge at one point (the center of the nodes) and are hinged. It is generally believed that the rods are made of perfectly elastic material, and the scheme itself is not deformed. Therefore, structural corrections for deflections are less than one *(Table 5.2)* due to the influence of the nodes' rigidity and the indistinguishability of the belts. These features to a lesser extent affect the magnitude of the axial forces in the rods: structural correction for stresses *k = 0,96* for welded trusses of light type *(Table 5.2)*. This trend is less appeared for heavy trusses with H-shaped cross-sections of rods, as well as for riveted trusses.

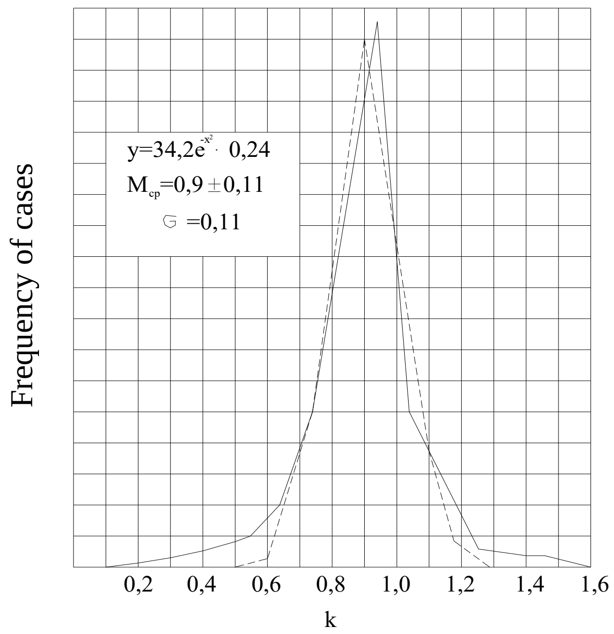

Fig.5.1. Curve of distribution of values of structural correction of light trusses

Table 5.2

Structural corrections of steel trusses of different types [3]

Truss type	***Structural correction***	
	By deflection	***By stresses in the middle of the farm in the belts***
Welded light type	*0,89*	*0,96*
Welded heavier type	*0,79*	*0,79*
Rivets of easy type	*0,76*	*0,68*
Rivets of heavier type	*0,59*	*0,55*

Obviously, the calculated model of trusses must correspond to their general static scheme (split, indivisible, freely supported, clamped). This position is clearly illustrated by the data in the *Table 5.3*.

5.4. Structural corrections of crane girders

During the above studies, G.A. Shapiro [3] evaluated the actual operation of some steel crane girders; the resulting structural correction are given in the *Table 5.4*. Quite unusual values of amendments in excess of one are the result of a rather complex operation of crane girders. G.O. Shapiro explains the data in the *Table 5.4* the presence of a biaxial stress state in the wall of the beams, the early appearance of plasticity, the influence of curvature of the beams' wall. In fact, the operation of crane girders is affected by many other factors that at the time of the tests (1936… 1951) were insufficiently studied. Thus, this is an example of how the values of the structural correction reflected the level of research on structures.

Table 5.3

Structural corrections to the steel long-span continuous truss
(with different calculation methods) [3]

Calculation scheme	***Structural correction***	
	by stresses in the middle of the truss in the lower and upper belts	***by the deflections in the middle of the farm***
Inseparable system	*0,96*	*0,89*
Split system	*0,71*	*0,75*
Freely supported on two supports truss	*0,88*	*0,86*

Table 5.4

Structural correction of crane girders by stresses
(according to the tests of G.A. Shapiro [3])

	Crane girders			
	1	**2**	**3**	**4**
Structural correction	*1,29*	*1,26…1,33*	*0,955…1,070*	*1,18…1,47*

During the transition from the method of allowable stresses to the method of limit states (50..60-ies of the 20th century) intensified full-scale experimental studies of the structure to clarify their actual work and identify design amendments. Some results for crane beams and crane girders obtained during this period are given in the *Table 5.5*. Not surprisingly, this table is not complete, because there are obvious difficulties in field research in existing shops, especially metallurgy with their extremely intensive operation and highly aggressive internal environment (high temperatures, gassiness, dynamic crane influences, etc.).

At the same time, most of the received structural corrections turned out to be less than 1, ie they had a character opposite to the data of G.A. Shapiro. It is possible that this indicated the available reserves in the studied beams and the possibility of some increase in the coefficient of working conditions.

According to the Moscow Institute of Civil Engineering (MICE), obtained a little later in the 60s of last century [6, 7], the structural correction for the deflections of welded crane girders is in the range of 0.85… 1.00.

Table 5.5

Structural corrections of crane beams and crane girders
(according to the results of tests of SPI Proektstalkonstruktsiya [5])

The name of the objects	***Structural corrections***			
	crane trusses and truss girders		***crane girders***	
	by the deflection	***by stress***	***by the deflection***	***by stress***
Open-hearth shop of Kuznetsk Metallurgical Plant	–	–	*1,07*	–
Open-hearth shop of Makeyevka Metallurgical Plant	–	–	*1,04*	*0,69 ...0,80*
Open-hearth shop of Dniprodzerzhynsk Metallurgical Plant	*0,72... 0,90*	–	*0,68... 0,72*	*0,87*
Steel plant of Elektrostal plant	*0,74*	–	–	–
Refining shop of the Monchegorsk plant "Severonickel"	*0,67... 1,0*	–	–	–
Mine yard of Krasny Oktyabr plant	–	–	*0,53*	*0,51*

5.5. Structural corrections of transverse frames and columns

The most complete data on constructive corrections of transverse frames was obtained by G.A. Shapiro during large-scale field tests of industrial buildings in the 30… 50 years of the last century [3]. Some of the results are given in the *Table 5.6*. Shop №1 - open-hearth, built in the early twentieth century, with a hinged connection of crossbars with columns and riveted steel structures. Shop №4 is also open-hearth, designed in the 1940s, with rigid transverse frames and lattice crossbars.

As can be seen from the table, the structural corrections for transverse displacements were very small compared to the calculation of a flat free-standing frame, which clearly indicates the significant closeness of this design model. Taking into account the spatial calculation model, the constructive amendment increases significantly, although it has not yet reached one, which indicates the incomplete perfection of the spatial model OVB 40..50 years.

Table 5.6

Structural correctionss OIB frames by transerve displacement [3]

№ shop	***Mark of the place of displacement measurement, m***	***Transverse braking of the bridge crane***		***Horizontal force at the marks of the bottom of the trusses or rail head***	
		Flat frame	***Spatial system***	***Flat frame***	***Spatial system***
Shop №1	*25,7*	*0,12*	*0,60*	*0,21*	*0,80*
	18,5	*0,16*	*0,80*	*0,15*	*0,58*
Shop №4	*15,2*	*0,09*	*1,00*	*0,08*	*0,51*
	12,4	*0,06*	*0,90*	*0,10*	*0,57*

As shown by field tests of steel columns OIB, theoretical and experimental values of normal forces differ slightly, structural corrections are close to one *(Table 5.7)*. Structural amendments of columns at times are much smaller due to their more complex nature (uncertainty of lateral forces of cranes, eccentric support of crane girders, complex design of elms and work platforms, etc.).

Table 5.7

Structural corrections of steel columns [3]

Column type		***Structural corrections***	
		by normal strength	***by bending moments***
Heavy (with folded I-beam branches)	*Extreme row*	*0,91…1,01*	*0,66…0,74*
	Middle row	*0,76…1,00*	*0,44…0,67*
Lightweight (with rolled corner branches)		*0,87…1,03*	*0,64…0,74*

5.6. Probabilistic estimation of the working conditions coefficient of stepped columns

The substantiation of the working conditions coefficients, in addition to the assessment of constructive amendments, may also be based on probabilistic studies and assessments of the structures’ reliability.

In particular, the peculiarity of the column designs is that they are subject to several random loads that have different probabilistic nature (constant, atmospheric, crane) *(Fig. 5.2).* Therefore, the assessment of the reliability of such structures is quite complex; it was obtained in studies conducted at the Department of CSWP PoltNTU for several years [8, 9, 10]. These studies allowed, in particular, to estimate the bearing capacity reserves of steel columns OIB, which were not taken into account by current regulations, and to propose a new working conditions coefficient.

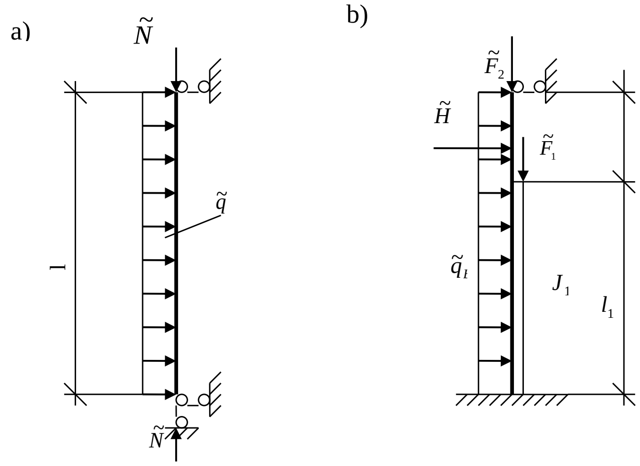

Fig. 5.2. Calculation schemes of columns OIB:
a - rack of constant cross section; b - stepped column

To obtain specific results, a number of characteristic stepped columns of industrial buildings designed according to the standards [1] were tested, some of the results are illustrated in *Table 5.8*, in which the following notations are accepted:

L is span of the transverse frame of the industrial building;

B is step of columns;

l_2 , l_1 are the lengths of the upper and lower parts of the column;

$Q_2(t)$, $Q_1(t)$ are the failure probability of the upper and lower parts of the column, respectively, with a service life of t = 50 years, determined approximate by the number of emissions $N_+(t)\leq 1$.

As can be seen from the *Table 5.8,* columns in a wide range of parameters were considered: at rigid and hinged connection of columns with crossbars, for frames with spans of 24… 36 m and steps of 6 and 12 m, with a warm and cold roof on the profiled flooring and reinforced concrete panels, with bridge cranes 30/5… 125/20 vehicles of 4K-6K and 7K modes, for snow and wind loads of I, II and III districts; all columns are selected by calculation without safety margin.

These tables show that the upper parts of stepped columns have a failure probability of the same order with columns of constant cross-section and rafters with heavy roofs, so we can talk about the approximate uniformity of this group of steel structures.

At the same time, systematically, both with rigid and hinged connection of crossbars with columns, the reliability of the lower parts was much higher than the upper. This is due to the application of more random loads to the bottom, in particular, the vertical crane load.

This position made it possible to expose the reliability reserves of the lower parts of the columns and from the condition $Q_1(t)\approx Q_2(t)$ to find a reduced cross-sectional area and the working conditions coefficient, which was equal to the tested columns γ_c =1,15…1,53. This coefficient, determined on the basis of the equivalence of parts criterion, can be recommended to use in the calculation formulas for solid and through the lower sections of stepped columns OIB, equipped with overhead cranes.

One of the parameters influencing this coefficient is the ratio of the longitudinal forces of the upper and lower parts of the columns. It is obvious that in order to differentiate the coefficient γ_c and detailing its connection with different parameters, it is necessary to further study this issue, which remains relevant today. However, the obtained data allowed to assign the coefficient of operating conditions γ_c = 1.15 for the lower parts of steel step columns in the first approximation and to recommend it in the codes of design and reinforcement of steel structures. In the development of DBN B.2.6-163: 2010 " Steel structures. Standards of design, manufacture and installation” [2] this recommendation is carefully taken into account in the form γ_C = 1,05 for OIB columns equipped with overhead cranes.

5.7. Working conditions coefficient of redundant structures

In the reliability theory of building structures and mechanisms, the calculation of redundant structures (individual beams and frames, multi-storey and multi-span buildings) is considered one of the most difficult problems. The reason for this is the complex nature of the destruction of redundant structures (RS), which differs from the nature of the destruction of a statically defined system in that if one or even several elements of the RS can remain operational. Therefore, the destruction of a redundant structure can occur, as the failure of individual elements, by going through different operational states, corresponding to different schemes and system probabilistic parameters.

As a result, assessing the reliability of the RS is a rather cumbersome task, the complexity of which is growing rapidly in line with the complexity of the system. Studies conducted at the Department of CSWP PoltNTU [8, 11, 12], managed to overcome these difficulties to some extent and develop methods for assessing the reliability of the RS, suitable for practical application, and propose a new scale of working conditions coefficients for such systems.

A comparative analysis of the RS reliability calculating methods: the method of states, logical-probabilistic method, the method of limit equilibrium. On this basis, the probabilistic equilibrium method (PEM) was developed. A full estimate of the RS probability failure with random strength and load was obtained by developing an PEM variant based on the kinematic limit equilibrium method. A variant of this method was used, called the "method of combined mechanisms", which consists of the main SIS destruction mechanisms: beam, surface (landslide) and nodal. The method is implemented in the form of an algorithm and a PC program, during the calculation of which combined mechanisms with different numbers of plasticity hinges are formed, statically unacceptable (excessive) and unlikely mechanisms are rejected *(Fig. 5.3).* The assessment of the probability of SIS failure was generally defined as the disjunction of the correlated failure conditions of the main mechanisms corresponding to the increased failure probabilities.

Using the developed methods and programs, a numerical experiment was performed to determine the reliability of 140 redundant structures of different purposes and configurations, and the number of floors varied from one to three, the number of spans - also from one to three. It is quantitatively confirmed that the elastic-plastic calculation of the considered RS leads to material savings within 10… 15% in comparison with the elastic calculation.

Table 5.8

Reliability evaluation of the stepped columns of industrial buildings

Variant	***Combination of cross bar with columns***	***Geometric parameters***		***Loadings***					***Failure probabilities***		***Coefficient of working conditions***
				Roof	***Crane***		***Areas***		***Upper part***	***Lower part***	
		$\frac{L}{B}$	$\frac{l_2}{l_1}$		***Q, mc***	***Mode***	***Snow***	***Wind***	$Q_2(t)$	$Q_1(t)$	γ_c
1	*rigid*	$\frac{24}{12}$	$\frac{3{,}97}{16{,}4}$	*heat r.c. panels*	*15/3*	*7K*	*III*	*I*	*0,48*	$0{,}96\times10^{-2}$	***1,15***
2	*rigid*	$\frac{36}{12}$	$\frac{6{,}6}{12{,}4}$	*heat r.c. panels*	*125/20*	*4K-6K*	*III*	*II*	*0,023*	$4{,}15\times10^{-4}$	***1,15***
3	*hinged*	$\frac{30}{12}$	$\frac{6{,}1}{11{,}0}$	*heat steel deck*	*100/20*	*4K-6K*	*III*	*II*	$9{,}83\times10^{-4}$	$2{,}24\times10^{-7}$	***1,32***
4	*rigid*	$\frac{30}{12}$	$\frac{6{,}4}{14{,}2}$	*heat r.c. panels*	*100/20*	*4K-6K*	*III*	*II*	*0,054*	$6{,}5\times10^{-5}$	***1,25***
5	*hinged*	$\frac{36}{12}$	$\frac{6{,}02}{14{,}4}$	*heat r.c. panels*	*80/20*	*7K*	*III*	*II*	*0,213*	$1{,}71\times10^{-6}$	***1,18***
6	*hinged*	$\frac{24}{12}$	$\frac{5{,}23}{17{,}0}$	*heat r.c. panels*	*50/10*	*4K-6K*	*I*	*III*	*0,54*	$5{,}0\times10^{-8}$	***1,19***
7	*hinged*	$\frac{24}{6}$	$\frac{3{,}25}{9{,}75}$	*cold r.c. panels*	*30/5*	*7K*	*II*	*I*	*0,71*	$2{,}78\times10^{-12}$	***1,32***

In addition, for the first time, the reliability reserve was quantified compared to individual elements and statically defined systems. It is proposed to take into account this reserve for the first time introduced "circuit reliability coefficient γ_S'', similar to the working conditions coefficient of the current codes. The substantiation of the coefficient values is based on taking into account its probabilistic nature and the condition of equal RS reliability, in which the failure probability of the system as a whole is equal to the failure probability of individual elements.

The analysis showed that when the mechanism of RS destruction approaches the beam mechanism, the coefficient γ_S decreases, and when the shear mechanism approaches, it increases; the maximum values of the coefficient γ_S are obtained for the complete mechanisms of destruction, the partial mechanism leads to a decrease in the coefficient γ_S.

The obtained coefficients of circuit reliability are in the range $\gamma_S = 1{,}18..1{,}27$ *(Table 5.9)*, this indicates significant reserves of RS bearing capacity, which does not take into account current regulations. This coefficient of RS operating conditions is intended for use in sections bearing capacity calculations of RS elements taking into account a plastic work stage.

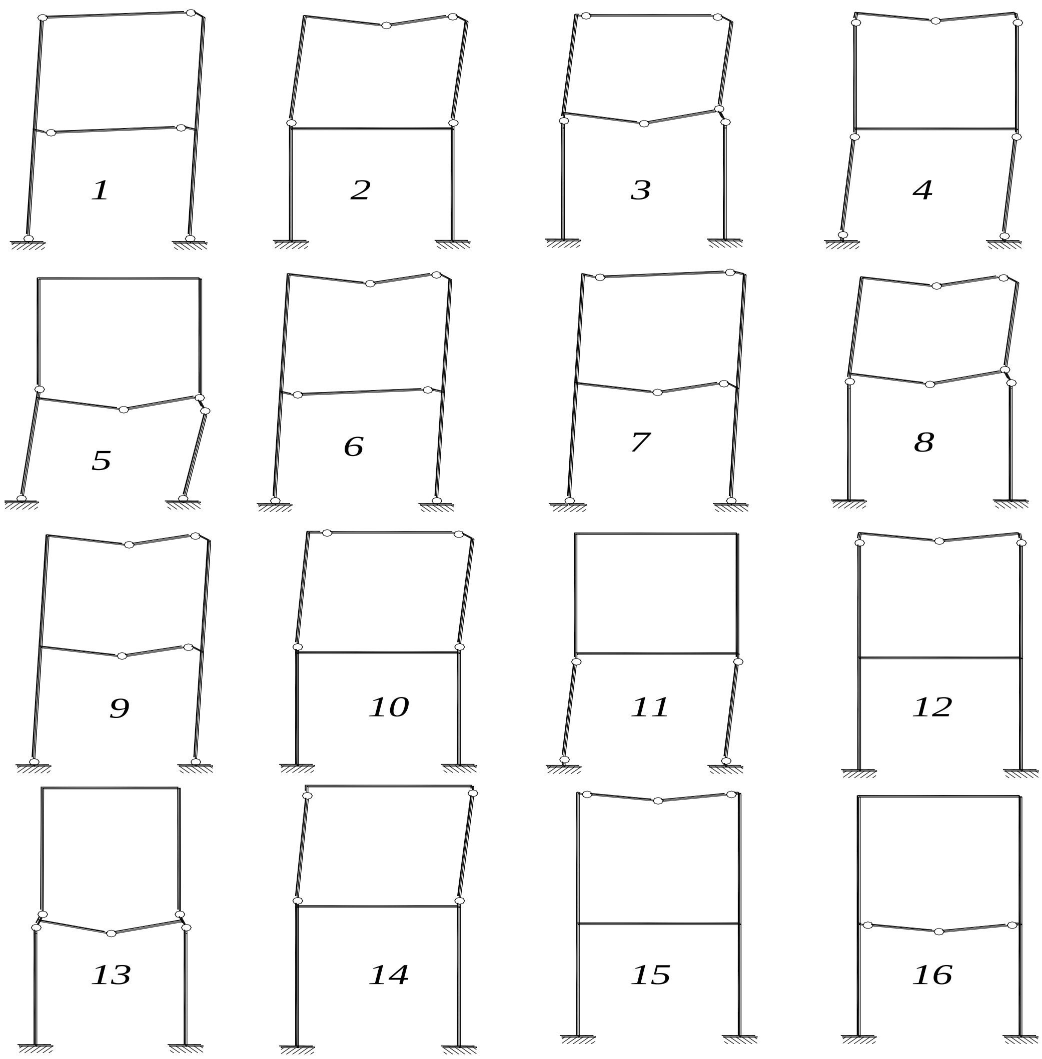

Fig.5.3. Possible mechanisms for the destruction of a two-story frame

Table 5.9

Estimated values of the working conditions coefficient γ_S for redundant steel structures

Spans number	*Floors number*		
	1 floor	*2 floors*	*3 floors*
1 span	*1,18*	*1,21*	*1,21*
2 span	*1,19*	*1,26…1,27*	*–*
3 span	*1,24*	*–*	*–*

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Control questions

1. What is the meaning of the working conditions coefficient in the method of limit states?
2. How is the ratio of working conditions in the design standards?
3. What does the constructive amendment have to do with the ratio of working conditions?
4. What is the constructive amendment of steel trusses?
5. What affects the structural repair of crane structures?
6. Give a probabilistic justification of the working conditions coefficient of stepped columns.
7. What is the working conditions coefficient of redundant structures based on?

LECTURE 6. RESPONSIBILITY COEFFICIENT

6.1. Determination of the responsibility coefficient

According to the Ukraine codes of the DBN [2], the reliability coefficient for responsibility $\boldsymbol{\gamma_n}$ (responsibility coefficient) takes into account the structure or object significance as a whole, as well as the possible failure consequences. In the previous codes of SNiP [1] this coefficient was called " reliability coefficient by purpose".

If to expand on the following concise definition, it can be noted that the responsibility coefficient is designed to differentiate the levels of safety depending on the social and economic structure significance, the consequences scale and the losses magnitude in the failure event. The main solved problem due to the coefficient $\boldsymbol{\gamma_n}$ is increasing economic efficiency by regulating the probability of failure by reducing it for buildings that are more important to society and the severe consequences of failure, and vice versa, increase the probability of failure for conventional structures.

The responsibility coefficient $\boldsymbol{\gamma_n}$ has the form of the general safety factor, by which all loads are multiplied and which is taken into account in the left part of the limit inequality of the limit state method. This coefficient can also be taken into account in the denominator of the right-hand side of the limit inequality.

6.2. Recommendations of the SNiP of the 80s on the reliability coefficient for the purpose

The first recommendations for the rationing of the coefficient were adopted in the form of resolutions of the USSR State Construction Committee in 1981-82 and were made as an appendix to the codes of loads and impacts [1]. It should be emphasized that these recommendations were introduced voluntarily, had no scientific basis and, apparently, were based on the experience gained in the design and trouble-free operation of buildings and structures.

Table 6.1.

Reliability coefficients for the purpose [1]

Responsibility class of buildings and structures	***List of buildings and structures***	***Reliability coefficient for the purpose*** γ_n
Class I	*The main buildings and structures that have a particularly* ***important*** *economic and (or) social purpose: the main buildings of thermal power plants, nuclear power plants, central units of blast furnaces, chimneys over 200 m high, television towers, buildings of the main primary network EASS, tanks for oil and oil products more than 10 thousand* m^3*, indoor sports facilities with grandstands, buildings of theaters, cinemas, circuses, indoor markets, educational institutions, children's preschools, hospitals, maternity hospitals, museums, state archives, etc.*	*1,0*
Class II	*Buildings and structures of objects that have* ***important*** *economic and (or) social purpose (objects of industrial, agricultural, housing and civil purposes and communications, not included in I and III classes)*	*0,95*
Class III	*Buildings and structures of objects with* ***limited*** *economic and (or) social purpose: warehouses without sorting and packaging processes for storage of agricultural products, fertilizers, chemicals, coal, peat, etc., greenhouses, hotbeds, one-story houses, wire supports communications, lighting poles of settlements, fences, temporary buildings and structures *, etc.*	*0,9*
** For temporary buildings and structures with a service life of up to 5 years may be accepted* $\gamma_n = 0{,}8$		

6.3. Recommendations of DBN B.1.2-14-2009 rules on the reliability coefficient for responsibility

Ukrainian codes DBN B.1.2-14-2009 «The main principles of ensuring reliability and constructive safety of buildings, construction works, building constructions and bases» [2] in the development of which the author of the lecture took part, significantly and deeply developed the basic principles of calculation of limit conditions, in particular regarding the responsibility coefficient. Subsequently, these issues were somewhat clarified DSTU-N BV.1.2-16: 2013 «Definition of consequences (responsibility) class and complication category of construction projects" [9], in the development of which also participated teachers of PoltNTU.

The values of the reliability coefficient for responsibility *(responsibility coefficient)* γ_n are determined depending on the class of consequences (responsibility) of the object and the settlement situation type according to *Table. 6.2*.

Table 6.2

Reliability coefficients for the responsibility of buildings and structures

Class	***Category***	***Values γ_n for settlement situations***				
		established		***transitional***		***emergency***
		limit states of the first group	***limit states of the second group***	***limit states of the first group***	***limit states of the second group***	***limit states of the first group***
СС3	*А*	*1,250*	*1,000*	*1,050*	*0,975*	*1,050*
	Б	*1,200*		*1,000*		
	В	*1,150*		*0,950*		
СС2	*А*	*1,100*	*0,975*	*0,975*	*0,950*	*0,975*
	Б	*1,050*		*0,950*		
	В	*1,000*		*0,925*		
СС1	*А*	*1,000*	*0,950*	*0,950*	*0,925*	*0,950*
	Б	*0,975*		*0,925*		
	В	*0,950*		*0,900*		

Note 1. *If the design standards for certain buildings or structures types do not provide specific recommendations for the distribution of structures by responsibility categories according to the consequences (responsibility)classes, they may be classified as category Б.*
Note 2. *For temporary buildings and structures with a fixed service life of up to three years, the values are accepted as for objects of class В, regardless of the consequences (responsibility) class of the structure.*

For constructions of mass application, as a rule, one value of the coefficient γ_n, is set, with which this construction should be used regardless of the consequences (responsibility) class of the object where it is actually applied.

Consequence (responsibility) classes of buildings and structures are used to ensure the reliability and structural safety of buildings, structures, linear engineering infrastructure, as well as building structures and foundations. The class of consequences (liability) is determined by the level of possible material damage and (or) social losses associated with the cessation of operation or loss of object integrity.

Possible social losses from failure should be assessed based on risk factors such as:

- danger to human health and life;
- a sharp deterioration of the ecological situation in the area adjacent to the site (for example, the destruction of storage facilities for toxic liquids or gases, the failure of sewage treatment plants, etc.);
- loss of historical and cultural monuments or other spiritual society values;

- functioning cessation of systems and networks of communication, energy supply, transport or other elements of life support of the population or society security;
- inability to organize assistance to victims of accidents and natural disasters;
- threat to the country's defense capabilities.

Possible economic losses should be estimated at costs associated with both the need to restore the failed facility and incidental losses (losses from cessation of production, lost profits, etc.).

Characteristics of possible consequences are the basis for the classification of construction objects into three classes of consequences (responsibilities) - CC1, CC2 and CC3 *(Table 6.3)* and five categories of complexity - I, II, III, IV and V *(Table 6.4)*.

Classification of buildings and structures is performed in accordance with the instructions of *Table 6.3*, regardless of each of the columns described in its characteristics of possible losses and losses from failure. The building or structure as a whole is assigned the highest of the received (largest by number) class.

Responsibility categories of structures and their elements. *Depending on the consequences that may be caused by failure, there are three categories of structures responsibility and their elements:*

A - structures and elements, the failure of which ***can lead to complete unfitness*** of the building (structure) as a whole or a significant part of it.

Б - structures and elements, the failure of which ***may complicate*** the normal operation of the building (structure) or the failure of other structures that do not belong to category A.

B - structures, failures of which ***do not lead to malfunction*** of other structures or their elements.

Responsibility categories are set by the designer and must be specified in the design documentation. Recommendations for defining these categories should usually be given in the design standards of buildings or structures of a certain type.

Category A can include structures of category A1 (main load-bearing structures), the reliability of which ensures the building or structure from complete destruction in the event of an emergency, even if its further use for its intended purpose becomes impossible without major repairs.

Category A1 should include elements whose failure could be a direct cause, an emergency with a direct threat to humans or the environment (safety valves in high-pressure vessels, parts and structural elements that seal tanks with highly toxic substances, etc.).

For structures and elements of category A, it is recommended to use the following safety principles separately or in any appropriate combinations:

- ***redundancy***, ie ensuring the implementation of basic functions due to the excessive number of elements and devices or their excessive capabilities (power, energy, etc.);

Table 6.3.

Classes of consequences (responsibility) of buildings and structures

Class of consequences (responsibility) of a building or structure	***Characteristics of possible failure consequences of a building or structure***					
	Possible danger to human health and life, number of people			***The amount of possible economic damage, m.e.***	***Loss of cultural heritage sites, categories of sites***	***Termination of transport, communications, energy, other engineering networks, level***
	who are constantly at the facility	***who are periodically at the facility***	***who are outside the facility***			
CC3 significant consequences	*More than 300 (400)*	*More than 1000*	*More than 50000*	*More than 150000*	*national significance*	*general state*
CC2 average consequences	*from 20 to 300 (50…400)*	*from 50 to 1000 (100.1000)*	*from 100 to 50000*	*from 2000 to 150000*	*local significance*	*regional, local*
CC1 minor consequences	*up to 20 (50)*	*up to 50(100)*	*up to 100*	*up to 2000*	—	—

Note 1. *A building or structure is assigned the highest class of consequences (liability) for one of all possible characteristics of possible damage from failure.*

Note 2. *When counting the number of people who may be in danger of death or health, it is assumed that there are permanent residents on the site if they are there more than eight hours a day and at least 150 days a year (generally not less than 1200 hours per year). People who visit the facility periodically are those who stay there for no more than eight hours a day for no more than 150 days a year (a total of 450 to 1,200 hours a year). Danger to life of people outside the facility is possible violation of their living conditions for more than three days.*

Note 3. *The minimum wage (m.w.) is set annually by the Law of Ukraine "On the State Budget of Ukraine" [3].*

Note 4. *Assignment of cultural heritage monuments to national and local significance is established in accordance with the Law of Ukraine "On Protection of Cultural Heritage" [6].*

Note 5. *The level of significance of engineering and transport infrastructure facilities is determined using Annex D of DSTU [9].*

Note 6. *The numbers in parentheses are placed in DSTU [9].*

- ***independence***, ie the functioning of one element (subsystem), if possible, should not depend on the ability to perform their functions by another element (subsystem);

- ***functions separation***, which reduces the probability of simultaneous failure of different elements (subsystems) due to a common cause;

- ***difference in principles***, ie the use of different in design and principle of operation of protective devices and elements.

The decision on the principles of security guarantee is made by the General Designer in agreement with the project customer and the organization that carries out scientific support of the project work and the relevant state supervisory authorities. For category A1 elements, the refusal to use the principle of independence must be specially substantiated.

Types of settlement situations. The following types of calculation situations should be considered when calculating structures:

- ***established***, which are characterized by the duration of T_{sit} implementation of the same order as the established service life of the T_{ef} construction site (for example, the operation period between two major repairs or changes in the technological process);

- ***transitional***, which are characterized by the duration of T_{sit} implementation is short compared to the established service life T_{ef} (for example, the construction period, overhaul, reconstruction);

- ***emergency***, which are characterized by a low probability of P_{sit} and, as a rule, a short duration of $T_{sit} << T_{ef}$, but which are quite important in terms of the possible failures consequences (for example, situations that occur during explosions, fires, equipment accidents, collisions of vehicles, as well as immediately after the failure of any structural element).

If the design codes do not specify the design situation, it is assumed that the relevant standards requirements belong to the established and transitional design situations that are forecasted. Emergencies must always be clearly stated.

6.4. Categories of construction complexity

The category of construction object complexity is indicated in the design task, used to determine the design stages and calculated during the development of design documentation. The calculation is given in the explanatory note of the project documentation for construction. If the calculated category of complexity does not coincide with the one specified in the design task, the task shall be amended accordingly.

The category of construction object complexity on the basis of the consequences (responsibilities) class is determined in accordance with *Table 6.4*, if the construction object is a separate building, house, structure, linear object of engineering and transport infrastructure.

When carrying out overhaul of the object, the object of construction may be part of it.

The construction site design, which includes several individual houses, buildings, structures or linear objects of engineering and transport infrastructure, should be carried out on the basis of initial data, including urban conditions and restrictions, on the construction site as a whole. The complexity category of such a construction object is determined by all indicators of *Table 6.4*, calculated for the construction object as a whole.

Based on the fact that facilities that, in accordance with the List of activities and facilities, pose an increased environmental hazard [6], as well as those that are designed taking into account the requirements of civil protection engineering and technical measures, are such that, in accordance with the Law "On Objects of High Danger" [5] pose a real threat to the emergency of a technogenic and natural character, they should be attributed to the V complexity category in accordance with the Procedure for classifying construction objects to IV and V categories of complexity [7]. The consequences (responsibility) class of such houses, buildings, structures or a linear object of engineering and transport infrastructure is determined and applied in the design in accordance with sections 4 and 6 of the standard [9].

In case of doubts about the construction objects assignment to a particular complexity category in order to obtain confirmation of the correct choice, the engineer may apply to expert organizations that meet the criteria established by the central executive authority, which ensures the formation of state policy in the field of urban planning [8], information about his official site.

6.5. Determination of actuarial risks in construction

Actuarial calculations are a system of mathematical and statistical methods for calculating insurance tariffs. The methodology is based on the use of probability theory, statistics and long -term financial calculations of the insurer's investment income. These calculations are performed by actuaries (actuary) - officially authorized persons who, having the appropriate professional training, use the methods of mathematical statistics to calculate insurance rates.

The actuarial is responsible for ensuring that the insurance funds are sufficient at the time the insurance company has to meet its obligations under the issued policies. Actuarial calculations make it possible to determine the insurance rate and the share of each policyholder in the creation of the insurance fund.

In the current conditions of accelerated scientific and technological progress, the development of capital-intensive industries, significant development around the world acquires insurance against technical risks, as well as liability to third parties associated with these works. Particularly developed technical risk insurance is in the United States, Germany, England, Japan and other countries that are members of the International Association of Technical Risk Insurers.

Table 6.4.

Determining the construction objects complexity category taking into account the class of consequences (responsibility)

Class of consequences (responsibility) of the building	***Class of consequences (responsibility) of houses, buildings, constructions, linear objects of engineering and transport infrastructure***	***Characteristics of possible failure consequences of houses, buildings, structures, linear objects of engineering and transport infrastructure***					
		Possible danger			***The amount of possible economic damage***	***Loss of cultural heritage sites***	***Termination of transport, communications, energy, other engineering networks***
		For the health and lives of people who are constantly on site	***For the health and lives of people who are periodically at the facility***	***For the health and lives of people who are outside the facility***			
		number of people	***number of people***	***number of people***	***m.e***	***categories of sites***	***level***
V	**CC3** significant consequences	More than 400	More than 1000	More than 50000	More than 150000	*national significance*	*general state*
IV	**CC2** average consequences	300-400	500-1000	10000-50000	15000-150000	*local significance*	*regional*
III		50-300	100-500	100-10000	2000-15000	–	*local*
II	**CC1** minor consequences	0-50	50-100	до 100	до 2000	–	–
I		0	до 50	до 100	до 2000	–	–

Currently in Ukraine, technical risk insurance is carried out as insurance of property interests during construction and installation works and covers the following areas of insurance.

1) Property insurance, which applies to such objects of insurance as products of construction and installation works, construction machinery and equipment, buildings and structures that are being reconstructed or overhauled.

2) Third party liability insurance. When it comes to insurance of technical risks, there are: insurance of the construction contractor from all risks (CAR - contractors all risks); installation risk insurance (EAR); car insurance; insurance of electronic devices.

If construction risks are taken into account, the first type of insurance is considered, as well as the insurance of a construction entrepreneur from all risks. It is important to note that in the design rates and values of reliability coefficients, although implicitly, permissible risk levels are laid.

The risk is expressed in the general case as follows:

$$R = Prob(F) \cdot C, \tag{6.1}$$

where *Prob*(F) is accident probability; *C* is quantitative consequences (number of deaths, waste of time or money).

It is important to note that the function describing the risk has an extreme (minimum), which indicates the possibility of optimizing the risk level *(Fig. 6.1)*. This is based on the following economic methodology for substantiation of the coefficient, developed at the Department of CSWP PoltNTU [10, 11].

6.6. Relationship between risks and coefficient $\boldsymbol{\gamma_n}$

This relationship is illustrated by the graph in *Fig. 6.2*. There are shown the the building structures cost $C_к$ (n.u.) that is plotted on the abscissa axis. In the general case, it is functionally related to the cross-sectional parameters of the structures. Here $C_{к,н}$ is the structure cost, designed according to current regulations without taking into account the coefficient $\boldsymbol{\gamma_n}$ (basic option).

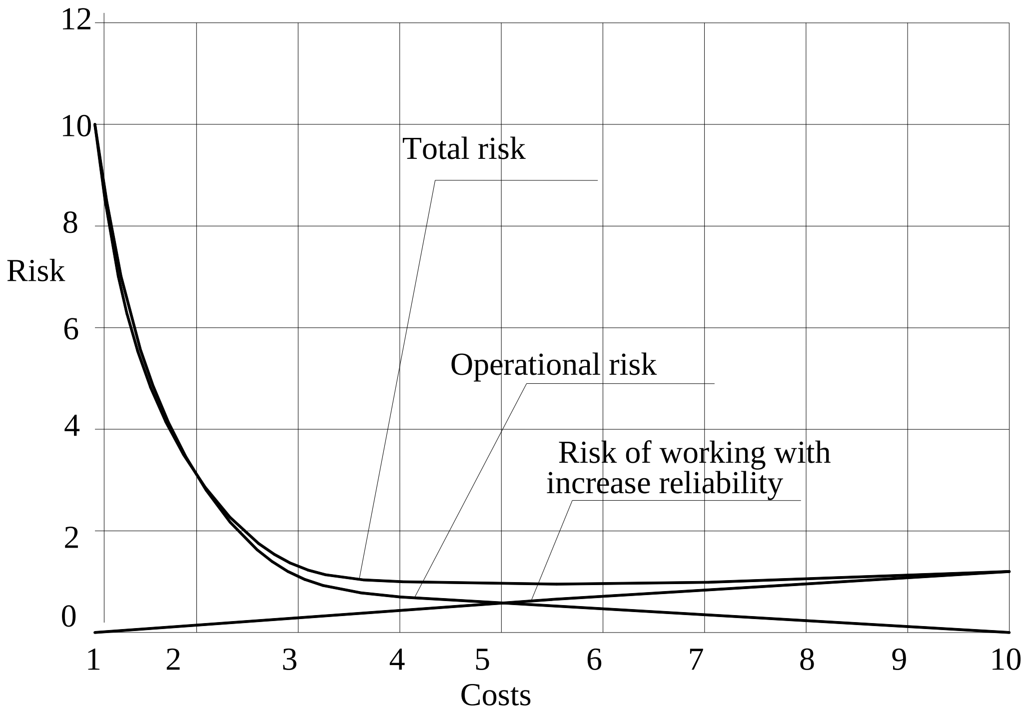


Fig. 6.1. To optimize the level of risk

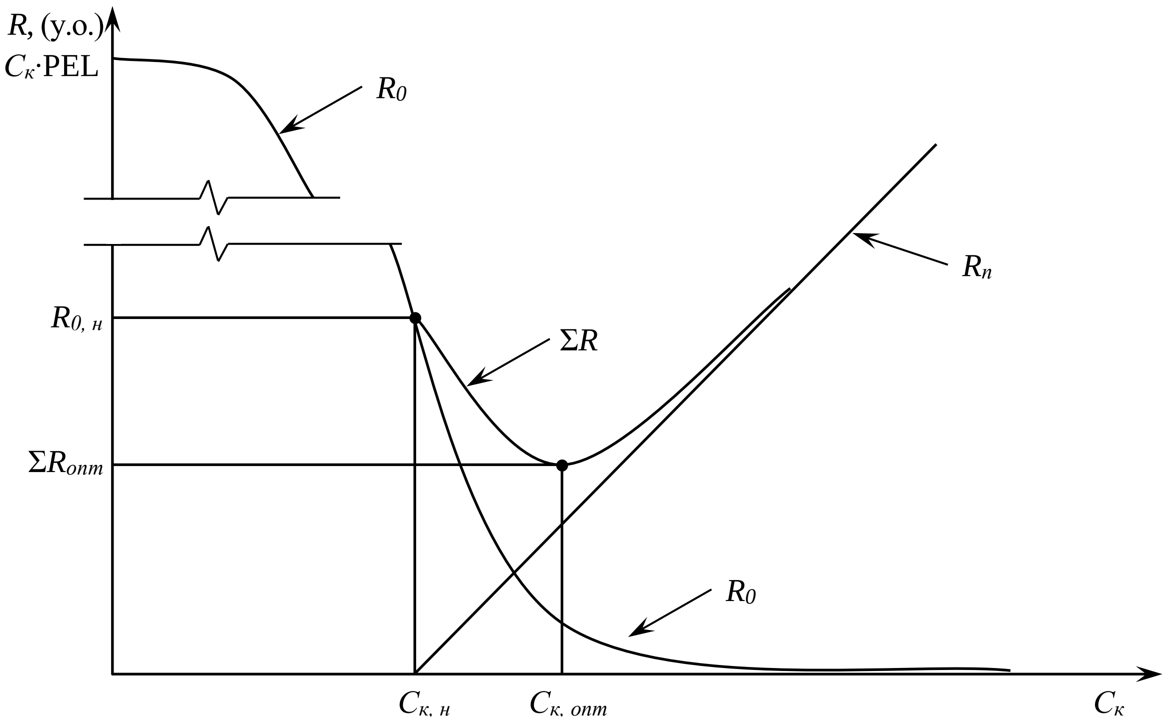


Fig. 6.2. Dependence $R - C_к$ for particularly responsible buildings

The value of risk R (*n.u.*) is plotted on the ordinate axis, and $R_{0,н}$ is the risk of losses in case of failure of the basic design variant.

The risk of project overspending, indicated R_n, is described in the graph directly according to the equation:

$$R_n = (C_к - C_{к,н})к, \tag{6.2}$$

where k is coefficient of proportionality (near the base point $C_{к,н}$), that is equal to $k \approx 1$ for steel and wooden structures and $k \approx 0{,}4...1{,}8$ for reinforced concrete structures.

Risk R_0 losses from the structure failure is defined as

$$R_0 = k \cdot C_к \cdot Q_0, \tag{6.3}$$

where Q_0 is the building object failure probability, which depending on the construction cost varies in curvilinear dependence, similar to the integral distribution function of a random variable *(Fig. 6.2)*.

The total risk consists of the costs risk in case of structures failure and the overspending risk.

$$\sum R = R_0 + R_n. \tag{6.4}$$

The total risk curve has a minimum at a level $\sum R_{onm}$ that corresponds to the value of the optimal value $C_{\kappa,onm}$ on the abscissa. Moreover, this value may be less than $C_{\kappa,н}$ (for buildings with limited liability, for which $\boldsymbol{\gamma_n < 1}$), or more $C_{\kappa,н}$, - for particularly responsible buildings *(Fig. 6.2)*, for which $\boldsymbol{\gamma_n > 1}$.

When determining the responsibility of buildings, the parameter of economic losses (PEL) is taken into account:

$$PEL = \frac{B_0}{C_{\kappa,н}}, \tag{6.5}$$

where B_0 is losses from failure (accident, destruction) of the object.

Buildings for which $PEL > 100$ are considered to be particularly significant objects.

It is proposed to define the value of the responsibility coefficient as

$$\gamma_n = C_{\kappa,onm}/C_{\kappa,н}. \tag{6.6}$$

As a result of numerical modeling, values γ_n and corresponding ratios of normative and reduced optimized risks $R_{0,н}/\sum R_{onm}$ were obtained, depending on PEL (*Table 6*). As can be seen from this table, for structures with PEL = 1… 10 the coefficients values γ_n and the risks ratio are almost indistinguishable from 1,0. At the same time, for the particularly responsible structures at the PEL > 250 and $\gamma_n \geq 1{,}15$ the value of optimal risk $\sum R_{onm}$ can be reduced against the normative $R_{0,н}$ in 20… 50 times, which should influence the determination of insurance premiums.

Table 6.5

Comparative estimation of coefficients γ_n of different responsibilities structures [10]

The load is represented as a stationary normal random process			***The load is presented in the form of random processes with the distribution of ordinate by normal (constant load) and polynomial-exponential law (snow load for the territory of Ukraine)***		
PEL	$\boldsymbol{\gamma_n}$	$\frac{R_{0,н}}{\Sigma R_{onm}}$	**PEL**	γ_n	$R_{0,н} / \Sigma R_{onm}$
10	*1,015*	*1,052*	*10*	*1,049*	*1,59*
50	*1,083*	*2,40*	*50*	*1,108*	*4,70*
100	*1,11*	*3,96*	*100*	*1,133*	*8,0*
150	*1,128*	*5,40*	*150*	*1,153*	*12,6*
250	*1,148*	*8,0*	*250*	*1,166*	*17,0*
500	*1,176*	*14,0*	*500*	*1,190*	*30,0*
750	*1,19*	*19,5*	*750*	*1,20*	*42,0*

6.7. Dependence of the responsibility coefficient on PEL [11].

The graph shown in *Fig. 6.2*, can be represented in dimensionless coordinates through PEL, dividing all monetary values into the The regulatory value of the structure element $C_{\kappa,\mu}$. This made it possible to identify the relationship between the values of the magnitude of optimal reliability $P_{L,opt}$ and PEL, which for all possible cases is quite well described in direct proportionality in the form

$$P_{L,opt} = a + b\,lg(\,PEL), \tag{6.7}$$

where a and b are line parameters, the numerical values of which depend on the specific calculation situation;

$P_L = -\,lg[1 - P(t)]$ is the probability of infallible construction work (in bel).

The largest scatter of the value $P_{L,opt}$ does not exceed 0.8 bel (and for the vast majority less than 0.4 bel). This creates the preconditions for unambiguously determined $P_{L,opt}$ only through the parameter of economic losses of PEL, and numerical values a and b assigned to the most unfavorable case in thereliability reserve. Such values are $a = 1{,}35$ and $b = 1{,}1$ which convert expression (6.7) to the following form (*Fig. 6.3*):

$$P_{L,opt} = 1.35 + 1.1\,lg(\,PEL). \tag{6.8}$$

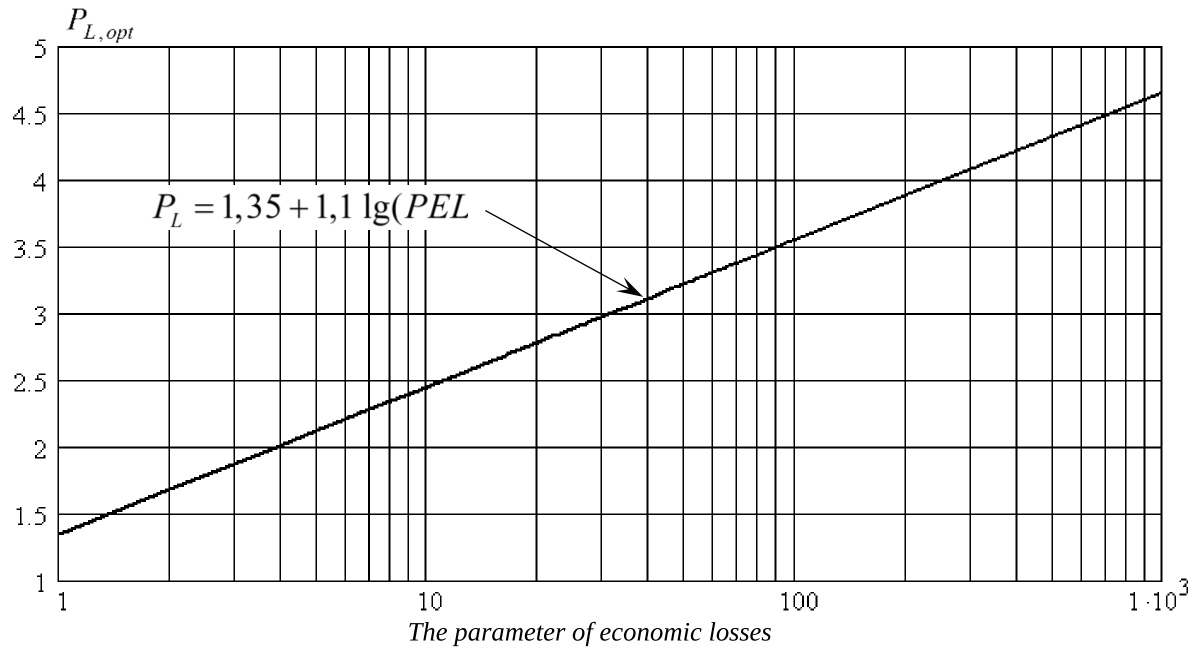
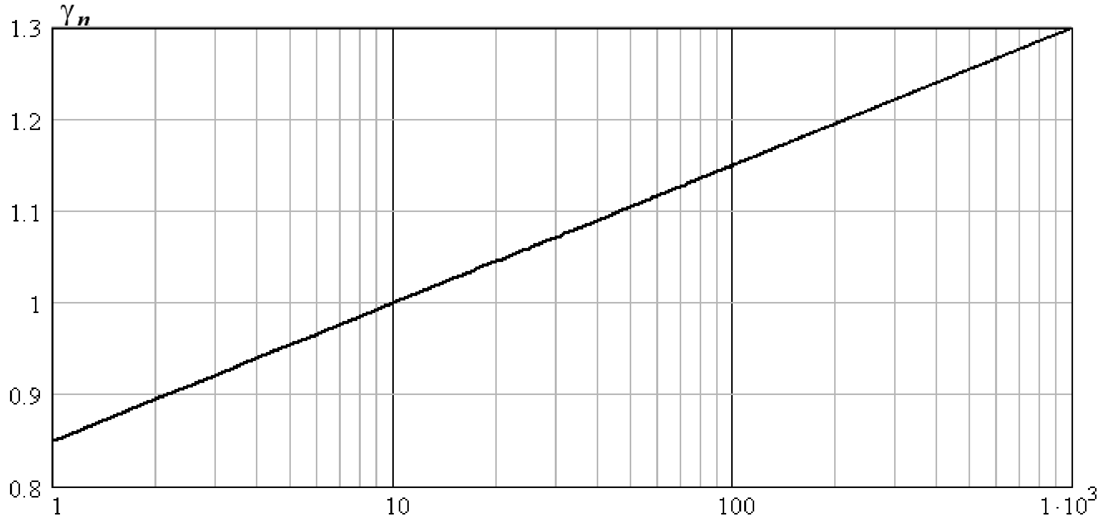


Fig. 6.3. Dependence of the optimal indicator of the element reliability on the parameter of economic losses

This approach opens the possibility of regulating the structures reliability using the reliability coefficient for the purpose, which should also be a function of the value of the parameter of PEL economic losses. This dependence of the coefficient γ_n on the PEL level can be represented similarly (6.8) in the form of a straight line *(Fig. 6.4)* with an average estimate:

$$\gamma_n = 0{,}85 + 0{,}15\, lg(\,PEL). \tag{6.9}$$

Thus, in this approach, the reliability factor for the purpose plays the role of a regulator, which leads to the actual element reliability $P(t)$ to the optimal $P_{L,opt}$ for socio-economic reasons.



The parameter of economic losses

Fig. 6.4. Dependence of the reliability coefficient on purpose on the parameter of economic losses

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Control questions

1. What is the meaning of the responsibility coefficient in the methodology of limit states?
2. How was the reliability coefficient normalized for the purpose of SNiP?
3. What determines the responsibility coefficient of buildings for DBN?
4. How is the consequences (responsibility) class of the building determined?
5. What is the complexity category of the construction object based on?
6. How are actuarial risks in construction determined?
7. What is the relationship between liability and risk?

LECTURE 7. DETERMINING THE CLASS OF CONSTRUCTION OBJECTS CONSEQUENCES

7.1. Forecasting the accident scenario of a construction site
7.2. Methods for determining the responsibility class of construction objects
7.3. Features of non-industrial construction projects
7.4 Calculation of the consequences (responsibility) class and the category of complexity of a residential building
7.5 Determining the consequences (responsibilities) class and the category of complexity of public and non-industrial buildings
7.6. Determining the consequences (responsibilities) class and the category of complexity of industrial buildings and structures

Introduction

The introduction of DBN B.1.2-14-2009 "General principles of reliability and structural safety of buildings, structures, building structures and foundations" [1] has caused numerous questions and ambiguities among user practitioners, in particular regarding the *Tables 6.2* and *6.3*. To remove possible issues and simplify the use of these rules, an auxiliary regulatory document was prepared - DSTU-N B.V.1.2.2-16:2013 "Determination of the consequences (responsibilities) class and the category of complexity of construction projects" [2], in the development of which the author of this course of lectures and other teachers of PoltNTU took part. Afterward, it has been created and implemented the National Standard of Ukraine DSTU 8855:2019 [3] “Building and construction . Determining the class of consequences (responsibility)”. The main provisions of these documents are presented in the following paragraphs.

7.1. Forecasting the accident scenario of a construction site

To calculate the possible material damage and (or) social losses in due to the object failure associated with the operation cessation or integrity loss, the designer determines the most probable predictions of a ***possible accident*** (eg, damage, failure, destruction of houses, buildings, structures, linear object of engineering and transport infrastructure or their parts), which occurred due to man-made or natural causes. The list of these forecasts is given in the explanatory project note in the section "Ensuring reliability and security" or "Complexity category calculation".

Possible damages are estimated based on the projected accident scenario, taking into account the measures envisaged by the project to localize a possible accident (for example, by dividing the construction site into separate parts). Recommendations for constructing an accident scenario are provided in Annex B of DSTU [3].

An accident scenario is a sequence of events model that may occur as a result of the initiating impact (overload, staff errors, emergency failure of protective devices, etc.) on the house construction, building or structure. The *so-called single failure principle* should be used when it is considered that the emergency situation is initiated by only one factor (failure of one structural element, one staff error, one technological process violation).

In addition to the usual calculation situations that should be anticipated during the design, the possibility of occurrence and consequences of emergencies that may occur due to non-design impacts or staff errors (designers, builders, operating staff, etc.) should be also analyzed.

It is recommended to consider, for example, the following events:

- failure and destruction of a separate load-bearing structure due to its overload by over-design combinations of loads and impacts;
- occurrence of large subsidence of soil bases during their emergency soaking;
- the impact of possible karst failure, landslides, etc .;
- the impact of vehicle collisions;
- the possibility of structures failure in case of fire;
- damage to building structures by emergency explosions (eg household gas);
- the possibility of technological regulations violation or damage to equipment (pipeline ruptures, falling loads, other non-design effects).

For multi-story buildings and structures, the hypothetical collapses listed in paragraph E.1.2 of DBN B.2.2-24 [12] should be considered as initiating events.

Identifying a list of possible events that can trigger an emergency, allows you to predict the causes and locations of dangerous phenomena and develop measures to localize a possible accident (division of the building into separate parts, installation of duplicate structures or additional ties, etc.).

Analysis of the emergency situation development is performed at the level of expert assessments. The development of an emergency situation should be considered step by step, taking into account the place of its occurrence and the possibility of its localization. The ultimate goal of such an analysis is to assess the area of destruction, the damage amount, estimating the number of people who are in a risk zone.

If an object is considered, each component of which is assessed separately, accident scenarios should also consider the emergency behavior of heat, water, gas, electricity and other networks that ensure the object functioning.

7.2. Methods for determining the responsibility class of construction objects

The assessment of possible economic losses should take into account:

- losses from destruction or damage to fixed assets;
- losses from finished industrial or agricultural products;

- losses from stocks of raw materials, semi-finished products and intermediate products.

Losses from complete or partial destruction of fixed assets are calculated based on the loss of their residual value, that is, book value, taking into account depreciation. Let us assume that the failure occurs at the moment of the average value of the established service life T_{ef} and calculate the residual value at this moment in time. Then the losses from the possible destruction of fixed assets are calculated by the formula:

$$\Phi = c\sum_{i}^{n} P_i\left(1-\frac{1}{2}T_{ef}\times K_{\text{a},i}\right), \tag{7.1}$$

where: Φ is projected losses (thousand UAN) ;

c is coefficient taking into account the relative share of fixed assets is completely lost during an accident. The value c can be estimated when analyzing the accident scenario (see paragraph 7.1) and pre-accepted c = 0,45;

P_i is estimated cost of the *i-th* lost fixed assets type, which should be understood as the total cost determined based on DBN D.1.1-1-2000 "Rules for determining the cost of construction" [16];

T_{ef} is the average value of the fixed assets operation time;

$K_{\text{a},i}$ is the depreciation deductions coefficient of the *i-th* fixed assets type;

n is fixed asset types amount (number).

Losses from the finished industrial or agricultural products loss of materials, crude materials, and semi-finished products required for the production of products are calculated based on the average values of wholesale prices for materials, crude materials, and semi-finished products.

Objects of cultural heritage of national or local significance include objects that are presented in the Ukraine State Register of Immovable Monuments [6] and the corresponding List of cultural heritage sites.

For construction projects designed in the protection zone, the possibility of their impact on cultural heritage sites of national or local importance should be taken into account, in accordance with DBN B.2.2-2-2008 "Composition and maintenance of historical-architectural supporting plan of the settlement" [13]. The size of the protection zone must not be less than two horizontal or vertical dimensions of the monument (the larger of them).

To determine the level of engineering and transport infrastructure construction facilities (national, regional or local) it is advisable to use indicators of urban planning documentation in accordance with the Law of Ukraine "On Regulation of Urban Development" [7]:

- engineering and transport infrastructure construction facilities of the national level should include facilities that are built in accordance with the

General Plan of the Ukraine territory, cross the territory or meet the need for these facilities at least two regions;

- engineering and transport infrastructure construction facilities of the regional level should include objects under construction in the Ukraine regions;
- engineering and transport infrastructure construction facilities at the local level should include objects that are being built on the territory of settlements.

For public roads should also take into account the classification provisions of the Law of Ukraine "On Motor Roads" [8].

At the stage of developing project documentation for overhaul of an existing object part (premises, apartments) without complete functional use suspension, the complexity category of the construction object is determined according to such documentation without taking into account the complexity category of the operated object.

When performing current repairs, re-planning and / or re-equipment of individual premises (apartments) without interfering with the load-bearing and enclosing structures, as well as engineering systems of the object, the class of consequences (responsibilities) and category of complexity are not determined.

To pre-determine the class of consequences (responsibilities) for the characteristics "Termination of engineering and transport infrastructure" should use Annex G DSTU [2].

Regardless of the classification according to the characteristics of *Table 6.3*, the consequences (responsibilities) class should be established not less than:

- CC3 - for high-risk facilities identified in accordance with the Law of Ukraine "On high-risk facilities" [9];
- CC3 - for civil defense shelters, regardless of location, capacity and protection class.

For built-in radiation protection shelters of civil defense the consequences (responsibility) class is accepted as for the whole building or structure. For separately located anti-radiation shelters of civil defense the consequences (responsibilities) class is determined on general terms.

7.3. Features of non-industrial construction projects

The number of people the possible danger is taken into account may be determined as follows:

- in residential buildings - the number of people who are constantly at the facility (N_1) is determined by the norm of 21 square meters of the total area for the owner (tenant) and each member of his family and an additional 10.5 square meters for the family (this norm does not apply when designing dormitories and housing for social purposes):
- in houses from social housing stock - the number of people permanently at the facility (N_1), in accordance with the established temporary minimum standards for social housing [10], is determined at the rate of 22 square meters of the total area for a family of two and an additional 9.3 square meters for each next family member;

- in social hostels - the number of persons who are constantly at the facility (N_1), in accordance with the established temporary minimum standards for social housing [10], is determined by the norm of 6 square meters of living space for each resident;
- in dormitories - the number of people who are constantly at the facility (N_1), in accordance with the §2.43 DBN B.2.2-15 “House building” [14], determined by the norm of 8 square meters of living space for each resident;
- in dormitories for post-graduate students - the number of people who are constantly at the facility (N_1), in accordance with the §2.43 DBN B.2.2-15 “House building” [14], is determined by the norm of 10 square meters of living space for each resident;
- in buildings up to 73.5 meters high to accommodate offices - the number of people periodically at the facility (N_2) - three people per room or ten people per 100 m^2 of the total area (in this case, one of the highest characteristics is taken);
- the number of people outside the facility (building) (N_3) should be determined by the following formulas:

$$N_3 = \alpha \times N_1; \tag{7.2}$$

where the coefficient α is determined according to *Table 7.1*.

For residential buildings, it can be assumed that $K_{a,i}$ is equal to one percent, and the established service life is 100 years. Then the calculation formula (7.1) changes as follows:

$$\Phi = 0{,}45 \sum_i^n P_i \left(1 - \frac{1}{2} 100 \times 0{,}01\right) = 0{,}225 \sum_i^n P_i. \tag{7.3}$$

Features of the consequences class of (responsibility) use in the design of located in seismic areas objects are given in paragraph 6 of DSTU [7].

Table 7.1

Coefficient α

The house height above ground level, m	***The coefficient α value depending on house placing:***		
	Rural area	***In a town or in a residential area of a city***	***In the city centre***
Less than 10	*1,0*	*1,0*	*1,3*
From 10 to 30	*1,0*	*1,3*	*1,5*
Over 30	*1,3*	*1,5*	*2,0*
Note. *The basis of the data table. 7.1 is set out in DBN B.1.2-14-2009 [1], which is developed based on the progressive international experience implemented in the international normative document ISO 2394: 1994 General principles on reliability for structures (General principles of reliability). The indicators in the table were borrowed from The Building Regulations 2000. Structure. APPROVED DOCUMENT A (Building codes and rules of 2000. Structure. Approved document A)*			

7.4. Calculation of the consequences (responsibility) class and the category of complexity of a residential building with underground parking.

Initial data. One-section 16-storey 196 apartment residential building in Kiev.

1. The residential building consists of two sections separated from each other by a deformation join, which have a common underground parking lot located under the house. Accept a 6-apartment section according to the formula for floor apartments *1-1-2-2-3-3*.

2. Determine the estimated residents number depending on the apartment area (at the rate of 21 m^2 per person plus 10.5 m^2 per family, *Table 7.2*).

The number of people who are constantly in the house (N_1) is *388*. Total The total number of people who are permanently on site (including the parking staff and concierge services - 5 people) is: *388 +5 =393* people. According to *Tables 6.3* and *6.4*, the consequences (responsibility) construction object class is CC2.

3. Temporary people staying in residential buildings is not standardized and in any case, does not exceed 50% of people who are constantly in houses, that is, N_2 will be *196* people.

By the number of people who are periodically at the facility, the residential building belongs to the class of consequences (responsibility) CC2.

4. The number of people outside the building N_3 (for a residential area) is determined by the formula:

$$N_3 = 393+196=589 \text{ people,}$$

By the number of people outside the object, the residential building belongs to the class of consequences (responsibility) CC2.

Table 7.2

Estimated number of people in the building

Apart-ment type	***Apartments area***	***Number of apartments per building***	***Total area of apartments per building***	***Accommodation in an apartment (Estimated occupancy rate)***	***Accommodation at the building***
1	*40,5 (30+10,5)*	*64*	*2592*	*1,43*	*92*
2	*52,5 (42+10,5)*	*64*	*3360*	*2*	*128*
3	*65,5 (55+10,5)*	*64*	*4192*	*2,62*	*168*
Total		*192*	*10144*		*388*

5. According to the calculation, the number of square meters in the house is *10144 m²* (*Table 7.2*). The parking area is *870 m²*. The number of parking spaces – *58* (in accordance with DBN B.2.3-15 [17]).

Note. *The estimated cost of 1 m² apartment area is 13,849 UAH per m² (as of January 1, 2019). The estimated cost of 1 m² underground parking is 23000 UAN.*

Note. *The example uses the indicator of the mediated housing construction cost in Kiev in accordance with the order of the Ministry of Regional Development No. 335 dated 6.12.2018. Cost indicator per 1 m² of the total area takes into account the non-apartment areas of the building, therefore, to calculate the value of the building as capacity, only the total area of its apartments is taken.*

When determining the class of consequences (responsibility) and categories of building complexity in a particular region, indicators of the indirect cost of construction by regions should be used, which are approved by the central executive body that ensures the formation of state policy in the field of urban planning and is in force at the time of the calculation.

The estimated building cost is:

$$13849 \times 10144 = 140484{,}256 \text{ thousand UAH;}$$
$$23000 \times 870 = 20010 \text{ thousand UAH.}$$

The entire estimated building cost is:

$$140484{,}256 + 20010 = 160494{,}256 \text{ thousand UAH.}$$

Forecasted losses are determined by the formula (7.3):

$$\varPhi = 0,225 \times P_i = 0,225 \times 74632,261 = 16792,258 \ \textit{thousands UAH} =$$
$$= 16792,258 / 6,7 = 2506,27 \ \textit{m.e.}$$

Note. *The minimum earnings should be specified at the time of the calculation in accordance with the Law of Ukraine "On the State Budget of Ukraine". The minimum earnings (m.e.) at the time of the calculation amounted to 6700 UAH (for 2022).*

In accordance with *Tables 6.3* i *6.4* the residential building belongs to the class of consequences (responsibility) CC2.

6. The building is not located in the protected area of cultural heritage sites and is not a cultural heritage site.

7. Accept that the failure of the building does not affect the termination of the transport operation, communications, and energy facilities.

Conclusion: According to DSTU [3], the class of consequences (responsibility) of the construction object is established by the very characteristic of the possible consequences obtained from the calculation results.

According to the criteria of the *Table 6.4* "Possible danger to the health and life of people who are constantly at the facility", "Possible danger to the life of people outside the facility", "The amount of possible economic damage" 16-storey 192-apartment residential building belongs to the class consequences (responsibility) **CC2**.

When designing a complex of residential buildings combined in one project, the category of complexity of the object as a whole should be determined taking into account the possible violation of normal living conditions of people who are constantly on site, due to failure of life support systems. The number of such people is counted as the total number of people permanently present in all buildings of the complex.

7.5. Determining the consequences (responsibilities) class and the category of complexity of public and non-industrial buildings

7.5.1. Grocery store with two-level underground parking

1. Determine the number of people who are permanently on the object N_1.

According to technological solutions, the number of store employees (sellers, service staff) is 35 people, parking service staff - 12 people.

2. Determine the number of people who are periodically in the store (N_2). The total store area is 1500 m^2, including 800 m^2 of shopping halls. According to paragraph 8.2 DBN B.2.2-23-2009 "Trade enterprises" [15] the number of customers is determined at the rate of 3 m^2 of retail space (including equipment) per person:

$$N_{1customers} = 800/3 = 267 \text{ people.}$$

Therefore, the total number of people who are permanently on the site is:

$$N_{1customers} = 35+12+267 = 314 \text{ people.}$$

According to the number of people who constantly visit the site, the grocery store with two-level underground parking belongs to the consequences (responsibility) class CC2.

3. Number of people who are periodically in the two-levels parking, equal to the number of parking lots: N_2 = 153 people.

According to the number of people who periodically visit the site, the grocery store with two-level underground parking belongs to the consequences (responsibility) class CC2.

4. The number of people outside the grocery store with two-level underground parking is determined depending on the total number of permanent residents in the three residential buildings are located near the store and in the grocery store with two-level underground parking::

$$N_3 = 314+153=467 \text{ people},$$

According to the number of people outside the facility, the grocery store with two-level underground parking belongs to the class of consequences (responsibility) CC2.

4. Determine the amount of possible economic damage.

A. Grocery store with a total area of 1500 m^2.

According to the object of the grocery store, the cost of 1 m^2 of the total area, including equipment, is 18000 UAN.

Note. *When calculating the value of a construction object, the cost indicators of the analogous object can be used.*

Estimated cost of the store:

$$18000 \times 1500 = 27000 \text{ thousand UAN}$$

Estimated losses for the store building are determined by formula (7.3):

$$\Phi = 0{,}225 \times 27000 = 6075 \text{ thousand UAN}$$

B. Two-level underground parking for 153 cars.

The total area of the two-level underground car park is 5,000 m^2. The 1 m^2 cost of the total parking lot area is 23000 UAN.

Estimated cost of parking:

$$23000 \times 5000 = 115000 \text{ thousand UAN}$$

Projected losses for two-level underground parking are determined by formula (7.3):

$$\Phi = 0{,}225 \times 115000 = 25875 \text{ thousand UAN}$$

B. The total projected loss for a grocery store with a two-level underground car park is:

$$Ф = 6075+25785=31950 \text{ thousand UAN}$$

The amount of possible economic loss in the minimum wage is:

$$31950/6,7 = 4768,67 \text{ m.w.}$$

According to *Tables 6.3* and *6.4*, a grocery store with two-level underground parking belongs to the consequences (responsibility) class CC2.

5. The building is not located in the protection zone of cultural heritage sites and is not a cultural heritage site.

6. Assume that the building failure does not affect the transport closure, communications, energy at the national, regional or local levels.

Conclusion. According to all the calculations of the possible consequences characteristics in accordance with *Table 6.3*, the grocery store with two-level underground parking belongs to the class of consequences (responsibility) **CC2**.

7.5.2. Indoor sport hall with seats for 100 people

Initial data. Indoor sport hall with seats for spectators.

1. ***General characteristics*** of the designed structure: the construction of an indoor universal sports hall is rectangular in plan with dimensions of 30 × 48 m^2, the height to the top of the supporting coating structures elevation is 10 m, the building area is 1440 m^2. Structural scheme: one-storey, one-span steel frame building with light enclosing wall structures.

2. ***According to the classification*** of DBN V.2.2-13-2003 [11], according to functional purpose and the nature of its use, the building is classified as educational and training.

3. ***The permanent staff is 6 people***. The number of shifts per day - 3. The estimated number of people constantly at the facility is determined using the standard values of the throughput in accordance with *Table 2, 9* DBN V.2.2-13-2003 [11]. The corresponding data are given below in tabular form (*Table 7.3*).

Table 7.3

Calculating the number of people in a sport hall

Kind of sport, account unit	***Throughput, people/shift***	
	during training sessions in the sport hall	***during completion, people***
Basketball, volleyball	*24*	*48*
	24	*48*
Table tennis, 3 tables	*4 (each table)*	*8 (4 each table)*
Group classes on general physical training	*35*	*–*

Thus, taking into account the service staff, the number of people who are permanently on site is:

$$N_2 = 6 + 48 = 54 \text{ people.}$$

According to the number of people who are constantly on the site, the gym belongs to the class of consequences (responsibility).

4. ***Determine the estimated number*** of people who are periodically at the facility, using the normative values of the throughput are given above:

The number of seats for spectators in accordance with the design assignment is 100.

The number of people who are periodically at the facility:

$$N_2 = 48 + 100 = 148$$

According to the number of people who are constantly on the site, the gym belongs to the class of consequences (responsibility) CC1.

1. ***The number of people*** outside the facility with a 15 m building height:

$$N_3 = \alpha \times N_2 = 1{,}3 \times 148 = 193,$$

where $\alpha = 1{,}3$ – for placing a structure in a residential area of a large city.

Thus, in accordance with the *Table 6.4* of the lecture notes on the criteria of possible danger to people, the construction object belongs to the class of consequences (responsibility) CC2.

6. Losses from destruction and damage to fixed assets for non-production purposes are calculated by the formula (7.1):

$$\Phi = c\sum_{i=1}^{n} P_i \left(1 - \frac{1}{2} T_{ef} \times K_{a,i}\right),$$

Where $n = 1$ – fixed assets number;

$c = 0{,}45$ – a ratio that takes into account the relative share of fixed assets, which is completely lost in the event of failure;

$T_{ef} = 100\ років$ – established service life;

$K_a = 0{,}01$ – depreciation rate;

$P_i = 33{,}54\ млн.\ гр.$ – the estimated cost of an analogue project, determined using a comparative approach to property valuation.

Thus, the losses are

$$\Phi = 0{,}45 \times 33540 \times (1 - 50 \times 0{,}01) = \ 7546,5 \quad thousandUAN =$$
$$= 7546{,}5/6{,}7 = 1126,3 \quad m.w.$$

Therefore, according to the criterion of the possible economic losses volume in the *Table 6.4* lecture notes, the construction object belongs to the class of consequences (responsibility) CC1.

The structure is not located in the protected area of cultural heritage sites and is not an object of cultural heritage.

Conclusion: According to the criteria of the general requirements of DBN V.1.2-14-2009 and DSTU 8855:2019 [3], as well as the above calculations, the construction of an indoor sport hall with seats for spectators for an educational institution belongs to the class of consequences (responsibility) CC2.

7.5.3. The logistics center warehouse

1. General characteristics of the building: the logistics center warehouse of high-shelf storage is rectangular in plan measuring 151 × 364 m, divided by a transverse deformation seam into three compartments with a maximum length of 133 m. The warehouse building is located outside the settlement at a distance of 5 km, consists of warehouses and an administrative part with a total area of 1500 m^2. The height to the top of the load-bearing structures of the pavement is 11.5 m, the area of the largest of the compartments - 20089 m^2. Structural scheme of the building is a one-storey, single-span frame structure with a mixed frame (reinforced concrete columns, overlapping - reinforced concrete beams with profiled flooring, insulated with mineral wool, enclosing wall structures - sandwich panels).
2. Number of people permanently working in the warehouse - 180, number of people permanently working in administrative premises - 60. Total number N_1 = 240 people. According to the number of people who are constantly on the site, the logistics center warehouse belongs to the class of consequences (responsibility) CC2.
3. Number of people who periodically stay on the site - N_2 = 120 people. According to the number of people who periodically visit the facility, the logistics center warehouse belongs to the class of consequences (responsibilities) CC2.
4. The number of people outside the facility is accepted N_3 = 60 people. According to the criterion "Possible danger to the people lives outside the facility", the logistics center warehouse belongs to the class of consequences (responsibility) CC1.
5. Possible economic losses are calculated based on the most probable forecast of the building accident, given in the explanatory note of the project. This forecast assumes the one of the compartments overlap destruction under the

influence of excessive dead and snow loads. As a result of the accident, technological equipment may be damaged and the warehouse may be shut down for a period of 30 days. After the necessary repairs, the operation of the logistics center is restored in full.

Losses from destruction and damage to fixed assets for warehousing purposes are calculated by formula (7.1):

$$\Phi = c\sum_i^n P_i\left(1-\frac{1}{2}T_{ef}\times K_{a,i}\right),$$

where n =1 is number of fixed assets;

c = 0,45 is a factor that takes into account the relative share of fixed assets, which is completely lost in the event of failure;

T_{ef} = 60 years is the established operation life for warehouse buildings;

K_a = 0,017 is depreciation coefficient;

P_i = 300 thousand UAN is the estimated cost of the analogue project.

Thereby,

$$\Phi=0,45\times300000\times(1-0,5\times60\times0,017)=66150 \text{ thousand UAN}$$

The amount of possible economic loss in the minimum wage is:

$$66150/6,7 = 9873,13 \text{ m.w.}$$

Given the amount of possible economic damage, the object belongs to the class of consequences (responsibility) CC2.

Conclusion. The consequences (responsibility) class the construction object is established according to the highest characteristic of the possible consequences obtained as a calculations result. According to the criterion of *Table 6.3* "The amount of possible economic damage", the composition of the logistics center belongs to the class of consequences (responsibilities) **CC2**.

7.6. Determining the consequences (responsibilities) class and the category of complexity of industrial buildings and structures

7.6.1. Cigarette shop of a tobacco factory

1. General characteristics of the building: cigarette shop rectangular in plan measuring 24 × 144 m, divided by a transverse deformation seam into two compartments 72 m long. Height to the top of the load-bearing structures is 10.5 m, building area 3456 m^2. Structural scheme of the building: one-storey, single-span frame structure with mixed frame (reinforced concrete columns, coatings - steel trusses with light roof enclosing structures, wall enclosing structures -

sandwich panels). Cigarette shop is located outside the village at a distance of 5 km.

2. Number of workers permanently working in the shop – N_1 = 25 people.

Number of workers who periodically visit the facility – N_2 = 10 people.

The number of people outside the facility is accepted – $N_3 = 60$ people.

Given the above indicators, the object belongs to the class of consequences (responsibility) CC1.

3. Possible economic losses are calculated based on the most probable forecast of the building accident, given in the explanatory note to the project. This forecast assumes the roof destruction of one of the temperature blocks under the influence of excessive dead and snow loads. As a result of the accident there may be damage to process equipment and the shop shutdown for a period of T_{stop} = 20 days. After the necessary repairs, the operation of the cigarette shop is restored in full.

4. Losses from destruction and damage to fixed assets for production purposes are calculated by formula (7.1):

$$\varPhi = ac\sum_{i=1}^{n} P_i \left(1 - \frac{1}{2}T_{ef} \times K_{a,i}\right),$$

where $n = 1$ is number of fixed assets;

a= 0,5 is coefficient taking into account the failure forecast, according to which one temperature block can be destroyed - half of the building;

$c = 0{,}45$ is a factor that takes into account the relative share of fixed assets, which is completely lost in the event of failure;

$T_{ef} = 60$ years is set operation life for industrial buildings;

$K_a = 0{,}01$ is depreciation coefficient;

$P_i =$ 400 thousands UAN is estimated cost of the analogue project.

Thereby,

$$\varPhi = 0{,}5 \times 0{,}45 \times 400000 \times (1 - 0{,}5 \times 60 \times 0{,}01) \\ = 63000 \quad thousandUAN = 63000/6{,}7 = 9402{,}98 \quad m.w.$$

5. The cigarette shop produces daily products, which are valued at average wholesale prices in the amount of *C = 12 million UAH*. Therefore, losses of finished products as a result of shop stopping are defined as:

$$P_P = C \cdot T_{stop} = 12 \times 20 = 240000 \quad thousandUAN = \\ = 240000/6{,}7 = 35820{,}9 \quad m.w.$$

6. Total losses from failure of the structure are defined as:

$$M_P = \varPhi + P_P = 9402{,}98 + 35820{,}9 = 45223{,}88 \quad m.w.$$

Given the amount of economic losses, according to *Table 6.3*, the building of the cigarette shop belongs to the consequences (responsibility) class **CC2**.

Conclusion. Given the provisions of DSTU, according to which the building as a whole is assigned the highest of the received class, the building of the cigarette shop of the tobacco factory belongs to the consequences (responsibility) class **CC2**.

7.6.2. The tank is located separately

Initial data. Steel tank for diesel fuel with a capacity of 5000 m^3.

1. A cylindrical tank for diesel fuel is not part of the tank farm, therefore it does not belong to the facilities that pose an increased environmental hazard. The reservoir is located in Vinnytsia region near agricultural land (pasture).

2. The permanent maintenance staff consists of three people, in addition, up to 10 people can periodically be on the site. The presence of unauthorized people is not expected near the object, and there are no factors that can lead to the appearance of people near the site. For these indicators, according to *Table 6.4*, the reservoir belongs to the class of consequences (responsibility) CC1.

3. Tank failure can lead to the following negative consequences:

- losses from the tank destruction (loss of fixed assets);
- losses from the loss of petroleum products reserves that are in the tank;
- losses from the environmental consequences of oil spills.

All these losses are assessed according to the Methodology for assessing losses from the consequences of man-made and natural emergencies, approved by the Resolution of the Cabinet of Ministers of Ukraine 15.02.2002 No. 175 (hereinafter referred to as the Methodology) [4].

4. Losses from destruction and damage of non-production fixed assets are calculated according to the formula (6.1):

$$\Phi = c\sum_{i}^{n} P_i\left(1-\frac{1}{2}T_{ef}\times K_{\mathrm{a},i}\right).$$

In this case:

- the number of fixed assets types $n = 1$;
- the coefficient taking into account the relative share of fixed assets that is completely lost c = 0,45;
- established service life T_{ef} = 40 years;
- the coefficient of depreciation deductions K_a = 0.02;
- the estimated tank cost according to the analogous project is P_i = 27 million UAH;

Then is

$$\Phi = 0{,}45 \times 27000 \times (1 - 0{,}5 \times 40 \times 0{,}02) = 7290 \quad thousandUAN =$$
$$= 7290/6{,}7 = 1088{,}1 \quad m.w.$$

5. Losses from reserves of petroleum products that are in the reservoir at the price of diesel fuel *26.74 UAH / l = 26740 UAH /* m^3 (as of 03/01/2020) are equal to *26740 × 5000 = 133 700 000 UAH*. This is *133700 / 6,7=19955,22 (m.e.).*

6. Losses from disturbance of agricultural land according to the methodology are calculated on the basis of standard loss indicators for various types of agricultural land by regions by the formula:

$$P_{с/г} = S \cdot P,$$

where *S* is the losses standard for various types of agricultural land by regions (*Table 3* of the Methodology [4])

P is the area in hectares of agricultural land of the corresponding type, withdrawn from use.

7. According to *Table 3* of the Methodology for pastures located in the Vinnitsa region, adjusted for an increase in the exchange rate *S* = 228.3 × 2 = *456.8 thousand UAH / ha*.

The diesel fuel spill is calculated based on the thickness of the liquid in the spill zone, which can be conventionally taken equal to 1 cm. Then

$$P = 5000 / 0{,}01 = 500000 \text{ м}^2 = 50 \text{ ha}.$$

The losses are equal to

$$P_{с/г} = 458{,}6 \cdot 50 = 22930 \text{ (thousands UAH)} = 22930 / 4{,}723 = 4855{,}0 \text{ (m.e.)}.$$

8. Total losses are equal to

$$T = 1088{,}1 + 19955{,}22 + 4855{,}0 = 25898{,}32 \text{ (m.e.)}.$$

Conclusion: According to the criterion "The amount of possible economic damage" *Table 6.4* of the lecture notes, the tank must be attributed to the class of consequences (responsibility) **CC2**.

6.6.3. Wind power plant (WPP)

1. A 4 MW WPP (two wind power plants WPP UNISON U88 and UNISON U93 with a capacity of 2 MW each) is located in the Novoazovsk district of Donetsk region.

2. Wind power plants do not belong to potentially dangerous objects, according to the Ukraine Law "On high risk objects" [9], based on the following:

- WPP does not belong to the facilities where hazardous substances are used, manufactured, processed, stored or transported;

- WPP does not belong to the facilities that are a real threat of emergencies of man-made and natural nature.

Permanent maintenance staff (N_1) is 3 people, in addition to the site can be periodically (N_2) up to 10 people.

According to the criteria "Possible danger to the health and life of people who are constantly on the site" and "Possible danger to the lives of people who are periodically on the site" the construction of wind power plant belongs to the class of consequences (responsibility) CC1 and II category of complexity.

3. A wind farm is an object of livelihood of a small settlement of the Novoazovsk district of the Donetsk region with a population of 800 people. When calculating the number of people outside the facility, it is necessary to take into account the total number of electricity consumers for whom the decommissioning of the wind farm may lead to living conditions disruption. According to the criterion "Possible danger to persons outside the facility", the wind farm belongs to the consequences (responsibility) class CC2 and III category of complexity.

4. Losses from destruction and damage to fixed assets for production purposes are calculated by formula (7.1):

$$\varPhi = c\sum_{i=1}^{n} P_i\left(1-\frac{1}{2}T_{ef}\times K_{a,i}\right),$$

where $n=1$ is a number of fixed assets;

$c=0{,}45$ is a factor that takes into account the relative share of fixed assets, which is completely lost in the event of failure;

T_{ef} = 20 years is set operation life for industrial buildings;

K_a = 0,05 is depreciation coefficient;

$P_i =$ 69750 thousands UAN is estimated cost of one WPP.

Thereby,

$$\Phi=0{,}45\times(2\times69750000)\times(1–0{,}5\times20\times0{,}05)= 31387{,}5 \text{ thousand UAN}$$

The amount of possible economic loss in the minimum wage is:

$$31387{,}5/6{,}7 = 4684{,}7 \text{ m.w.}$$

Given the amount of possible economic damage, the construction of a wind power plant belongs to the consequences (responsibility) class CC2.

5. According to DBN B.1.1-12 the site has a seismicity of 5 points.

6. The object is not located in the protection zone of cultural heritage objects and is not a cultural heritage object.

7. The facility is connected to its own electricity network with a voltage of 35 kW, and therefore does not affect the transport closure, communications, energy at the national and regional levels. The internal electrical network is not

designed to ensure the WPP interoperability, so the failure consequences of the internal network are not considered separately.

Conclusion. According to DSTU, the consequences (responsibility) class of the construction object is set according to the highest characteristics of possible consequences obtained from the calculations results. According to the criteria of *Table 6.3* "Possible danger to people outside the facility" and "Amount of possible economic damage", the wind power plant belongs to the class of consequences (responsibility) **CC2**.

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Control questions

1. How is the construction accident scenario predicted?
2. How are losses from possible destruction of fixed assets taken into account?
3. What features are taken into account when accounting for residential buildings losses?
4. How is the calculation of losses for industrial facilities performed?
5. How is the consequences (responsibility) class CC and the category of complexity of the construction object assessed?

LECTURE 8. LOADS: CLASSIFICATION, COMBINATION OF LOADS

8.1. Classification of loads

First, let's define the terms "loadings" and "loads".

The loading is any cause as a result of which internal stresses, strains or other parameters of a condition change in a structure (p. 3.10 [1]).

The load is an effect, which means both direct force effects and effects from displacement of supports, temperature changes, shrinkage and other similar phenomena, which produce reactive forces (p.3.26 [1]).

In the future we will use the combined term as load.

Depending on ***occurrence cause***, the loads are divided into (paragraphs 6.5.3… 6.5.5 [1]):

- *Basic loads*, which are the inevitable consequences of natural phenomena or human activities;
- *Episodic loads*, which are realized extremely rarely (one or several times during the operating time of the building) and the duration of which is incomparably short compared to operational time T_{ef}. Episodic loads also include accidental loads and effects that are undesirable results of human activity (consequences of gross errors), or the results of unfavorable coincidences (accidental loads can include very rare effects of natural origin, such as loads from tornadoes, tsunamis, etc.).

Depending on the time variability, the loads are divided into:

- *permanent loads*, which are active during the entire period of operation of the object and their values do not significantly change over time;
- *variable loads*, for which the change of values in time relative to the average cannot be neglected.

Depending on ***the characteristic duration of the uninterrupted action*** on the structures T_d the variable loads are divided into:

long-term loads, the duration of which T_d may be close to the established working life T_{ef} of the building object;

- *short-term loads*, which are implemented many times during the operating time of the structure and for which the duration of operation is significantly less than T_{ef} ($T_d << T_{ef}$) and which can be divided into repeated and episodic.

Depending on ***the method of application of loads*** in space, the loads can be divided into:

- *fixed loads*, which can act only on well-defined places of structure;
- *free loads*, which can be arbitrarily distributed throughout the structure within certain specified limits.

Consideration of the free loads can be reduced to the consideration of a number of fixed loads complexes that were obtained by fixation of the possible distribution of the free loads in the space.

Loads depending on ***the reaction of the structure*** are divided into:

- *static loads*, which do not cause significant accelerations of the structure, which allow to neglect the inertial forces;
- *dynamic loads*, which cause such accelerations that inertial forces can not be neglected.

The relevance of the loads to the class of static or dynamic depends on the ratio of the properties of these loads to the properties of the structures. The parameters of the structures for which the loads or influences begin to create a dynamic effect (e.g., the limit value of the period of natural fluctuations) must be established in the design cods.

The load can be represented by the sum of two components that are static and dynamic. In order to simplify the calculation in some cases specified in the design codes, the dynamic loading can be regarded as static, and the dynamic effect which depends on the reaction of the structure can be taken into account by an appropriate increase of the load or by multiplying the results of the static calculation by the coefficient of dynamic.

8.2. The loads of different duration

Permanent loads should be considered (p. 4.11 [2]):

a) the weight of structure parts, including the weight of load-bearing and enclosing structures;

b) weight and pressure of earth (embankments and backfills), rock pressure.

The force from the preload, which is stored in the structure or in the foundation, should be taken into account as the force from the permanent loads

Variable long-term loads should include (paragraph 4.12 [2]):

a) weight of temporary partitions, concrete leveling and blinding for equipment;

b) the weight of stationary equipment: verstats, apparatus, motors, tanks, pipelines with fittings, supports and insulation, conveyors, stationary elevating machines with their ropes and guides, as also the weight of liquid and solid substances that fill the equipment;

c) pressure of gases, liquids and bulk bodies in tanks and pipelines, overpressure and discharge of air that occurs during ventilation of mines;

d) loads on the floor from storage materials and shelving equipment in warehouses, refrigerators, granaries, bookstores, archives and similar premises;

e) temperature technological loadings from stationary equipment;

f) the weight of the water layer on water-filled flat roofs;

g) the weight of industrial dust depositions, if its accumulation is not excluded by appropriate measures;

h) loads from people and livestock, equipment on the floors of residential, public and agricultural buildings with quasi-constant design values;

i) vertical loads from overhead travelling crane and underslung crane with quasi-constant design values;

j) snow loads with quasi-constant design values;

l) climatic temperature loads with quasi-constant calculated values;

m) loadings which are caused by strains of the base and which are not accompanied by a radical change in earth structure;

n) effects that are caused by changes in humidity, components of the aggressive environment, shrinkage and creep of materials.

Variable short-term loads should include (paragraph 4.13 [2]):

a) loads from the equipment which occur in the start-up, transition and test modes, as also during its repositioning or replacement with limit or operational design values;

b) weight of people, repair materials in the areas of maintenance and repair of equipment with limit or operational design values;

c) loads from people, livestock, equipment on the floors of residential, public and agricultural buildings with limit or operational design values, except for the loads specified in *a, b, c, d* in the previous paragraph;

d) loads from mobile lifting and transport equipment, loaders, stacker cranes, hoist block, as well as from overhead travelling crane and underslung crane with limit or operational design values;

e) snow loads with limit or operational design values;

f) climatic temperature loadings with limit or operational design values;

g) wind loads with limit or operational design values;

h) ice loads with limit or operational design values.

The episodic loads include (p. 4.14 [2]):

a) seismic loads;

b) explosive loadings;

c) loads which are caused by sharp violations of the technological process, temporary malfunction or destruction of equipment;

d) loadings that are caused by deformations of the base, which are accompanied by a radical change in earth structure (when soaking subsidence soils) or its subsidence in areas of mine workings and karst areas.

Mechanical loadings, which are taken into account in the calculation directly, are considered as a set of forces applied to the structure (load), or as forced movements and deformations of structural elements. Other loadings of non-mechanical nature (for example, loadings of aggressive environment) are considered in calculation indirectly.

8.3. Design values of the loads and loadings

1. Types of design values of loads. DBN design codes [2] for the first time established several values of loads.

Characteristic value is the main (basic) value of the load, which is set in the design codes.

For each of the main loads and loadings the two main design values are set - ***the operating value and the limit value***, and for each emergency loading one limit value is set.

In addition to the main design values, additional schematized design values can also be set for the main influences. These values are related to an idealized model of their dependence on time and are designed to take into account special effects (creep, shrinkage, prestressing losses, fatigue, etc.). These design values are ***cyclic and quasi-permanent***.

1. ***The operating design value*** is the value of the load, which characterizes the conditions of *normal operation* of the structure. As a rule, the operating design value is used to check the boundary conditions of the second group, due to difficulties in normal operation (occurrence of unacceptable structural displacements, unacceptable vibrations and unacceptable large revealing of cracks in the reinforced concrete structures, etc.).

2. ***Limit design value*** is the value of the load corresponding to the *extreme situation* that may occur no more than once during the life of the structure, and it is used to check the limit states of the first group, exceeding which is equivalent to complete loss of structure’s function.

3. ***Cyclic design value*** is the value of the load, which is used for calculations of the structures for *durability* and it is defined as a harmonious process, which is equivalent for the resultant action to the real accidental process of the variable load.

4. ***The quasi-permanent value*** is the value of the load, which is used to take into account the rheological processes that occur under variable loads, and it is defined as the level of constant loading, which is equivalent in the resultant action to the actual accidental load process.

2. Design values of permanent loads. *Operating design values* of permanent loads G_{de} are assumed to be equal to their nominal values, which are set considering the geometric and other characteristics in the design documentation. For structures which are in operation this values are equal to their average values determined during the real tests.

The limit design value of the permanent load G_{dm} is set so that it cannot be exceeded with a given probability of P_G. It is assumed that the probability of overloading the limit value is a hundred times less than the probability of exceeding the operating design value.

3. Design values of the variable load. *The operating design value* of the variable load Q_{de} is set so that the possible action of a high intensity load does not exceed this value on average (e.g., 2 %). The time rate η is set in terms of effective use of the structure for its functional purpose.

For episodic variable loadings, the operating design value Q_{de} is not standardized.

The limit design value of the variable load Q_{dm} is determined under the assumption that it is not exceeded during the given time T with the given probability P_Q.

As a rule, the set working life T_{ef} is selected as T, and the probability of P_Q is assumed so that the value Q_{dm} can be exceeded on average not more than once during the T_{ef} time.

As a rule, in the codes of loads and influences the dependence of Q_{dm} on the repeated period T must be given.

4. Schematized design values of loads. These Q_{di} values are set depending on the properties of the real process of loading, which are significant for the phenomenon under consideration, and which can cause a failure of the structure.

To take into account long-term rheologic processes (shrinkage, loss of strength), the schematic *quasi-permanent* design value $Q_{di}(t) = Q_{di}$ is set, and to take into account the phenomenon of fatigue, the schematic *cyclic* design value in the form of a harmonic law with a characteristic frequency ω_{di} is set.

The value Q_{di} is determined under the assumption of equivalence of the results of the calculation for the action of the real loading process $Q_d(t)$ and for the action of the loading with idealized dependence on the time.

In necessary cases, the schematized cyclic value can be considered as one of the components of the total load (for example, the pulsatile component of the wind load).

5. Design value of the emergency load. The limit value of this load U_{dm} is set similarly to Q_{dm}; if it is necessary with a different probability of not exceeding the $P_U(T_{ef})$ of the set operating time. According to the value U_{dm} in the cods of loads and loadings the average frequency of appearance of such load or the probability of its realisation during the term T_{ef} is set.

8.4. Probabilistic description of the loads

Mathematical models of a random field, differentiated, Markov or pulsed random process, sequence of overloads, sequence of maximum values for characteristic time intervals, as well as other models that adequately reflect the real process of loading can be used for probabilistic description of variable load process.

Mathematical models of the accidental value or accidental field, which represents the space intensity of the pressure, are used for the normalization of permanent loads and loadings.

A specific probability model for the standardization of each load is selected taking into account the physical nature, the nature and specific features of the loading process, the nature and amount of available statistical information, the complexity of the standardization procedure and the accuracy of the evaluation of the design values of the loads.

In determining the design values of loads and loadings on the structures in operation, the actual values of the required parameters, the results of meteorological observations for a particular construction area, as well as data on loads and impacts on the structure which are obtained as a result of experimental and statistical studies are taken into account.

8.5. Application of the design load values

Variants of application. The classification of loads accepted in these codes corresponds to their physical nature and provides the possibility of calculation of various types of building structures, taking into account all the necessary design situations and limit states, namely:

a) verification of the strength, stability and other criteria of the loading capacity under a single load in the extreme operating conditions (emergency situation or stable and transient calculation situation, which can be realized a limited number of times during the service life);

b) verification of the rigidity and crack resistance in the mode of normal operation (stable calculation situation);

c) checking endurance limit under repeated loads (stable design situation);

d) taking into account the creep of materials and other rheological processes under permanent and long-term loads (stable design situation).

The above types of loads and design values should be used in accordance with *Table 8.1*. The letters indicate the types of calculations listed here, for which certain types of calculated values are used.

Table 8.1.

Application of the load types by the method of calculation

Design value	***The load types Види навантажень***			
	Основні Basic			***Episodic***
	Permanent	***Variable***		
		Long-term	***Short-term***	
Operating	*c, d*	*b*	*b*	
Limit	*a*	*a*	*a*	*a*
Ciclic		*c*		
Quasi-permanent		*d*		

Limit states of the first group. When checking the limit states of the first group, the limit design values G_{dm} of permanent loads and the limit design values $Q_{dm}(T_{ef})$ of variable and accidental loads are taken into account, which correspond to the established term of operation of the structure T_{ef}, as well as the schematized cyclic design values Q_{dc}, if they are the components of variable loads, which are considered.

Limit states of the second group. The loads for checking the limit states of the second group are set depending on the function and operational requirements to the structure under consideration:

- if the limit state of the second group can be allowed once in T_e years, then the operating design values Q_{de} of permanent loads are used, as well as the limit design values $Q_{dm}(T)$ of long-term and short-term variable basic loads are used, which are corresponding to the frequency period T;
- if during the operation of the structure exceeding the limit state of the second group can be allowed during a certain part of the specified service life of the structure T_{ef}, the operating design values Q_{de} of permanent loads as well as the operating design values $Q_{de}(\eta)$ of the variable basic loads corresponding to this part η are used for the calculation.

The nature of the test, as well as the values of T and η are established by the design codes of the structures by taking into account the function, features of work, operating conditions and operational requirements of the structures. For example, the periodicity of exceeding the stiffness standard T_n can be equal to the inter-repair period or another interval of time, typical for the mode of operation of the particular structure. The part of the set service life η can be determined on the basis of the required readiness factor or other operating parameters.

8.6. Standardization of the combination of loads and loadings

Formation of load combinations. The combinations of loads and loadings are formed as a set of their design values or forces corresponding efforts and / or displacements, which simultaneously affect on the object of calculation and which are used to the verification of the structure for the conditions of the specified limit state in a certain design situation. The combination includes loads that can physically act simultaneously and have the most adverse effect on the structure in terms of the limit state under consideration. Loadings that mutually exclude one another cannot be part of the same combination.

Two types combination of loads can be used in the calculation of the structures:

- ***basic***, which are used to verify reliability in established and transient calculation situations;
- ***emergency***, which are used to verify reliability in emergency calculation situations.

To check the limit states of the first group, the main combinations of the permanent loads with limit design values; limit design values, cyclic or quasi-permanent values of the variable loads are used.

To verify the limit states of the second group, the basic combination of permanent loads with operational design values, as well as operational design values, cyclic or quasi-permanent values of variable loads are used.

The method of taking into account combinations of design values of repeated variable loads or design values of schematic cyclic loads should provide the ability to determine the value of the total load effect, as well as the frequency, periodicity or probability of its realization.

Factors of the load combination. Reduced probability of simultaneous action of several random loads is usually taken into account by multiplying the sum of load effects from the action of design values of all loads by the total factor of the load combination $\psi \leq 1$. It is also allowed to use separate factors of the load combination for individual types and groups of loads and their combinations (for example, factor of the total crane loads combination or factor of the total permanent loads combination).

The basic combinations of loads include permanents loads and at least two variable loads, the last ones are taken with factor of ψ_1 = 0,95 for long-term loads and ψ_2 = 0,90 for short-term loads.

For the emergency combinations of loads which include permanents loads and at least two variable loads, the last ones are taken with a factor of ψ_1 = 0,95 for long-term loads and ψ_2 = 0,80 for short-term loads. The emergency load is accepted with the factor of loads combination ψ_1 = 1,00.

Note. *In the basic combinations, when taking into account three or more short-term loads, their design values are allowed to be multiplied by the factor of the combination ψ_2, which is taken for the first (for the level of loading) short-term load is 1.0, for the second is 0.8, for the rest is 0,6.*

While choosing the most unfavorable combinations of loads and loadings for one short-term load should be taken:

a) load from one source (pressure or discharge in the tank, components of snow, wind or ice load, load from one loader, one crane, etc.);

b) the load from several sources, if their combined action is taken into account in the value of the load (load on the floor, which is determined by taking into account the factors ψ_d or ψ_n; load from several cranes by taking into account the factor ψ; ice-wind load which is determined according to section 10 [2]).

8.7. Emergency combination of loadings

Apart from the basic loadings, only one emergency load can be a part of the emergency combination of loads. In this case, the load effect from the most dangerous emergency load in the given calculation situation is added (possibly taking into account the appropriate factor of combinations) to the total load effect of the main loads, that are determined by taking into account their factors of combinations.

Verification of the emergency design situation may be performed on the basic load combination, but taking into account the repair or relaxation of the structure as a result of the action of the emergency loading (for example, decreasing the load-bearing capacity of the structure as a result of the action of fire or failure of some elements during the explosion).

Consideration of emergency situations requires clarification of new concepts which are introduced into the code document [1]. Such concepts are ***design emergencies (DE)***, for which the design should be provided with special equipment for active management and protection. The list and the main parameters (fire load, explosion force, flood level, etc.) are determined by special standards on the basis of the establishment of possible social and material damages and losses with the necessary devices for their prevention.

In addition to the DE parameters, the parameters of the ***maximum possible disaster (MPD)*** of natural and (or) technogenic nature are set for a particular object. Methods for determining the MPD and its parameters are also set by special codes.

It is allowed to accept the parameters of the MPD based on the probability of their occurrence 100 times less than the accepted probability of occurrence of the DE.

During the development of special codes and determination of the parameters of the DE and MPD, the phenomena that can be caused by the following initial events are considered:

- catastrophic exceedances in the intensity of natural loadings of the level that is established by the current codes for the area of construction;
- technological disasters (vehicle accidents, explosions, fire, molten metal, etc.) that occur within the object or in its closest vicinity;
- gross mistakes of the personal at the stages of design, construction or operation of the object;
- serious defects or gross discrepancy characteristics of construction materials and products, elements of the equipment to the requirements of the standard and technical documentation.

During consideration and classification of the listed causes of the DE and MPD, the loading of secondary factors (explosions, occurrence of fire, collapse of protective barriers, impacts from falling elements, etc.), which caused the primary accident, should also be taken into account. It is recommended to develop and analyze scenarios of accident development.

Depending on the category of structures and elements, the requirements for functioning in emergency situations and safety measures according to *Table 8.2* must be ensured. In this case, the design failure is considered taking into account the influence of secondary factors and in combination with a single failure of the safety elements, which is independent of the causes of DE, or with a single gross error of staff, which is independent of the causes of DE.

Table 8.2

Requirements for operation and safety in emergency situations

Incident number depending on the situation	***Loadings that have to be taken into account (+)***					***Requirements that are set for the elements of the category***		
	from normal operation	***DE***	***MPD***	***one failure of elements of protection***	***one staff mistake***	***A1***	***A***	***B***
1	+					*F*	*F*	*F*
2		+		+		*F*	*B*	*R*
3		+			+	*F*	*B*	*R*
4			+			*B*	*B*	
Notations*: F - it is necessary to ensure full functionality;* *B - all safety-related functions must be ensured;* *R - it is necessary to ensure the possibility of renovation by repair*								

8.8. Probabilistic study of load combination

The value of combination factor, as a rule, is established by probabilistic methods of the condition that the total loading effect and the design values of individual loads are equal; it depends on the type of the loads that are taken into account and their parts in the sum of the total loading effect.

Today the recommendations of the codes do not have a comprehensive and probabilistic concept, they include the value of factors of combination without taking into account the role of that or another load, that is part of the combination and how much it contributs to the overall stress-strain state. At the same time, the ratio of the loads that act on the structure should influence the factor of combinations. This was confirmed, for example, in the research of K.S. Lositskaya (TsNIISK, Moscow) [3], the results of which are illustrated by the graph in *Fig. 8.1*. Here p represents the part of wind load in the combined effect of wind and snow load. The graph clearly shows that the factor of combinations, is close to one with the predominant action of one load, that is when $p \approx 1{,}0$. The factor of combinations gradually decreases to $\psi \cong 0{,}75$ at approximately equal action of both loads $(p \approx 0{,}5)$.

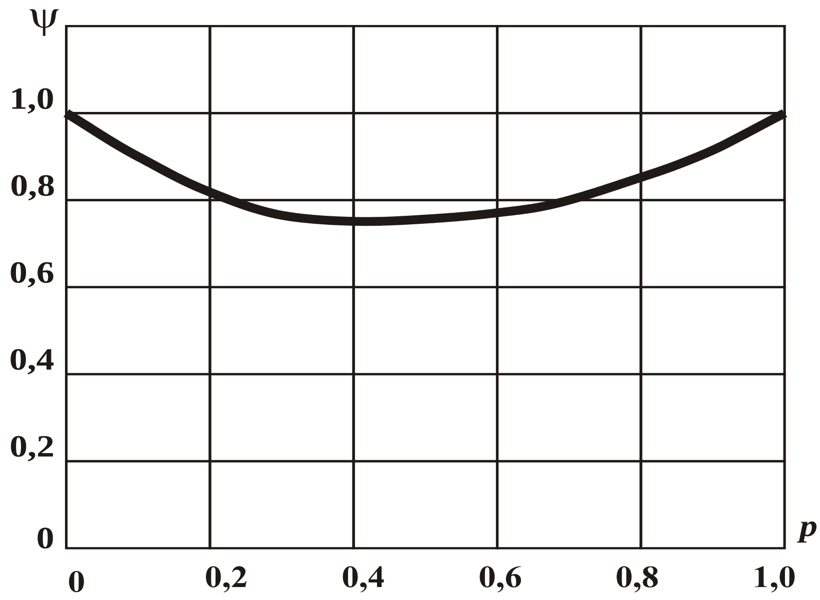

Fig. 8.1. Combination factor of wind and snow loads

V.A. Pashinsky considerably developed this problem by solving the problem of combining wind, snow and crane loads [4]. The obtained results are presented in *Fig. 8.2* in the form of diagrams, which shows the dependence of the factor of combinations on the particles C_B, C_C and C_K of wind, snow and crane loads in the combined action of these loads.

As can be seen in *Fig. 8.2*, the factor of combinations can significantly decrease to the value $\psi \cong 0{,}725$, in comparison with the value $\psi = 0{,}90$, which is regulated by the current codes [2].

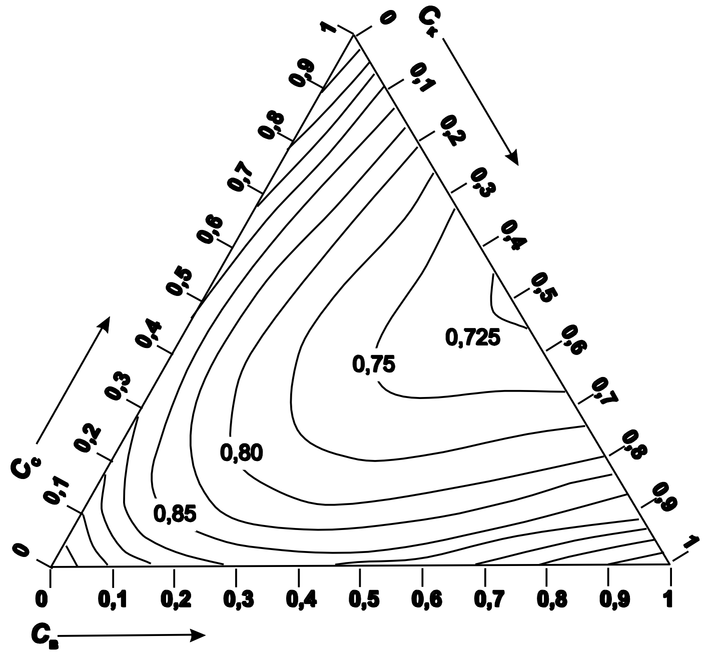

Fig. 8.2. Combination factor of wind, snow and cran loads

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Control questions

1. How are the loads on the structure classified?
2. How are the loads divided depending on the duration?
3. What are the design values of loads and loadings?
4. How are the design values of loads applied?
5. How are the combinations of loads and loadings standardized?
7. How is the emergency combination of loads formed?
8. Probabilistic nature of loads and combinations of loads.

LECTURE 9. NATURE AND DESCRIPTION OF SNOW LOAD

9.1. Snowfall is a dangerous natural phenomenon

Snow cover plays a significant positive role in human life. It is of great importance in the formation of climatic and hydrological regimes of areas. Its role in agriculture is especially important as a factor that provides soil moisture and protection from winter frosts and other crops. Due to the low thermal conductivity of snow, the snow cover protects the soil from deep freezing, the presence of snow on the ground provides storage of wintering plants and plant seeds in the soil. It is no coincidence that the popular saying goes: "Snow in the fields is a harvest in the barns."

At the same time, heavy snowfalls disrupt traffic, block the work of municipal services, break trees, destroy buildings. It is known that every winter comes unexpectedly, and heavy snowfalls are often catastrophic (*Fig. 9.1*). The description of numerous force majeure situations related to snowfalls is contained in the monograph [3]. Some of them are presented here.

Fig. 9.1. Snow drifts and traffic jams

February 2006. The heaviest snowstorm in the northeastern United States, the strongest in decades, was followed by winds of up to 100 kilometers per hour. More than 200,000 people were left without electricity due to power outages in various states. Airports were closed, more than two thousand flights were canceled. The mayor urged residents not to leave home unnecessarily. Many citizens skied the streets. In New York, there were continuous snowplows that cleared the city at a rate of 60 tons of snow per hour, which cost the city a million dollars a day. In Central Park, the thickness of the snow cover reached 69 cm, which was a record since 1869, when meteorologists began to conduct similar observations. Heavy snowfall also occurred in the Czech Republic. In the mountains in the north of the country there was a three-meter layer of snow. There is a danger of an avalanche at ski resorts. In Austria, a layer of snow fell to 90 cm, more than 20 thousand people were mobilized to remove snow from the roofs.

February-March 2009. Heavy snowfall in Western Europe killed seven people. Difficult weather conditions disrupted the work of European airports, led to chaos on highways and railways. Thousands of passengers spent the night at train stations, airports or their own cars on the highway due to bad weather. In the UK, the thickness of the snow cover reached 30 cm, in London the heaviest snowfall was recorded in 18 years. The capital's Heathrow Airport had to cancel 650 flights; city buses stopped.

As a result of snowfall in Spain and France, many roads were blocked due to many kilometers of traffic jams. In Bulgaria, heavy snowfall completely paralyzed road traffic. Huge queues of heavy trucks and buses lined the roads of international importance. During the day, there were about 200 car accidents in the country, in which 17 people were injured. The height of the snow cover in Sofia was 60 cm (*Fig. 9.2*).

In the United States, heavy snowstorms killed 55 people. In the eastern states, the height of snow cover reached 40 cm. In the US capital, a state of emergency was declared. Half a million homes were left without electricity in Kentucky, and 350,000 consumers were left without electricity in Oklahoma and West Virginia. Schools, universities and government agencies were interrupted. Hundreds of flights were delayed, and thousands of passengers were stranded at airports. Traffic on many highways has stopped, huge traffic jams have formed.

Fig. 9.2. Snow drifts on the roads

December 2009 - January 2010. A sudden cold snap and heavy snowfall have killed more than 80 people in Europe. Such weather instantly paralyzed Europe's transport system. In the tunnel under the English Channel 3 trains were stoped. Most European roads have been covered with snow so much that any traffic on them has become dangerous. Record traffic jams with a total length of 500 km were formed on the roads of Belgium. Hundreds of flights were canceled at international airports, and thousands of people were forced to spend from a few hours to several days at airports.

Ukraine has suffered from snowfalls and colds no less than other European countries, especially Odesa, Kherson and Donetsk regions, where the monthly winter rainfall fell in a few days. In Odessa, where the height of the snow cover reached one meter, a state of emergency was imposed, all schools, the international airport and seaport were closed, traffic was paralyzed, and there were interruptions in food supplies. More than a thousand cars got stuck on the Odesa-Illichivsk highway in a five-kilometer traffic jam. A state of emergency was also declared in the Donetsk region, bus stations, the airport were closed, and passenger traffic was suspended. More than one and a half thousand cars were trapped on the Donetsk highway.

9.2. Snow load is a danger to buildings

Particular danger is the snow cover and its uneven deposits on the roofs and floors of buildings, where there are a large number of people and expensive technological equipment. The snowy winters of recent years have been marked by the collapse of various buildings in a number of countries, which have had serious and even tragic consequences.

January 1978. The largest accident in the history of the United States was occurred in the city of Hartford (Connecticut, USA), which was caused by snow overload. In the city center, where a hockey match was held during the day, the coverage of the sports arena measuring 92 by 110 m collapsed at night from a height of 30 m. The investigation revealed errors in the calculations: the structure was actually designed for a load of 0.864 kPa, while the specified design load was equal to 1.104 kPa. As a result of increased snow loads, the compressed elements of the upper belts on the northern and southern sides of the pavement were overstrained by 852%, on the western and eastern sides - by 213%. The load on the compressed bevels of the coating structure by 72% exceeded their bearing capacity.

January 1987. In Zaporizhia (Ukraine) the roofs of four factories fell in one night. The accident at the Zaporizhstal charge yard was especially troublesome - when the roof fell, it broke the bridge cranes and blocked the way for the delivery of scrap metal. Martina was on a starvation ration. Every day Zaporizhstal did not add 1,000 tons of products. The failure in the charge yard resulted in huge losses for many recipients of Zaporizhzhya metal.

Fig. 9.3. Collapse of the sports complex
(Bad Reichenwal, Germany)

Fig. 9.4. Destruction of the pavement of the trade and exhibition pavilion
(Katowice, Poland)

February 2004. The biggest construction accident in Russia in recent years took place in Moscow - the cover of the Transvaal Park water park collapsed. 1,300 people were in the water park at the time of the disaster, 426 people were directly at the epicenter of the collapse. As a result of the tragedy, 28 people died and 190 people were injured. One of the reasons for the collapse of the experts was the effect of snow load on the surface, which was calculated according to low standards, later increased for Russia. In addition, the low quality of the construction materials (in particular, concrete) and the production of works, as well as the miscalculations of the author of the project in the design of a unique thin-walled reinforced concrete shell of the water park.

January 2006. The roof of a sports and entertainment complex built in the 1970s collapsed in the Bavarian town of Bad Reichenwald, Germany. A section of the flat roof collapsed on the skating rink, where at least 50 people were at the moment, all of them were trapped. Fifteen people died, including children. The reason for the collapse - a half-meter layer of snow on the roof (*Fig. 9.3*).

In the city of Katowice (Poland) the roof of the trade and exhibition pavilion with an area of 10 thousand m^2 collapsed (*Fig. 9.4*). The President of Poland Lech Kaczynski called this accident the greatest man-made catastrophe in the recent history of the republic. At this time, the pavilion hosted a traditional international exhibition of carrier pigeons, which many visitors came to see. About 1,000 people were trapped, the death toll exceeded 100 people.

The main cause of the disaster was a thick layer of snow that formed on the roof, with ice thicker than a meter. This situation was facilitated by the fact that, according to experts, during the construction boom that engulfed Poland in years before the current crisis, the construction of large commercial centers used projects developed in countries with warm climates.

Fig. 9.5. Typical appearance of snow crystals under a microscope

9.3. Formation of snowfall

The formation of snowfall in the atmosphere depends on many factors, but mainly on the ambient temperature and the presence of supercooled water. Initially, as a result of condensation of water vapor in the rising warm air mass, a cloud is formed. As soon as the temperature in the cloud drops below 0ºC, conditions are favorable for the formation of snow. At a temperature of about - 5 ºC, the crystallization nuclei present in the atmosphere form small ice crystals.

The formation of a snow crystal begins with an ice crystal. All snow crystals usually have a hexagonal structure, they are never 5-carbon or 7-carbon. New particles grow on all six vertices, and over time a beautiful openwork pattern is obtained, which *Fig. 9.5* confirms. Thousands of snow crystals in the process of growth and complication form a snowflake. The shape and size (0.1… 2 mm) of snowflakes depend on air temperature. No snowflake repeats another, the shape of each is unique. Snowflakes have a low density, their weight averages 1 milligram, rarely 2 milligrams, their rate of fall is relatively small - an average of 0.9 km/h. The number of snowflakes falling on the planet per year was calculated. There are on average 350 million snowflakes in one cubic meter of snow, and 10^{24} snowflakes all over the Earth.

The air gives white snowflakes, the content of which is 95%. There are cases when the snow has acquired a different color than white. Thus, in the Arctic and mountainous regions of Russia, snowflakes are often pink or red, which is caused by algae between the snow crystals. In 1969, black snow fell in Sweden, apparently absorbing soot and industrial pollution from the atmosphere. In 1955, phosphorescent green snow fell in the state of California (USA), the origin of which has remained a mystery to scientists.

Concluding the description of the physical nature of snowflakes, we note that they clean the air from dust and fumes, so it is so easy to breathe during a snowfall. Moreover, the snow reflects the dangerous spectra of sunlight, so the northern peoples do not have many diseases that afflict the inhabitants of the south. Therefore, we can talk about a certain snow medical geography, little studied so far.

9.4. Snow cover on the territory of Ukraine

The formation of snow cover in Ukraine is associated with a general seasonal decrease in air and soil temperatures, as well as snowfalls caused by the invasion of Arctic air masses and their interaction with the air of temperate latitudes [6].

The earliest (in the first decade of November) snow cover is established in the northeast of the country and in the mountainous regions of the Ukrainian Carpathians. In the second decade, snow cover is established in the forest-steppe, mountainous regions of Crimea and Precarpathia. By the end of November, snow usually covers most of the country. In the south, snow cover appears later: in the Black Sea lowlands and in Transcarpathia - in the first decade of December, in the Crimea - in the second decade of December. These terms are average, because depending on the characteristics of weather processes, the appearance of snow cover in some years varies significantly. With the early onset of winter snow cover can appear almost all over Ukraine in early November, and in years with warm autumn snow cover is established only in the first or second decades of December. The period of time between the appearance of snow cover and the formation of stable snow cover before the winter is on average one month, but in some years, it may increase to 100 days or be absent when stable snow cover throughout Ukraine is established with the first snow (for example, the winter of 1963/64).

Stable snow cover is formed first of all (in the first half of December) in the Ukrainian Carpathians and in the north-east of Ukraine. In a large part of the country, stable snow cover is established in the third decade of December, in the southern regions and Transcarpathia - in the first decade of January. In some years, stable snow cover occurs in the second or third decades of November (almost a month before the average dates) or later than usual - in the first decade of January. Stable snow cover lies on the territory of Ukraine for about 60… 70 days, and in the direction from east to west the duration of its occurrence decreases. In the areas of Volyn-Podilska Upland, Donetsk Ridge and in the mountains of Crimea the duration of stable snow cover increases up to 80 days, in the mountainous areas of the Ukrainian Carpathians it increases to 100… 110 days. The same duration of stable snow cover is observed in the north-east of Ukraine. In some winters, the period of stable snow cover can be reduced to 20… 30 days or extended to 130… 140 and even 150… 160 days. At the same time, Ukraine had years without stable snow cover (for example, the winters of 1965/66, 1974/75 and 2021/22).

In the spring, with the increase in air temperature, the destruction of stable snow cover begins. First of all, it happens in the far west of the country, in Transcarpathia and Precarpathia in late February - early March, in the mountainous regions of Crimea - in the first decade of March, later - in the second decade of March - in the north-eastern regions. The period between the destruction of stable snow cover and the final snowfall is called post-winter or pre-spring. In Ukraine, it is short-lived and is about 15 days in the northern and eastern regions and 25 days - in other areas. However, the post-winter may increase to 100 days. At the same time, in some years the time before winter and post-winter are not observed, ie winter comes and ends in a short period. The rise of snow cover is observed first of all in the Steppe Crimea and in the south of the Black Sea lowland - at the beginning of the second decade of March. At the end of the second decade of March, snow falls in the far west. In most parts of the forest-steppe and Polissya snow cover falls by the end of March, and in the northeast and in the mountainous regions of the Ukrainian Carpathians - in early April.

The period between the appearance and rising of snow cover is called the duration of snow cover. As can be seen on the map of *Fig. 9.6*, the average number of days with snow cover in the plains of Ukraine is 30… 100, in the northeast and in the mountainous regions of the Ukrainian Carpathians - 100… 110, on the Donetsk ridge and in the Crimean mountains - about 90, in western Ukraine - less than 80, in the Black Sea lowlands and in the Steppe Crimea - 30… 40, on the coast - about 20. In some winters, deviations from the average can reach large values. Thus, in the northeast the number of days with snow cover can vary from 40 to 160, and in the south and west - from 2 to 100 days.

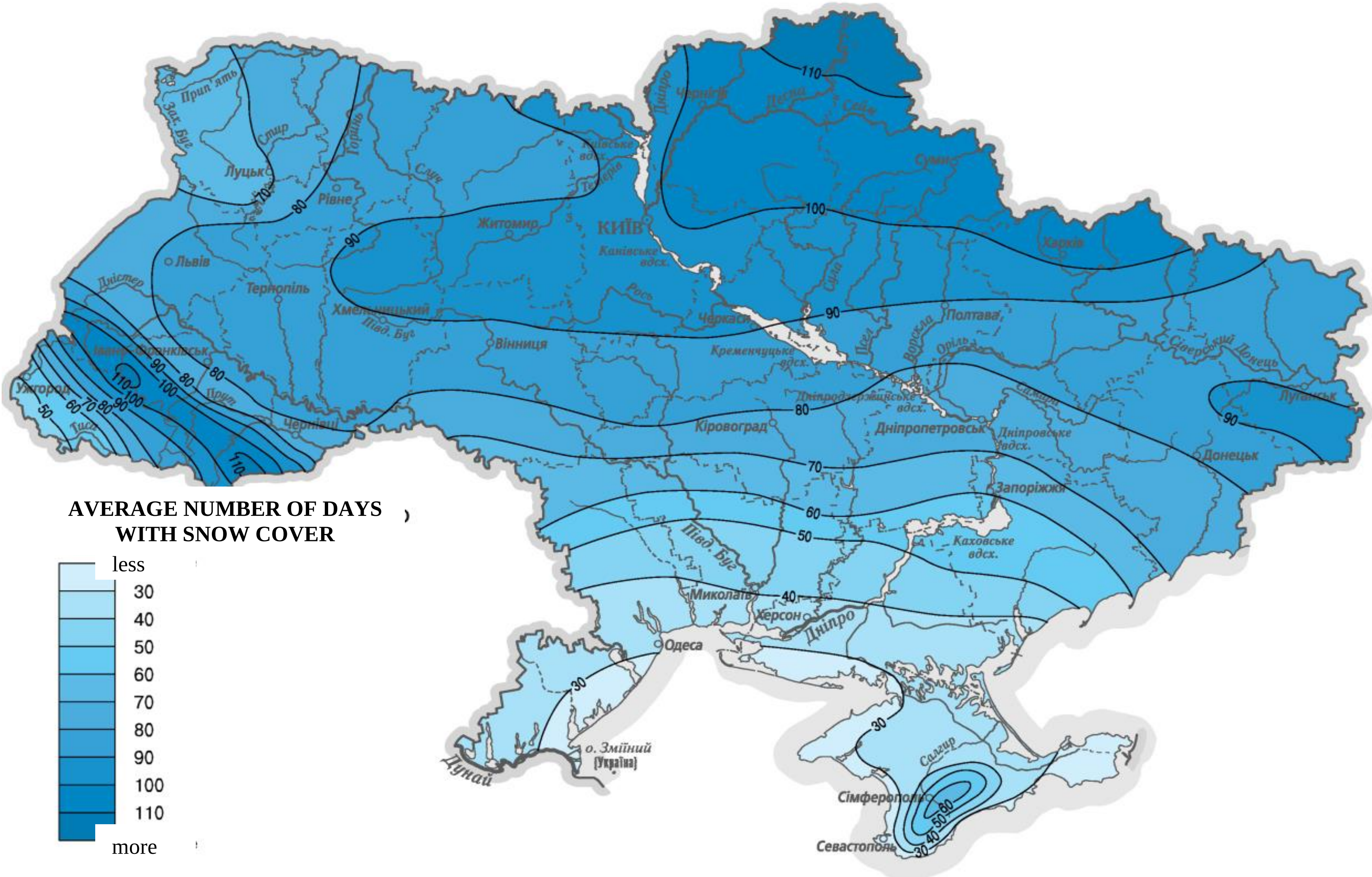

Fig. 9.6. The average number of days with snow cover for the territory of Ukraine

9.5. Regional features of snow cover (Poltava)

The snow climate of Poltava, located in the central part of Ukraine, which belongs to the II district according to SNiP [2] and to the 5th district according to the codes of Ukraine [1], can be considered typical for most of Ukraine and the CIS. The period of observations of snow cover, the results of which are discussed below, is 80… 90 years (1886… 1979) [5].

In Poltava, snow cover can be observed from October to April. The average date of its appearance is November 17, with the early onset of winter, this date can be shifted by a month or more (once every 20 years - until October 16, in some years the snow was observed in September). At the same time, in years with a long warm autumn snow cover appears in the second decade of December. The time period before the onset of winter averages 36 days. The date of formation of stable snow cover, which falls on average in the third decade of December, varies from year to year depending on the peculiarities of the circulation of the pre-winter period (the standard is 23 days). Snow melting and destruction of stable snow cover occurs on average at the end of the first decade of March (the standard of this date is 22 days), and on average on March 25 the snow disappears completely. Once in 20 years the snow cover can fall until February 22 or April 19, in some years the snow was observed even in May.

The average period between the appearance and rising of snow cover is 125 days and in same years, it can be from 68 to 190 days. The average number of days with snow cover in the city is 87 days, ranging from 34 to 143 days. On average, there are 10 days of snow per month. Stable snow cover lasts in the city for about 74 days, with significant fluctuations in some winters from 28 days to 143 days. At the same time in rather rare (recurrence of 10%) especially warm winters stable snow cover is not formed at all.

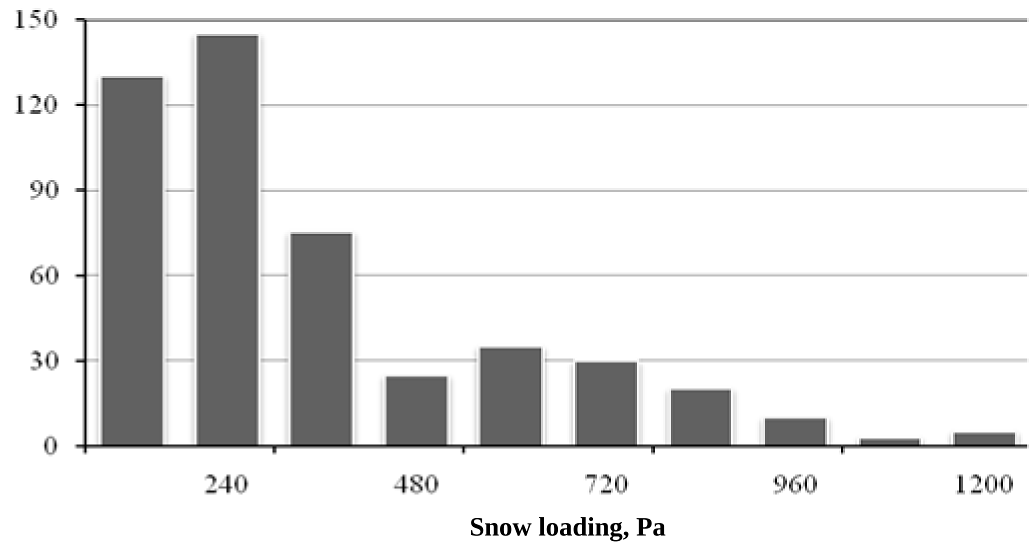

Fig. 9.7. Histogram of snow load
(Meteorological station Poltava)

During the winter, the height of the snow cover increases gradually. In November-December it is less than 5 cm, in January it begins to grow and in February-March it reaches an average of 11 cm. The average height of snow cover is 20 cm (46% of winters), in some years height is 48 cm (recurrence 10%). The density of snow cover varies slightly during the winter: from 0.23 g / cm^3 (second decade of February) to 0.27 g / cm^3 (first decade of March). The average density for winter is 0.34 g / cm^3, the maximum density for the observation period is 0.50 g / cm^3.

The interval of snow load change on the ground is illustrated by the histogram in *Fig. 9.7*. The average of the largest snow loads in winter (water supply) is 53 kgf / m^2 (0.55 kPa), in some winters the snow load exceeded 100… 110 kgf / m^2 (1.0… 1.1 kPa).

9.6. Collection of initial data on snow cover

Snow cover is formed as a result of accumulation of snow on the ground in the process of deposition of solid precipitation (snowflakes, icy rain, hoarfrost and ice), rainfall, when most of the precipitation later freezes. Information on snow cover is essential for various sectors of the economy, especially for construction. Such information is the result of snow surveillance observations that allow to assess the climatic regime of the territory, patterns of formation and distribution of snow cover in the area. Snow deposits are characterized by the thickness of the snow that has just fallen, formed during a specific period. According to individual values of snow thickness, the total snow accumulation per unit snowfall, day, month or year is determined. The total amount of precipitation is equal to the sum of the layers of liquid and solid precipitations.

1. System of snow gauge observations. The existing system includes the following types of observations:

- daily determination of the coverage degree of the visible part of the meteorological station with snow and the nature of its occurrence;
- daily observations of changes in the height of snow cover on permanent rails;
- snow surveys on various elements of the landscape;
- control snow measurements (when choosing and replacing permanent routes);
- special observations on the distribution and thickness of the ice crust on the soil surface;
- snow gauges at stock stations;
- daily observations of solid precipitation;
- snow gauge observations in the mountains along the routes;
- observation of avalanches.

Regular snow surveying has been carried out at meteorological stations since 1924. At a fairly large site, the conditions of which are typical for this area, 100 measurements of snow cover height are performed and 10 samples are taken to determine the density of snow. As a rule, snow surveys are performed in the field (area exposed to wind), in a forest glade or in the woods under tree canopies (areas protected from the wind). In areas with stable negative winter air temperatures and stable snow cover, snow surveys are conducted every decade, and in areas with unstable snow cover (for example, in Ukraine) - every 5 days. Surveying is performed when the research area or route is covered with at least 50% snow. Some meteorological stations conduct snow surveys in ravines and in mountainous terrain.

The accuracy and scope of ground assessments of snow cover, especially in hard-to-reach areas, are increased by aerial methods, including aerial photography and aerial visual observations. Thus, from the plane character of occurrence of a snow cover, its condition and height (approximately on percentage of closedness and on height of a shadow of a rail established on an open equal place) are defined. Useful experience has been gained by Japanese researchers who have successfully used airships and helicopters for snow measurements. An important role belongs to meteorological satellites, which together with other meteorological elements create a picture of the distribution of snow cover in some parts of the world, especially in those where there are no systematic observations of snow cover.

a b

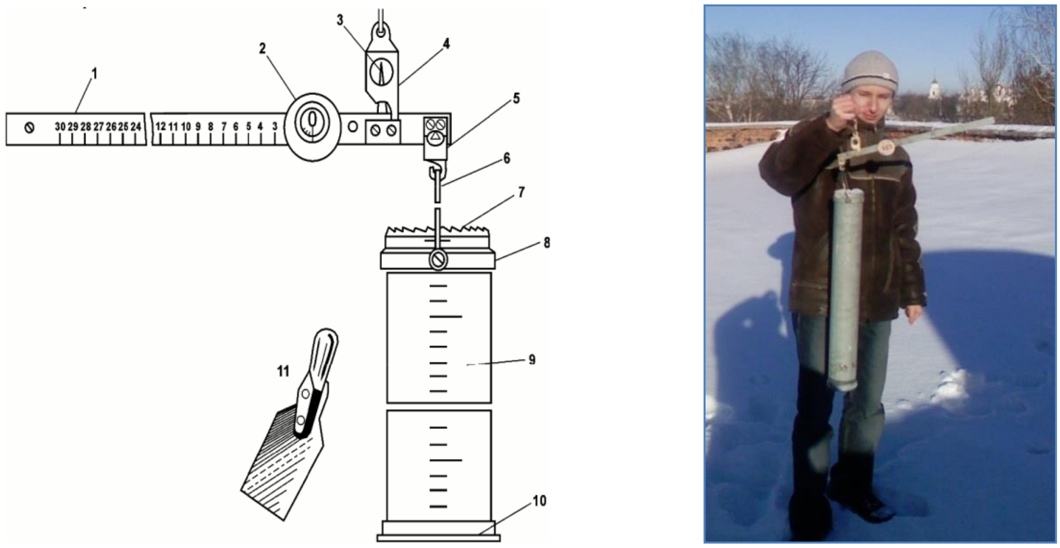

Fig. 9.8. To the method of snow gauge observations
a - weight gauge BC-43: 1 - brass line of scales, 2 - moving cargo; 3 - arrow,
4 - handle, 5 - hook for hanging the cylinder, 6 - bracket,
7 - ring with a sharp edge, 8 - movable ring, 9 - cylinder, 10 - cover, 11 - blade;
b - measurements (graduate student Dryzhiruk YV)

2. Devices for snow gauge observations. Since 1922, meteorological stations in the USSR and then the CIS have been measuring snow density with a weight snow gauge. The device consists of a cylinder with a lid, balance and a blade. A centimeter scale is applied to the cylinder to measure the height of the cut snow column (*Fig. 9.8, a*). The cylinder cuts the snow cover through and thus forms a core that includes all layers of snow cover.

The cylinder is suspended from the balance, on the line of which is a scale with divisions in grams. The height and weight of each sample on a place are noted, then density of snow is calculated (*Fig. 9.8, b*). This determines the average density of snow cover at this point (averaging over all layers that actually have different densities). If the height of the snow cover is less than 5 cm, density samples are not taken, so the result of such snow removal is only the average height of the snow cover. The method of performing snow gauge surveys is regulated by the guidelines [7].

3. Snow gauge network. According to the described method, snow cover is observed at meteorological stations and posts. There were about 4,000 meteorological stations and more than 6,000 meteorological stations throughout the former USSR, and about 200 meteorological stations and more than 400 meteorological stations in Ukraine. To objectively assess the parameters of the snow cover, it is necessary to have such a density of the snow gauge network that one station accounted for an area of 3… 6 thousand km^2; in forest areas - by 8… 10 thousand km^2. For the entire territory of the former USSR, one station (post), where snow observations were made, accounted for an average of 3234 km^2. The most fully studied areas of Ukraine, where one station is per 1000… 1200 km^2.

It is useful to know that there are about 7,000 meteorological stations worldwide, of which 3,800 are part of the World Meteorological Organization (WMO) network, and the need for meteorological information was met by only 20%. Snow surveys are performed in all countries with snow cover. In this regard, the United States stands out, where systematic surveys have been conducted since 1910 and there are more than 1,000 routes for monthly snow surveys. The average density of the US snow gauge network is one station per 600 km^2. At the same time, in Canada, one station accounts for an average of 3,600 km^2, and in half of Canada's territory, one station accounts for 12,500 km^2.

For climatological generalizations and further transition to substantiation of design snow loads, in addition to the density of the snow gauge network, the duration of the observation period at stations and posts is of great importance, which as of the early 70's differed significantly at different stations:

- more than 25 years - at 3% of stations;
- from 15 to 25 years - for 50% of stations;
- from 10 to 15 years - at 47% of stations.

It is obvious that so far, the volume and duration of snow surveillance observations on the existing network of stations have increased significantly.

However, a number of stations have remained short-lived, and their snow survey materials can only be used to estimate snow load or as additional data to long-range stations and permanent rail observations lasting more than 50 years.

4. Initial snow data. The first results of snow surveys began to be published in the late 1930s; quite complete continuous data on most observation points can be obtained from 1945. Materials of daily snow surveys on the main sites were published in meteorological monthly magazines of regional departments of hydrometeorological service [8, 9], results of snow surveys (after technical and critical analysis) were published in meteorological yearbooks [10, 11], as well as were accumulated on electronic media. Quite voluminous generalized results of observations of snow cover are also published in the Climate Handbook of the USSR [12]. These meteorological directories have 34 issues corresponding to certain territories of the former USSR. For example, data for the territory of Ukraine form issue 10. After the transition to market relations, free access to data on snow observations has virtually ceased. Useful information is contained in climate atlases [13, 14].

5. Taking into account the type of meteorological sites. As mentioned above, regulations stipulate that measurements of snow cover should be performed on well-protected sites near the construction site. Meanwhile, a significant part of meteorological sites is open to wind. For example, in Ukraine over the past 70 years, regular measurements of snow load on the ground were performed at 1145 meteorological stations and posts, of which 618 - open areas, 466 - sites protected from wind and 60 - sites for which no information is available [4]. Thus, most meteorological sites in Ukraine (probably too for Europe) are open to wind.

R.I. Kinash (Lviv Polytechnic National University) compared the results of snow load measurements at "pair" stations, where measurements were performed simultaneously on closed and open sites located nearby [4]. In total, data from 6-year measurements at 83 sites were obtained and processed. The results of measurements showed that the snow load on the open areas is on average slightly less than the load on the closed areas, but a fairly close linear correlation was found between the data of different sites. Approximation of this dependence allowed to describe the relationship of snow loads in closed areas S_c with loads in open areas S_{op} linear dependence of the type $S_c = k\, S_{op}$, where the transition factor is for average values $k = 1{,}164$, for average annual highs $k = 1{,}185$.

9.7. Climatic characteristics of snow cover

Height of snow cover. The height of snow cover significantly depends on weather and climatic conditions and local factors, including the duration of snow, frequency and amount of snowfall, density and structure of snow, snow transfer and accumulation, protection of the area and more. Therefore, to obtain relatively stable values of the height of the snow cover, which has high variability, you should use an averaging period of at least 7… 10 years. According to the thus obtained probabilistic values of snow cover height at different points, an assessment of the nature of their distribution is made depending on the physical and geographical conditions. However, the height of the snow cover does not allow to give an objective assessment of the snow load on the ground, for which it is also necessary to assess the density of the snow cover.

Density of snow cover. This important climatic snow parameter is also a variable, both in time and space. It differs for different observation points, both in the plains and especially in the mountains, and varies even in a single snow gauge point. The density of snow depends on such factors as wind speed, air temperature, height of snow cover, duration of its occurrence. It is accepted that for calculations of snow loading on constructions the density of snow in a decade of the greatest height of a snow cover is considered.

Numerous calculation models have been developed to determine snow density. The relatively simple stochastic model used, in particular, in Germany, has the form [15]:

$$\tilde{\rho}_{\Gamma} = [3 - 1{,}5\, exp(-1{,}5 m_H)]\varepsilon_{\Gamma}, \tag{9.1}$$

where m_H is the average thickness of snow cover on the ground, expressed in meters; ε_{Γ} is a factor that takes into account the error of estimation and represents a lognormal random variable with an average value $\varepsilon_{\Gamma} = 1$ and a coefficient of variation $V_{\Gamma} = 0{,}2$.

An approximate forecast of changes in the density of snow cover (kg / m^3) during the winter season can be made using the following empirical relationship [16]:

$$\rho = 152 - 0{,}31T + 1{,}9u, \tag{9.2}$$

where T is the average air temperature (° C), u is the wind speed (m / s).

Since T and u are determined by the terrain, the above formula takes into account a significant change in the density of snow cover in the field.

Detailed recommendations for determining the density of snow cover (kg / m^3) were received by TsNIISK (Moscow):

$$\rho = \left(90 + \sqrt{h}\right)\left(1{,}5 + \sqrt[3]{T}\right)\left(1 + \sqrt{v}\right), \tag{9.3}$$

where h is the height of the snow cover (m); T – average temperature (˚C) for the period of snow accumulation (taken not lower than - 25˚C); v is the average wind speed (m / s) for the same period.

The codes of SNiP [2] and DBN [1] do not provide recommendations for determining the density of snow. Therefore, we note that the codes of Eurocode [17] regulate several values of the bulk density of snow. The density of freshly fallen snow is assumed to be 1.0 kN / m^3; if several hours or days have passed since the snowfall, the specified density value must be doubled; if the "age" of fallen snow is calculated in weeks or months, the snow density should be taken equal to 2.5… 3.5 kN / m^3; finally, the density of wet snow should be considered equal to 4 kN / m^3. In addition to these values, Eurocode separately provides for the values of snow density in snow bags: 2 kN / m^3 and 3 kN / m^3.

Detailed statistical data on the density of snow cover for the territory of Ukraine is given by R.I. Kinash in the capital reference book [4]. The long-term snow gauge results presented in this publication for one of the plain meteorological stations (Semenivka village, Chernihiv region) (*Fig. 9.9, b*) clearly illustrate the temporal variability of snow density, which varies for this point in the range from 0.10 to 0.50 g / cm^3.

The parameters of snow cover density have a significant territorial variance for the territory of Ukraine: average density is 0.200 ... 0.267 g / cm^3, coefficient of variation V = 16.71%, maximum density is 0.270… 0.700 g / cm^3.

9.8. Determination of snow load on the earth's surface

Long-term snow gauge observations allow to reliably estimate such an important parameter for the normalization of snow load as the *water supply* in the snow cover. In meteorology it is usually expressed in millimeters, it is numerically equal to the equivalent thickness of the layer of melt water.

Perhaps the origin of this term is explained primarily by agronomic and hydrological orientation of meteorological observations of snow cover: accumulated by the end of winter water supply in the snow cover determines soil moisture and fertility, as well as the magnitude of spring floods. So far as the density of water is equal to one, from the value of the water supply it is easy to move to the weight of snow cover per unit area, ie to the standard required for the calculation of snow loads (1mm = 1 kgf / m^2 = 10 Pa):

$$S = 0{,}01 \cdot h_B, \tag{9.4}$$

where S is the snow load in kPa; h_B is average on the site water supply in the snow cover, mm.

Meteorological data on the water supply in the snow have been collected and published since 1936. These data provide comprehensive and the most accurate initial information about the snow load on the earth's surface.

Earlier this date, snow surveys included measurements of snow cover height and density. In such cases, the snow load can be defined as the product of the height of the snow cover and its volumetric weight (density):

$$S = 0{,}1 \cdot \rho \cdot h, \tag{9.5}$$

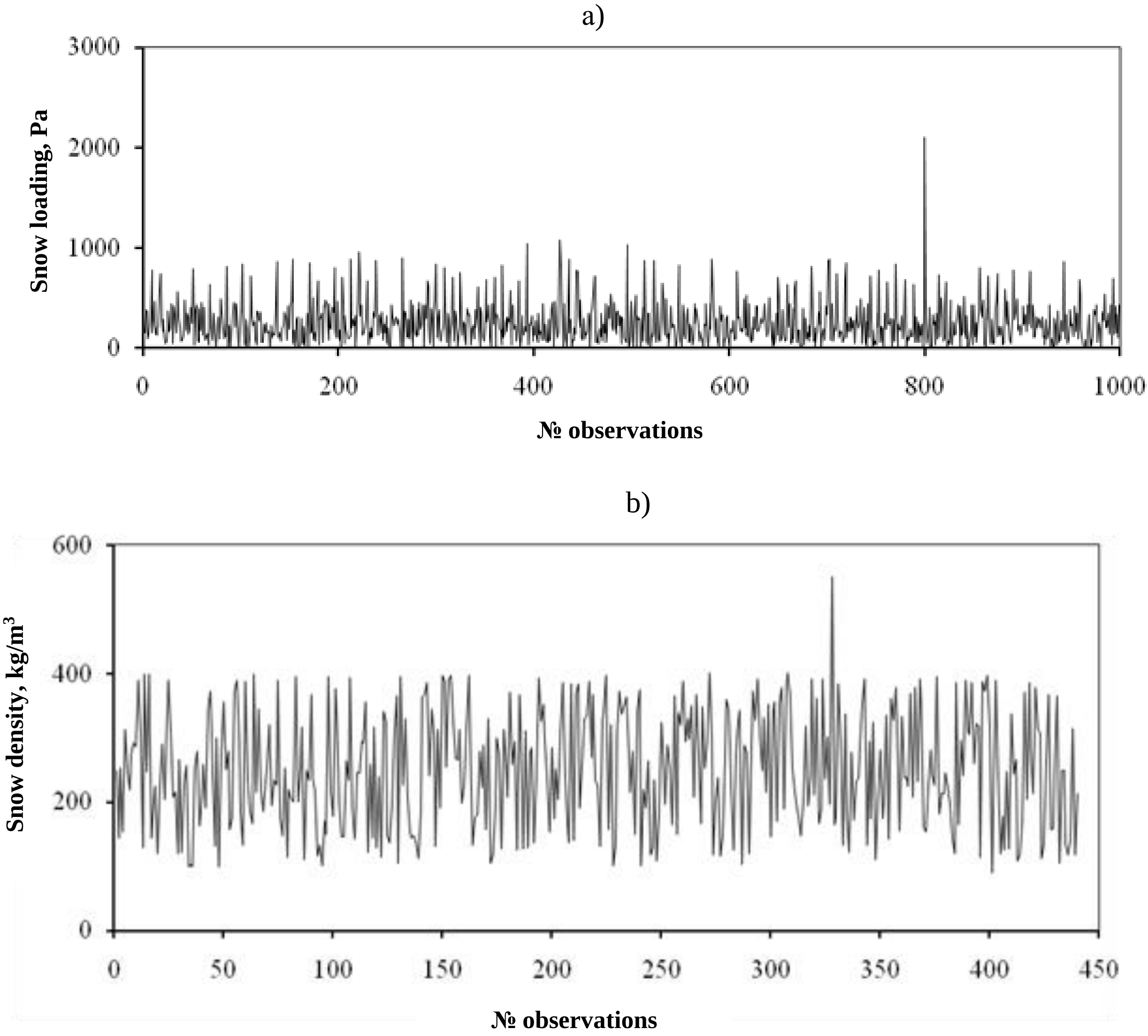

Fig. 9.9. Implementation of a random process of snow loading and snow density (for Semenivka station)

where S is the snow load in kPa; ρ is density of snow in g / cm^3; h is height of snow cover in cm

As mentioned above, depending on climatic conditions, the snow cover can have different heights and densities both in space and time. Therefore, the values of snow load obtained by formula (9.5) can have a significant variance. However, according to meteorologists, this approach gives very informative results and can significantly extend the climate series of observations of snow load.

It is interesting to note that there are recommendations [18] that directly link the height of the snow cover with the snow load:

$$S_{50} = 1{,}91(h_{50})^{1{,}33},$$

where S_{50} is snow load on the ground (kPa) with a recurrence period of 50 years; h_{50} is height of snow cover (m) also with a recurrence period of 50 years.

However, the main source of reliable experimental data on snow load on the earth's surface should be considered data from snow surveys, during which the water supply in the snow was established, or observations when both the height of snow cover and its average density were measured.

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Control questions

1. Why is snowfall a dangerous natural phenomenon?
2. Does the snow load pose a danger to buildings?
3. How is snowfall formed?
4. Describe the snow cover in Ukraine.
5. How is the initial data on snow cover collected?
6. Density of snow as the main climatic characteristic of snow cover.
7. How is the snow load on the earth's surface determined?

LECTURE 10. STANDARDIZATION OF SNOW LOAD

10.1. Design values of snow load

Snow load is a variable for which three design values are set:
- limit design value;
- operational design value;
- quasi-constant design value.

The limit design value of the snow load on the horizontal projection of the roof is calculated by the formula

$$S_m = \gamma_{fm} S_0 C, \qquad (10.1)$$

where γ_{fm} is the reliability coefficient of the limit value of snow load;
S_0 is characteristic value of snow load (in Pa);
C is the coefficient determined by formula (10.4).

The operational design value is calculated by the formula

$$S_e = \gamma_{fe} S_0 C, \qquad (10.2)$$

where γ_{fe} is the reliability coefficient of the operational value of snow load;
S_0, C are the same as in formula (10.1).

Quasi-constant design value is calculated by the formula

$$S_p = \left(0{,}4 S_0 - \overline{S}\right) C, \qquad (10.3)$$

where $\overline{S}$ = 160 Pa;
S_0, C are the same as in formula (10.1).

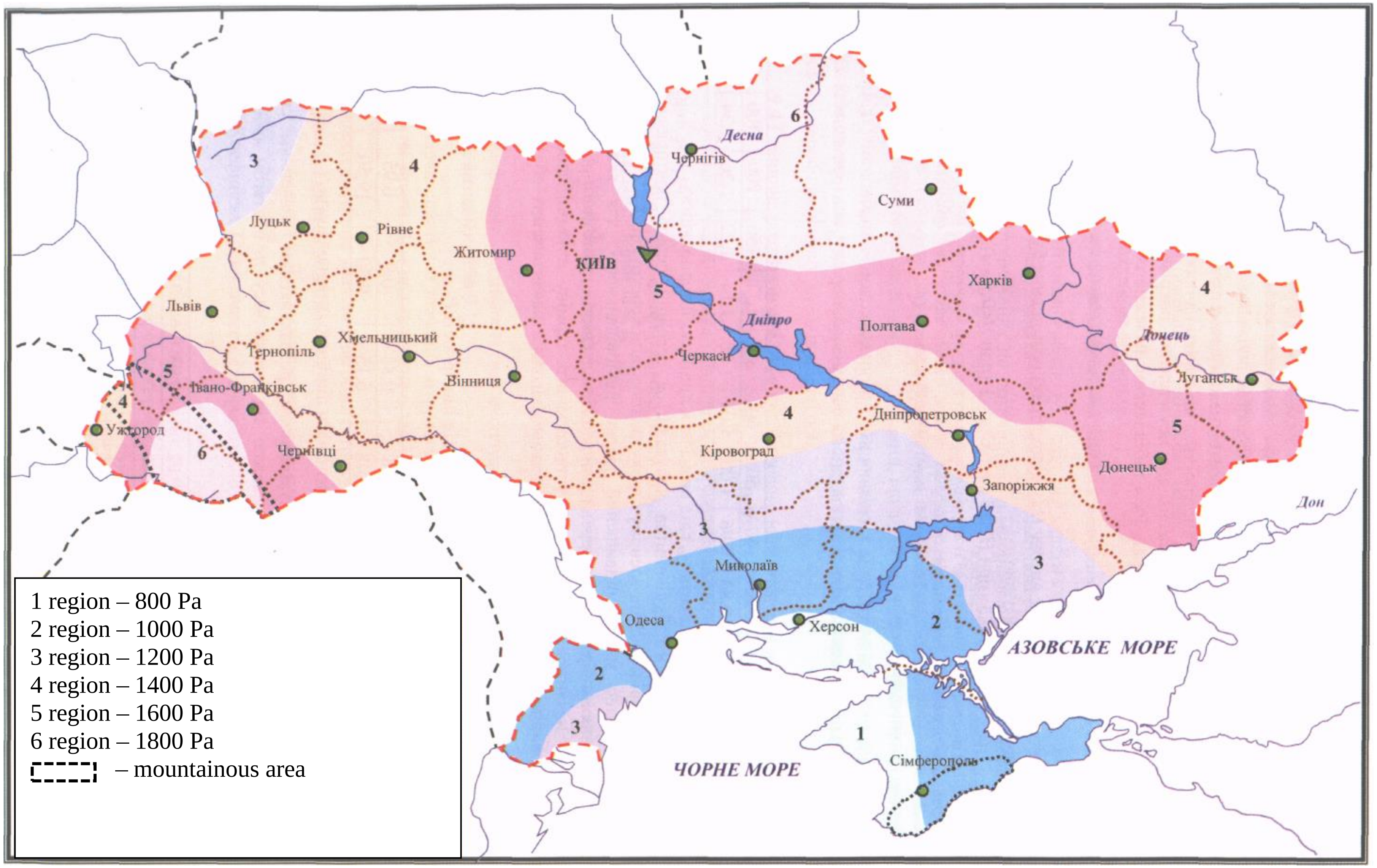


Figure 10.1. Map of zoning of the territory of Ukraine by characteristic values of snow cover weight

The characteristic value of snow load S_0 (in Pa) is equal to the weight of snow cover per one square meter of soil surface, which can be exceeded on average once in 50 years.

The characteristic value of snow load S_0 is determined depending on the snow districts on the map *(Fig. 10.1)* or in Annex E [1].

Note. If necessary, it is allowed to determine the value of snow load S_0 by statistical processing of the results of snow surveys.

10.2. Coefficients of the method of calculating the snow load

The coefficient *C* is determined by the formula

$$C = \mu C_e C_{alt}, \tag{10.4}$$

where μ is the coefficient of transition from the weight of snow cover on the soil surface to the snow load on the roof;

C_e is coefficient that takes into account the mode of the roof operation;

C_{alt} is the coefficient of altitude.

The coefficient μ is determined by Annex F [1] depending on the shape of the roof and the scheme of distribution of snow load, intermediate values of the coefficient should be determined by linear interpolation.

Note. *In cases where more unfavorable operating conditions of structural elements occur during partial loading, you should consider schemes with a snow load that acts on half or on quarter of the span (for roofs with lanterns - in areas of width b). If necessary, snow loads should be determined taking into account the expected further expansion of the building.*

Variants with increased local snow loads, given in Annex G, should be taken into account when calculating slabs, decks and spans, as well as when calculating those elements of load-bearing structures (trusses, beams, columns, etc.), for which these variants determine the size of sections.

Note. *Simplified snow load schemes equivalent to the load schemes given in Annex G may be used in the calculation of structures. When calculating the frames and columns of industrial buildings, it is allowed to take into account only the uniform distribution of snow load, except for places of difference in roof, where it is necessary to take into account the increased snow load.*

Detailed definition of the coefficient μ for different forms of roofs is set out below in paragraph 10.10.

The coefficient C_e takes into account the influence of the operating mode on the accumulation of snow on the roof (cleaning, melting, etc.), it is set by the design task.

When determining the snow loads for uninsulated roofs of shops with increased heat dissipation with roof slopes of more than 3% and ensuring proper drainage of melt water this coefficient should be taken C_e = 0.8.

Note. In the absence of data on the operation of the roof, the coefficient Ce may be taken as equal to one.

The coefficient C_{alt} takes into account the height H (in kilometers) of the building above sea level, it is determined by the formula

$$C_{alt} = 1{,}4H + 0{,}3 \text{ (at } H \geq 0{,}5 \text{ km)};\ C_{alt} = 1 \text{ (at } H < 0{,}5 \text{ km)}. \quad (10.5)$$

Note. Formula (10.5) is used for objects located in mountainous terrain, and gives an approximate value in the margin of safety. In the presence of the results of snow surveys conducted in the area of the construction site, the characteristic value of the snow load is determined by statistical processing of snow survey data and is taken $C_{alt} = 1$.

10.3. Reliability coefficients according to the values of snow load

The reliability coefficient γ_{fm} for the limit design value of the snow load is determined depending on the specified average recurrence period T according to *Table 10.1*.

Table 10.1

The values of the reliability coefficient γ_{fm}

T, years	***1***	***5***	***10***	***20***	***40***	***50***	***60***	***80***	***100***	***150***	***200***	***300***	***500***
γ_{fm}	*0,24*	*0,55*	*0,69*	*0.83*	*0,96*	*1,00*	*1,04*	*1,10*	*1,14*	*1,22*	*1,26*	*1,34*	*1,44*

Intermediate values of the coefficient γ_{fm} should be determined by linear interpolation.

For objects of mass construction, it is allowed to take the average recurrence period T as equal to the established service life of the structure T_{ef}.

For objects with a high level of responsibility, for which the technical task sets the probability P of not exceeding (providing) the limit value of the snow load during the specified service life, the average recurrence period of the limit value of the snow load is calculated by the formula

$$T = T_{ef} K_p, \quad (10.6)$$

where K_p is the coefficient determined according to *Table 10.2* depending on the probability of P.

Table 10.2

The values of the coefficient K_p

P	***0,37***	***0,5***	***0,6***	***0,8***	***0,85***	***0,9***	***0,95***	***0,99***
K_p	*1,00*	*1,44*	*1,95*	*4,48*	*6,15*	*9,50*	*19,50*	*99,50*

Intermediate values of the coefficient K_p should be determined by linear interpolation.

The reliability coefficient γ_{fe} for the operational design value of the snow load is determined according to *Table. 10.3* depending on the proportion of time η during which the conditions of the second limit state may be violated.

Table 10.3

The values of the coefficient γ_{fe}

η	*0,002*	*0,005*	*0,01*	*0,02*	*0,03*	*0,04*	*0,05*	*0,1*
γ_{fe}	*0,88*	*0,74*	*0,62*	*0,49*	*0,40*	*0,34*	*0,28*	*0,10*

Intermediate values of the coefficient γ_{fe} should be determined by linear interpolation.

The value η is accepted according to the design codes or is set by the design task depending on their purpose, responsibility and consequences of exceeding the limit state. For objects of mass construction, it is allowed to accept η = 0,02.

When designing tall structures, the relative dimensions of which satisfy the condition *h/d>7*, it is necessary to additionally perform a test calculation for vortex excitation (wind resonance); here *h* is the height of the structure, *d* is the minimum size of the cross section located at the level of *2/3h*.

10.4. Probabilistic substantiation of snow load codes

The development and publication of the State Codes of Ukraine DBN B.1.2-2006 "Loads and loadings" [1] in terms of snow loads was preceded by many years of work by Ukrainian researchers, including A.V. Perelmuter and M.A. Mikitarenko (VAT UkrNIIProektstalkostruktsiya named after V.M. Shimanovsky), V.A. Pashinsky, S.F. Pichugin, Yu.V. Dryzhyruk (Poltava National Technical University named after Yuri Kondratyuk), R.I. Kinash (Lviv Polytechnic State University) and others [3, 4, 5, 6, 10]. The results of snow surveys performed at 222 meteorological stations and posts of Ukraine during 1950-1990 with the duration of climate series from 21 to 35 years were used for statistical research and normalization of snow load. The location of observation points is quite uniform throughout the territory: the distance between the nearest of them is 30… 60 km. In general, a representative sample of more than 100,000 snow survey results was used to normalize Ukraine's snow load.

The obtained results indicate a significant territorial variability of snow load, which differs significantly from its rationing according to SNiP [2], according to which almost the entire territory of Ukraine belonged to the lowest snow I (S_0 = 0.5 kPa) and II (S_0 = 0.7 kPa) districts. Meanwhile, the experimentally substantiated design values of snow load, corresponding to the basic average recurrence period T = 50 years, vary from 0.76 kPa for Kherson region and Crimea to 1.79 kPa in the north-eastern regions of Ukraine. The rather high values (1.20… 1.80 kPa) of snow load were registered at some southern meteorological stations. The analysis of research data, in addition, confirmed that in Ukraine there are particularly snowy winters, for example, in 1963-64, 1966-67 and 1986-87. At some points, the maximum weight of snow cover exceeded 2.0 kPa, which did not fall out of the total set of annual maximums (see the analysis of interannual variability of snow load in paragraph 10.6). The presence of such values increases the coefficient of variation of the samples of annual maxima and the design values of snow load.

Territorial zoning of Ukraine according to the characteristic values of the snow cover weight includes six territorial districts with estimated characteristic values from 0.8 to 1.8 kPa (*Fig. 10.1*). The zoning reserves included in the developed map can be analyzed by comparing the actual values for each weather station with the corresponding district values. It turns out that the actual design loads exceed the district loads by no more than 12% for 21% of weather stations. At the same time, due to the necessary generalization, the design values were increased by an average of 16.4% and in some cases exceed the actual loads by 50%. Such deviations are a natural result of the smoothing procedure, which brings the data of individual meteorological stations closer to the general level due to the influence of macrometeorological factors, and compensates for random errors in calculating design values from individual meteorological stations.

For the transition from the base period of recurrence T = 50 years to other values of T (in years) the dependence for the coefficient of reliability of the limit design value of snow load was justified for the territory of Ukraine:

$$\gamma_{fm} = 0{,}24 + 0{,}45 \lg T . \qquad (10.7)$$

In the text of the DBN, this relationship is given in tabular form (*Table 10.1*).

In the codes of DBN [1] it is accepted that the operational design value of snow load S_e depends on the proportion of time η during which it may be exceeded. According to 64 meteorological stations of Ukraine, the operational design values of snow load S_e were calculated, depending on the geographical area and the share of the service life of the structure η [4]. This coefficient can be determined by one of the numerically similar formulas:

$$\gamma_{fe} = 2 \cdot \sqrt[3]{-\lg \eta} - 1{,}9, \qquad \gamma_{fe} = -0{,}48 \lg \eta - 0{,}36. \qquad (10.8)$$

You can also use the corresponding DBN table (see *Table 10.3* above), built on formulas (10.8).

Quasi-constant design value S_p of variable snow load should be used for structures that are in conditions of long-term rheological processes. In this case, according to the general provisions of load rationing [1], this value must be equivalent in terms of impact on the design of the actual load process. Analysis of data from 64 Ukrainian meteorological stations allowed to do without a separate map for the values of S_p. A fairly simple dependence of S_p (10.3) on the characteristic value of S_0 [4] was introduced in DBN. It should be noted here that the SNiP codes established a long-term component of the snow load only for the third snow region, which occupies a small area in the northeast of Ukraine. Here the long-term snow component was equal to 0.42… 0.46 kPa. According to the DBN codes, the quasi-constant value of snow load for the same area, referred to the 6th snow region, is equal to 0.56 kPa. Thus, the regulated DBN zoning of the design values of S_p , as well as the above-described zoning of the design values of the snow load, provides a margin of safety of structures in comparison with the codes of SNiP.

Already after the release of DBN there were reasonable proposals (R.I. Kinash, J.S. Hook [12]) for more detailed snow zoning of the mountainous Carpathian region (within the Transcarpathian region) with the introduction of additional 5 districts (from 7 to 11-th) with characteristic snow loads in the range of 2.2… 4.0 kPa.

10.5. The effect of applying the codes of DBN

It should be emphasized that the limit design values of snow load included in DBN [1] in most cases exceed the corresponding values set by SNiP. On the one hand, this leads to an increase in cross-sections and material consumption of load-bearing structures of roofs, but on the other – to increase the level of their reliability. As an example, the monograph [4] presents a comparative analysis of the level of reliability, design and economic performance of load-bearing structures of light roofs, designed for the action of snow load with SNiP 2.01.07-85 [2] and DBN [1].

Necessary indicators are established by experimental design of triangular steel trusses under a roof from corrugated asbestos-cement sheets on steel beams. The trusses are designed for the territory of Ukraine for a constant design load of 0.4 kPa and snow loads corresponding to the geographical regions. When designing according to DBN, the limit design values of snow load are determined by the formula (10.1) at C = 1 for four service life: 10, 25, 50 and 100 years. Taking into account these five options and 25 regions of Ukraine, a total of 125 trusses were considered. The design results for DBN for a service life of T = 10 years are close to SNiP, with increasing service life, the metal content of trusses increases. At the same time, a significant increase in the design values of the snow load leads to a slightly smaller increase in material consumption. For example, at T = 50 years, the design values of snow load increase by an average of 58%, and the weight of trusses – only by 22%. This is due to the constant load, the size of most knots and a number of rods selected for maximum flexibility.

The failure probabilities $Q(T)$ of all designed trusses were also determined for the service life T = 10, 25, 50 and 100 years. As expected, the probability of failure of trusses designed for snow load according to SNiP [2], have a significant spread across the regions of Ukraine and increase significantly with increasing service life ($Q(T)$ = 0.06… 0.61 at T = 50 years). This is quite logical, because these trusses were designed taking into account snow loads, independent of service life. The received estimations testify to insufficient reliability of roof trusses which calculations took into account snow codes of SNiP. Trusses designed according to DBN [1] have more stable reliability indicators, which indicates a more detailed and accurate territorial zoning of the characteristic values of snow load in DBN. Thus, when increasing the service life of trusses to T = 50 years, the probability of their failure is in the range from 0.01 to 0.06, which is fully consistent with the required level of reliability of mass structures.

It is obvious that the increase in material consumption and cost of load-bearing structures due to the introduction of new codes of snow load should be offset primarily by the social effect of trouble-free operation of load-bearing structures. In addition, the feasibility of introducing new codes of snow load is evidenced by a certain economic effect. Its calculation was based on a comparison of the wholesale prices of trusses designed according to SNiP and DBN, increased in proportion to the probability of failure of these trusses. Despite the increase in the weight of trusses, the positive economic effect of increasing their reliability is observed for almost the entire territory of Ukraine, it increases with increasing service life of trusses and at T = 50 years on average in Ukraine reaches 16%.

It should be noted that this approach taken took into account only the cost of trusses, taking into account the necessary replacements due to possible failures. This approach did not take into account the cost of dismantling emergency structures, work on the installation of new trusses, losses from downtime and loss of destroyed equipment in case of actual failure. Therefore, the obtained values of the economic effect should be considered as an approximate estimate from below. The real economic effect of increasing the reliability of trusses designed to increase the value of snow loads should be much greater.

10.6. Analysis of interannual variability of snow load

An objective assessment and a reasonable long-term forecast of snow load should take into account the presence (or absence) of a long-term trend in the weight of snow cover. In particular, especially snowy winters are sometimes observed on the territory of Ukraine, which required analyzing the interannual variability of snow load taking into account the frequency and dates of such winters.

RI Kinash published the useful generalized information on this issue for the territory of Ukraine in the monograph [5], which provides data on the change in the time of the average snow load in Ukraine from 1933 to 1996 (*Fig. 10.2*). As can be seen in the graph, during the whole rather long period of time the values of the average snow load in the region range from 200 to 400 Pa, without showing a clear tendency to overall increase or decrease. Significant deviations beyond these limits are explained by the small number of observations in some years, ie insufficient representativeness of the data. Therefore, the conclusion is substantiated that the long-term process of snow loading in Ukraine is stationary. Moreover, the example of the constant snow load over the last 60 years suggests that the fears of global climate change related to warming are too exaggerated.

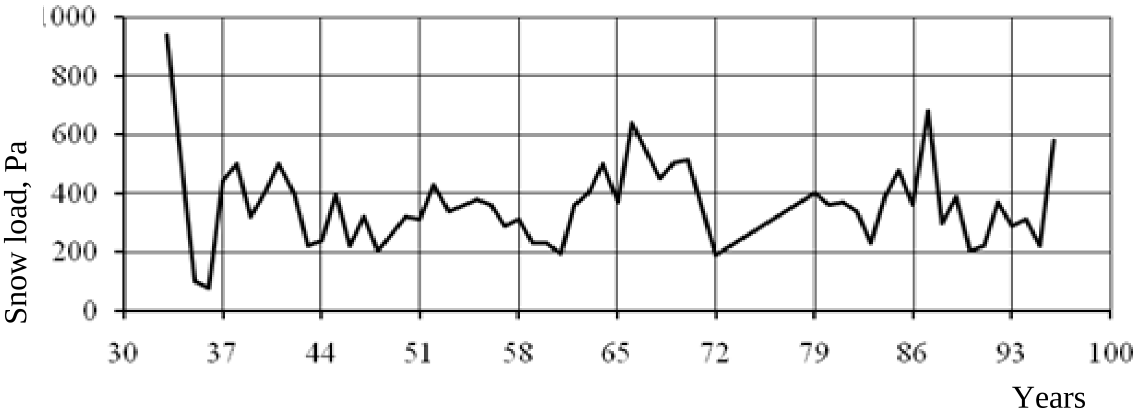


Fig.10.2. The average snow load in Ukraine by years of observations

Meanwhile, meteorologists note that over the past century, the average annual temperature in Ukraine has risen by 0.7 degrees, and the average temperature of the cold period has risen by two degrees. The trend towards warming, which intensified in the early 1980s, continues. According to the accepted scenarios in the world (usually the British and American scenarios of climate change on Earth), in the next 20 years, due to the growing amount of carbon dioxide in the atmosphere, the average annual temperature in Ukraine will rise by 1… 1.5 degrees. Due to this, winters in this region will remain unstable, with little snow, with sharp temperature fluctuations throughout the year. It is possible that in some areas of Ukraine snow cover may become a rarity, which does not preclude the occurrence of excessive snowfall in some years.

This scenario is confirmed by the analysis of long-term data for Poland, which revealed some negative trend (decrease) of average values of annual snow load maximums according to 38 weather stations out of 47 studied (*Fig. 10.3*), although at 9 weather stations the trend was opposite. At meteorological stations in Japan, where regular observations are made for 110… 120 years, long-term series of observations show a clear trend of increasing average winter temperatures, which confirms the thesis of global warming. At the same time, 11-year cycles of changes in the height of snow cover in the islands of Japan are clearly traced.

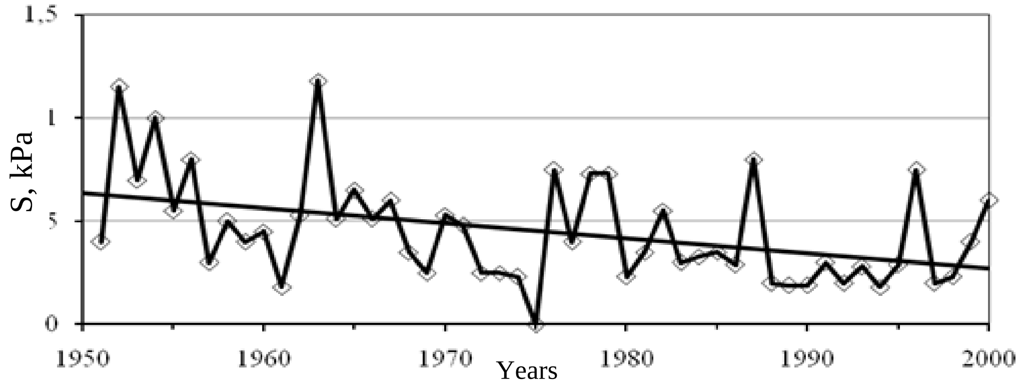


Fig. 10.3. Trend of annual maximums of snow load on the ground (meteorological station Rzeszow, Poland)

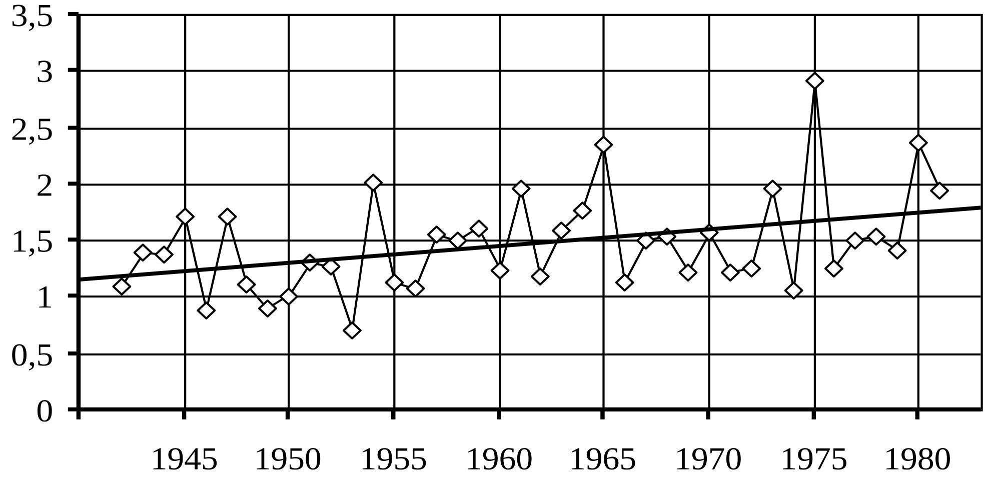


Fig. 10.4. Trend of annual snow load peaks (Leningrad region, Russia)

Meanwhile, in some areas of Russia there was a noticeable positive trend of snow loads in the second half of the twentieth century (*Fig. 10.4*). As climatologists predict global warming by 2050 and beyond, it is assumed that the projected maximum snow loads will increase by 25… 35%, particularly in the north-western region of the European part of Russia.

10.7. European standards Eurocode

In the codes Eurocode [9], the characteristic value of snow load is basic. This value, denoted as S_k , is equal to the weight of snow cover per 1 m^2 of land surface, which may be exceeded annually with a probability of 0.02 (or on average once every 50 years). These values are determined from the maps of territorial zoning of the member states of the European Community, an example of which is shown in *Fig. 10.6*. The European Ground Snow Load Map is divided into 9 homogeneous climatic regions (*Fig. 10.5*), which also includes maps of the Czech Republic, Iceland and Poland. Each of the regions is divided into zones, for which the maps show the characteristic values at sea level (altitude A = 0).

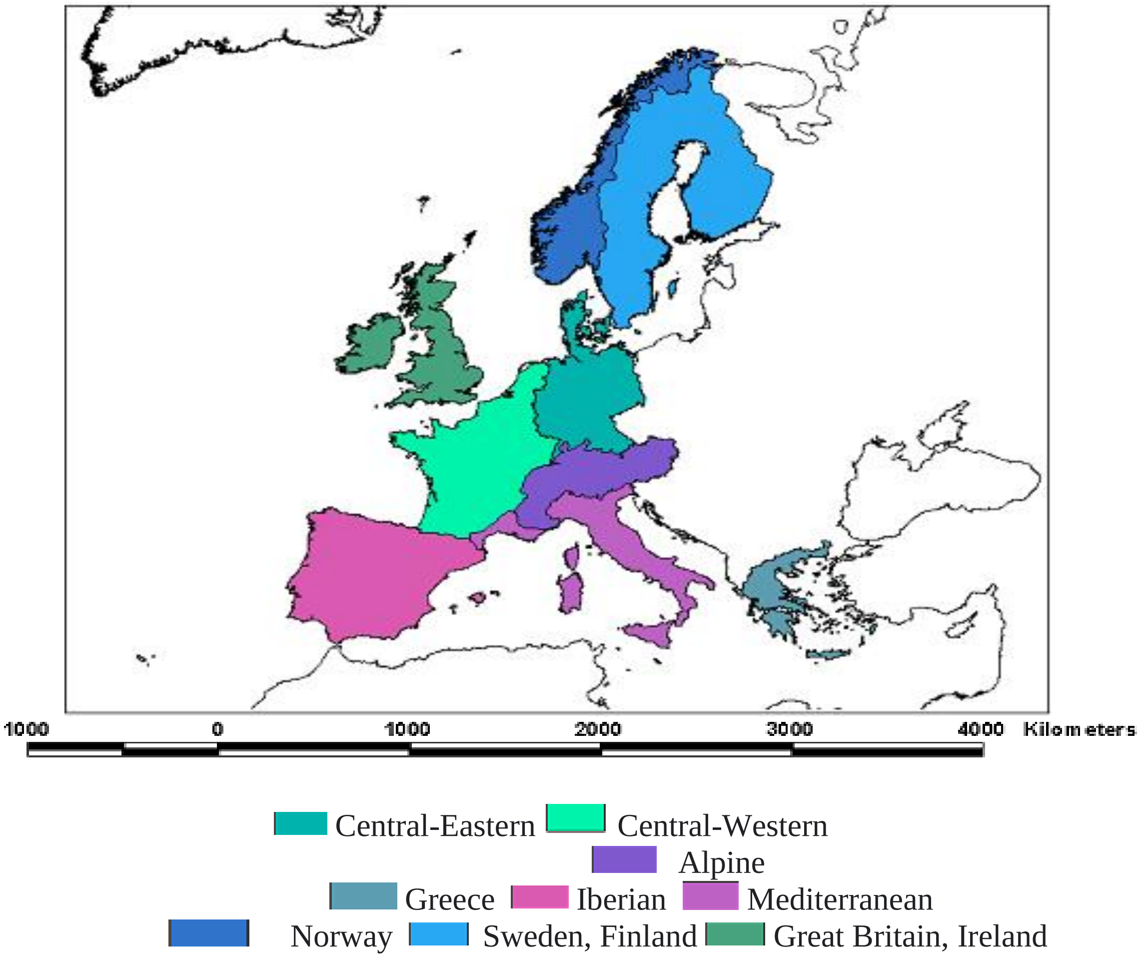


Fig. 10.5. Map of snow areas according to Eurocode

For the transition to a specific construction site, the corrective formula of the following general form is recommended:

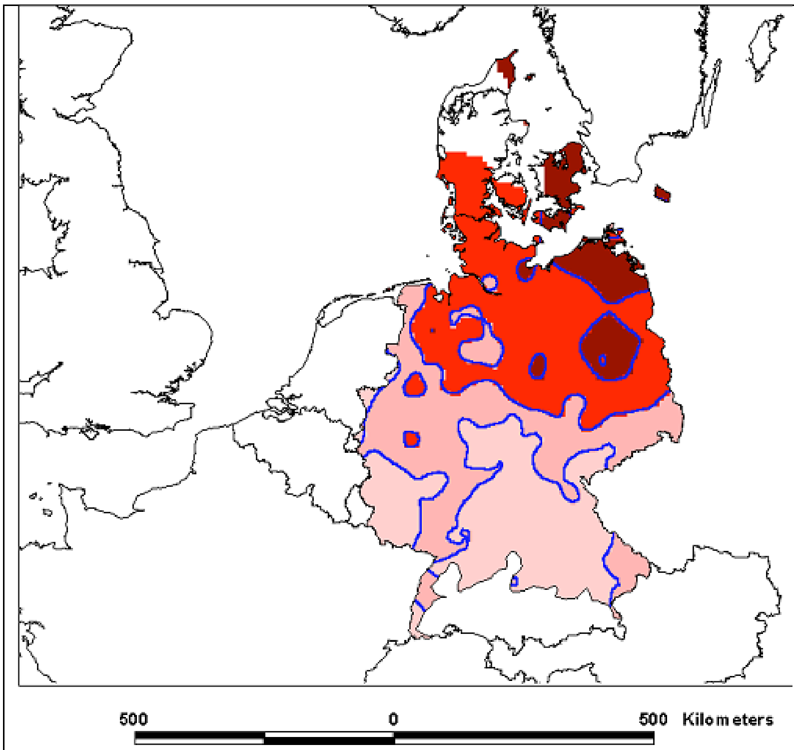
$$S_k = (aZ + b)\left[1 + \left(\frac{A}{c}\right)^2\right], \tag{10.9}$$

where Z is the zone number on the map of the region; a, b, c – numerical factors, individual for each region.

In particular, for the Central-Eastern region, formula (10.9) takes the form

$$S_k = (0{,}264Z - 0{,}002)\left[1 + \left(\frac{A}{256}\right)^2\right].$$

A detailed review of snow load rationing according to Eurocode is given in the monograph [3].



1 zone ($S_k = 0{,}3$)
2 zone ($S_k = 0{,}5$)
3 zone ($S_k = 0{,}8$)
4,5 zones ($S_k = 1{,}2$)

Fig. 10.6. Snow zones of the Central-Eastern region (S_k в кН/м^2)

Fig. 10.6. Snow zones of the Central-Eastern region

10.8. Influence of the absolute height of the terrain on the snow cover

In mountainous areas, absolute (geographical) altitude is usually considered a determining factor in the distribution of snow cover. This is due to the fact that there is a relationship between lowering the temperature with increasing altitude and the corresponding decrease in melting losses. In addition, the absolute humidity required for precipitation decreases with increasing altitude.

According to M.V. Zavaryna [7], the increase in the average density of snow (and, accordingly, the snow load on the ground) per 100 m of height reaches 0.02 g / cm^3 due to the fact that with increasing absolute height increases the height of snow cover.

According to some researchers, in some areas in a certain altitude interval a linear relationship can be found between snow accumulation, which determines the snow load, and absolute altitudes. In particular, R.I. Kinash, based on meteorological data for the Ukrainian Carpathians, concluded that up to 700 m the snow load does not depend on the height of the terrain, and for altitudes 700…1600 m it grows linearly with a proportionality factor of 0.292, which connects the height (in meters) with load (in kilograms per square meter) [44].

The recommendation for heights exceeding 400 m was given by V.A. Pashinsky [4] in the following form:

$$S_H = S_0 + 0{,}0029(H - 400), \qquad (10.10)$$

where H is the height above sea level (in m); S_0 is design snow load for the plains (in Pa); S_H is design snow load for areas with a height of H above sea level.

In a slightly different form (10.5), this recommendation was included in the DBN [1].

These dependencies differ slightly from the recommendations of Polish codes [5], which for the Carpathian Mountain zone of Poland regulate the relationship between altitude and snow load as follows:

$$S_H\left(\frac{kH}{m^2}\right) = 0{,}003H(m). \qquad (10.11)$$

Dependence (10.11) extends to the height range of 300… 1000 m, for high altitudes the snow load is set individually.

G. Shpete [8] gives for Central Europe an empirical dependence on geographical altitude for the average value of annual maximums of snow cover height:

$$m_H = AB^H, \qquad (10.12)$$

where H is the altitude above sea level (m); A and B are coefficients depending on climatic conditions: for coastal areas A = 0.25; B = 1,000875; for inland areas A = 0.25; B = 1,000202; for mountainous areas A = 0.25; B = 1.001919.

10.9. Influence of melting snow on roofs

Snow cover on the roof surface is in different temperature conditions in height. The upper layer is under the influence of outside air, which has a temperature that is constantly changing. The bottom layer is in contact with the roof, which has a higher and more stable temperature in winter. Under the influence of the temperature gradient due to the difference in temperature of the snow layer and the outside air, the snow melts.

Quite often the insulated roof over the heated building has insufficient heat transfer resistance. Thus, melting of snow on roofs is caused by existence of heat losses through a roof, heat from heating cables, a network of steam heating and openings of ventilating wells. Since the thermal conductivity of snow is usually low, the snow on the roofs of heated buildings helps to reduce their heat loss. As the thickness of the snow cover increases, the zero isotherm will shift to the thickness of the snow, which will lead to melting at the boundary between the upper surface of the roof and the snow. The thickness of the snow layer decreases until the zero isotherm returns to the roof. The new snowfall improves the thermal insulation capacity of the snow layer, the zero isotherm moves into the snow and melting begins again, until the thickness of the snow stabilizes at a value that cannot insulate the surface from subzero temperatures.

Fig. 10.7. Snow cover on the fuel roof

Consider the case when the melting of snow occurs due to the fact that the temperature at the boundary of the upper surface of the roof is equal to or above 0˚C. The minimum (critical) height of snow cover that meets this condition is defined as [11]

$$h_{CH} \geq \lambda_{CH}\left[-\frac{t_3}{t_B}(R_T + R_B) - R_3\right], \tag{10.13}$$

where t_B is the temperature inside the room; t_3 is outside air temperature; R_B is resistance to heat absorption from indoor air; R_T is thermal resistance of the roof; R_3 is resistance to heat transfer to the outside air; λ_{CH} is coefficient of thermal conductivity of snow cover; for fresh dry snow λ_{CH}= 0.07 kcal / m-g-deg.

From this formula it is seen that as the temperature of the outside air decreases, the critical height of the snow cover increases. At the same time, increasing the indoor air temperature reduces the critical height of the snow cover.

The critical height of snow also depends on the thermal resistance of the roof: with the improvement of thermal properties of roof materials increases the likelihood of snow accumulation of greater height.

The above considerations were confirmed by field observations of snow cover on the roofs of PoltNTU. On the roof of one of the educational buildings, a low level of snow, a layer of ice and a large number of icicles in the area of the roof groove were systematically observed (*Fig. 10.7*). The roof of the building included a ribbed reinforced concrete slab of 30 mm thick, a layer of expanded clay insulation 220 mm thick and a rolled roof (50 mm). Direct measurements showed that the air temperature inside the room was + 15ºC, outside - 8 ºC (in February). The corresponding critical height of the snow cover was 15 cm, according to formula (10.13).

The obtained height corresponds to the results of snow gauge observations, according to which in February 2010 the maximum height of snow cover on the surface of this building was 11… 15 cm, while on more insulated surfaces of other buildings height values varied from 18 to 25 cm. The following example shows that the formula (10.13) quite fully takes into account the process of melting snow on the floor due to heat loss and the nature of snow deposition on the roof (*Fig. 10.7*).

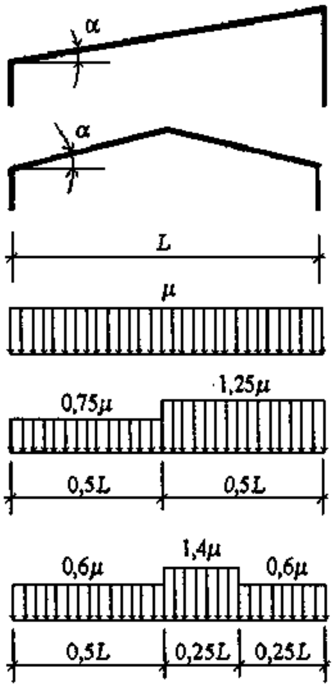
Based on research conducted at PoltNTU [10, 11], the dependence of the roof operation coefficient C_e has been installed on its thermal resistance R and the characteristic value of snow load S_0, which is represented by a simple analytical formula and allows to differentiate the value of C_e depending on these parameters:

$$C_e = 1 - 0{,}00022 S_0 \times exp(-0{,}6R). \tag{10.14}$$

The calculations performed according to the developed method of normalization of the roof operation coefficient for different variants of coverage in different regions of Ukraine showed that the value of C_e varies within 0.6… 1.0. In most cases, for uninsulated roofs with thermal resistance R_n <0.5 (m^2·°C) / W, the coefficient C_e is less than the recommended value of 0.8. The coefficient of the operating mode has the greatest influence when calculating light uninsulated roofs, the own weight of which is much less than the amount of snow load. The application of the adjusted value of C_e saves steel costs from 5 to 30%.

10.10. Schemes of snow loads and coefficients μ (Appendix Ж DBN [1])

Scheme 1. Houses with one-slope and two-slope roofs



Variant 1

Variant 2

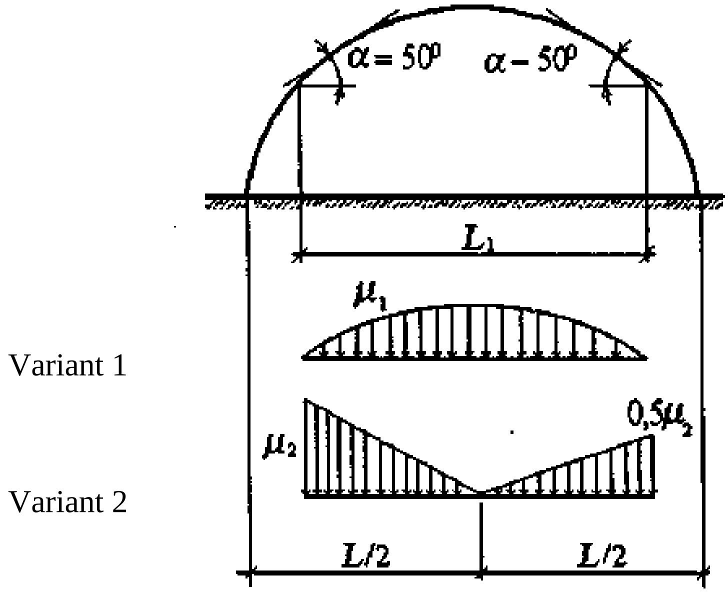
Variant 3

$\mu = 1$ при $\alpha \le 25^0$; $\mu = 0$ при $\alpha > 60^0$.

Options 2 and 3 should be considered for houses with gabled roofs (profile b), with option 2 – when $20^0 \le \alpha \le 30^0$, and option 3 – when $10^0 \le \alpha \le 30^0$ only in the presence of bridges or aeration devices on the ridge of the roof.

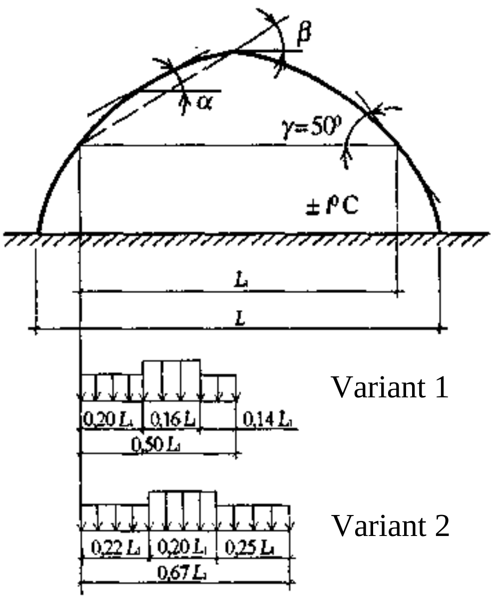
Fig. 10.8. Snow load on sloping roofs

Scheme 2. Houses with vaulted and close to them coatings



$\mu_1 = \cos 1{,}8\alpha$; $\mu_2 = 2{,}4 \sin 1{,}4\alpha$,where α – the angle of inclination of the coating, deg

Scheme 2 '. Roof in the form of arrow arches



Option 1 applies to buildings without a drawbridge.

Option 2 applies to buildings with a drawbridge.

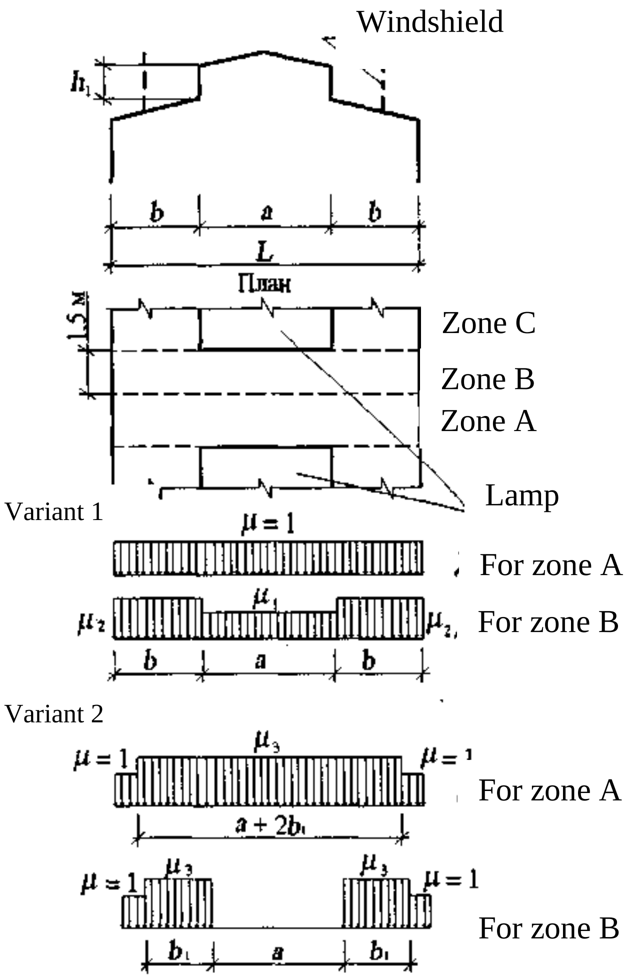
With a cold roof and cold mode inside the building (-*t* ° *C*): μ_1= 1.35; μ_2 = 1.75.

With a cold roof and warm mode in the building $(+\ t\ °\ C)$: $\mu_1 = 2.1$; $\mu_2 = 2.2$.

At $\beta \geq 20°$ it is necessary to use scheme 1*b*, taking $L = L_1$.

Fig. 10.9. Unbalanced snow load on the vaulted roof
(underground passage "Golden City" in Poltava)

Scheme 3. Buildings with longitudinal lanterns closed from above



$$\mu_1 = 0{,}8;\ \mu_2 = 1 + 0{,}1a/b;\ \mu_3 = 1 + 0{,}5a/b_1,$$

but no more than:

4.0 - for trusses and beams at the specified value of the coating weight of 1.5 kPa and less;

2.5 - for trusses and beams with a specified value of the weight of the coating over 1.5 kPa;

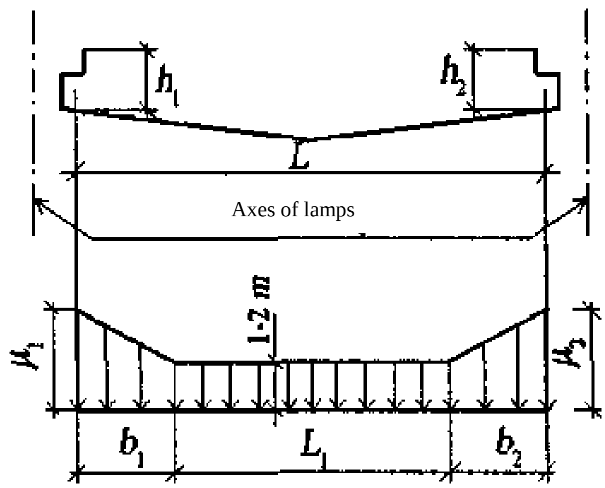
2.0 - for reinforced concrete slabs with a span of 6 m and less and for profiled steel flooring;

2.5 - for reinforced concrete slabs with a span of more than 6 m, as well as for girders regardless of the span;

$b_1 = h_1$ but not more than b.

When determining the load at the end of the lamp for zone *B*, the value of the coefficient μ in both options should be taken as equal to 1.0

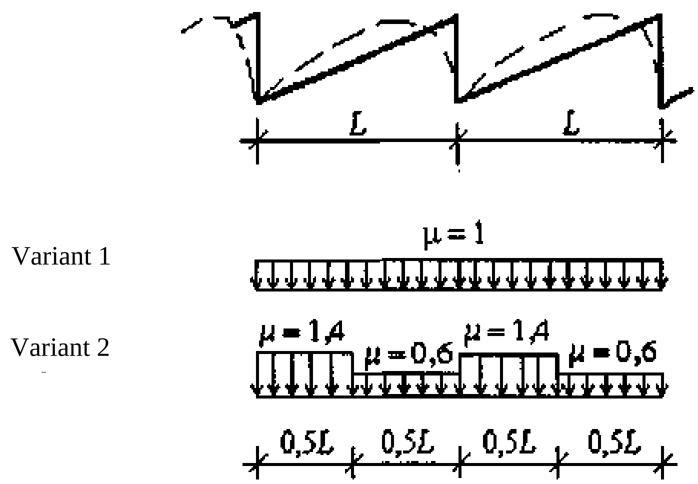
Scheme 3 '. Buildings with longitudinal lanterns open from above



$$\mu_1 = 1 + m\left(2 + \frac{L_1}{h_1}\right); \ \mu_2 = 1 + m\left(2 + \frac{L_1}{h_2}\right).$$

The values of *b* (*b*$_1$, *b*$_2$) and *m* should be determined according to the instructions in scheme 8; the span *L* is taken to be equal to the distance between the upper edges of the lanterns.

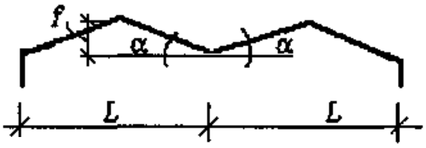
Scheme 4. Shed roofs



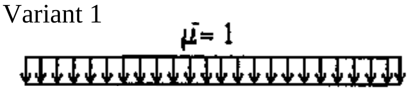
Schemes should be applied to shed coverings, including roofs with inclined glazing and vaulted outline of a roof.

Fig. 10.10. Deposition of snow in the ends of the roofs

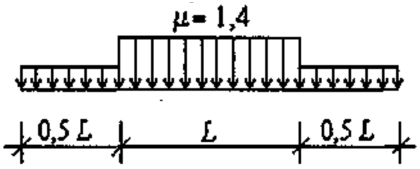
Scheme 5. Two-span and multi-span buildings with gabled roofs



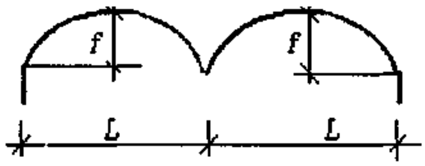
Variant 1



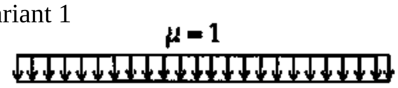
Variant 2



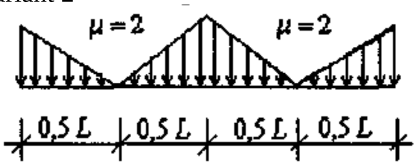
Scheme 6. Two-span and multi-span buildings with vaulted and close to them coatings



Variant 1

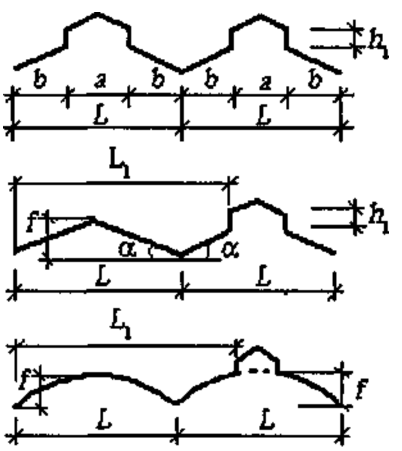


Variant 2



Option 2 should be considered at $f/L > 0.1$.

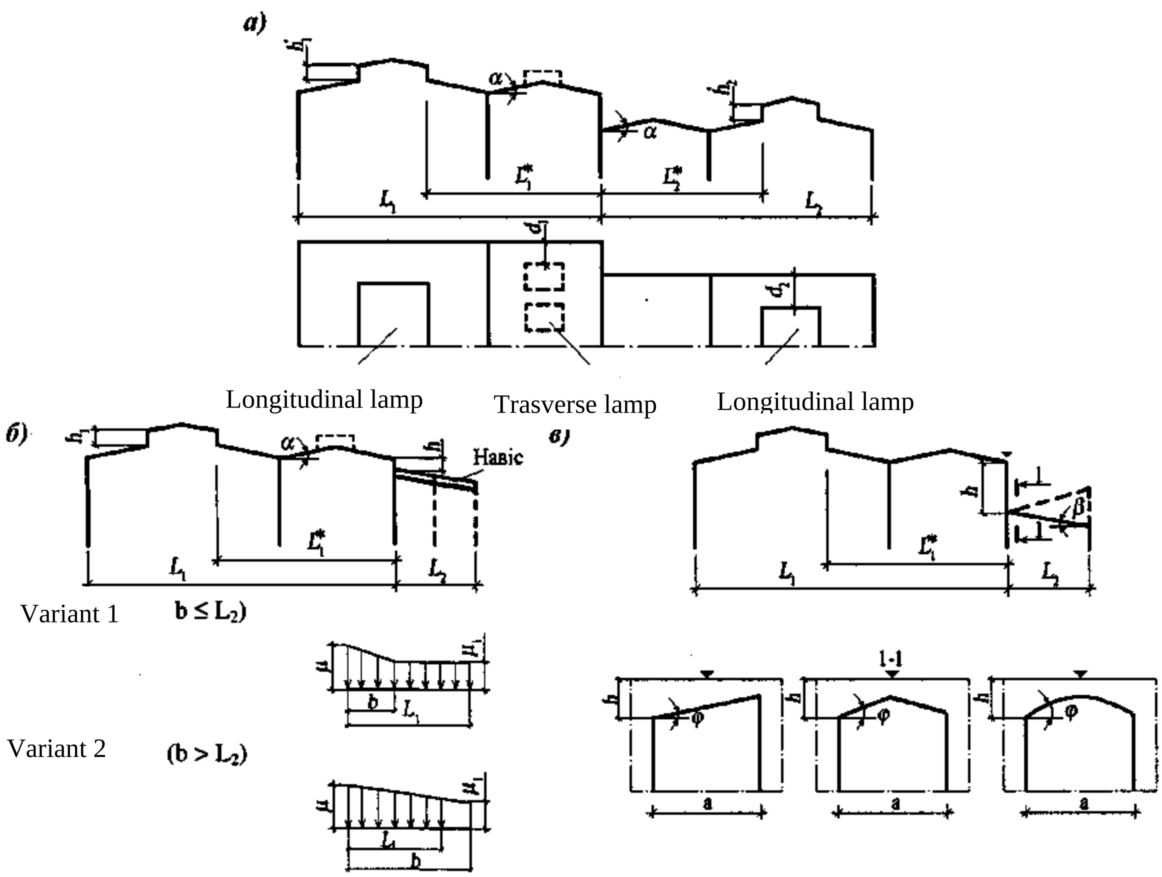
Scheme 7. Two-span and multi-span buildings with gabled and vaulted roofs with a longitudinal lantern



The coefficient μ should be taken for spans with a lantern in accordance with options 1 and 2 of scheme 3, for spans without a lantern - with options 1 and 2 of schemes 5 and 6.

For flat pitched ($\alpha \leq 15^0$) and vaulted (f/L <0,1) coatings at L_1 > 48 m, the local elevated load should be taken into account, as near differences in height (see Scheme 8).

Scheme 8. Buildings with a difference in height



The coefficient μ should be taken as equal to

$$\mu = 1 + \frac{1}{h}\left(m_1 L' + m_2 L_2'\right),$$

where h is the height of the difference, m, which is counted from the eaves of the upper floor to the lower roof and at a value of more than 8 m h is determined to be equal to 8 m;

L_1', L_2' – lengths of the upper (L_1') and lower (L_2') areas of the cover from which the snow is transferred to the zone of difference in height, m;

m_1 , m_2 – particles of snow carried by the wind to the height difference.

Scheme 9. Buildings with two height differences

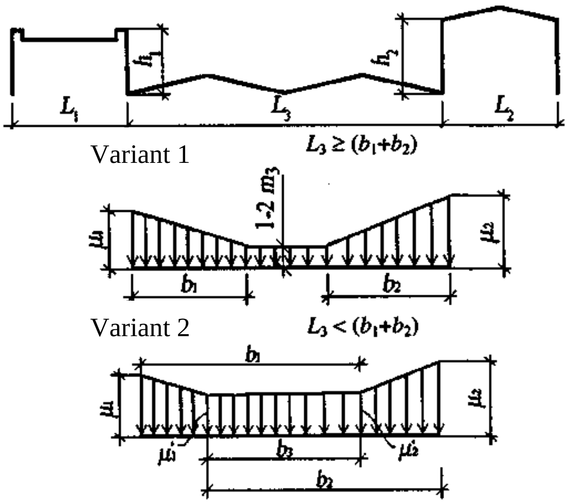
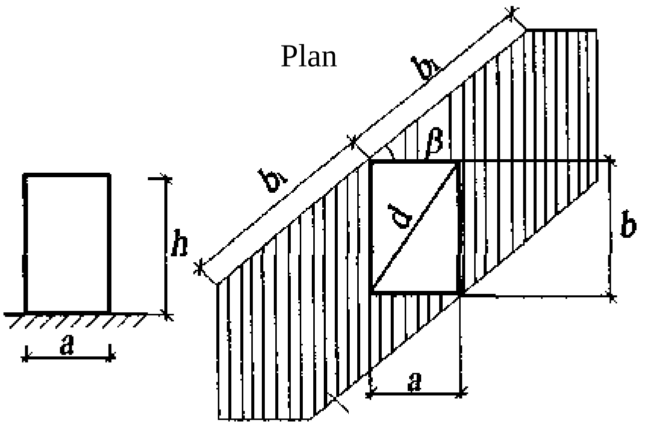


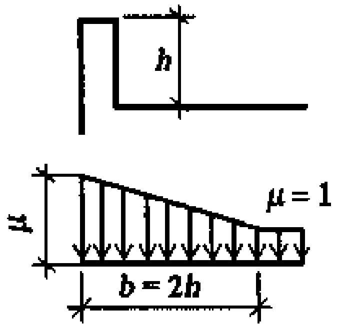
Fig.10.11. Snow deposition at roof height differences (snow bags)

Scheme 11. Areas of coverings adjacent to ventilation shafts which rise above a roof, and other superstructures



The scheme applies to areas with superstructures with a diagonal of the base of not more than 15 m. Depending on the design structure (calculation of slabs and rafters), it is necessary to take into account the most unfavorable location of the zone of increased load (at any angle β).

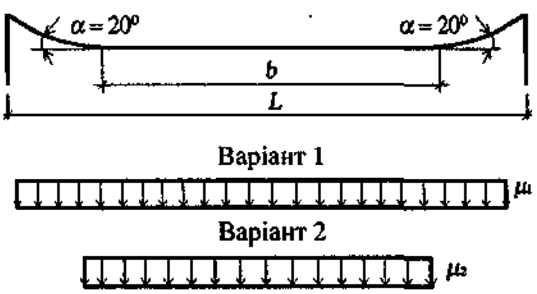
Scheme 10. Covering with parapets



The scheme should be used at $h > \frac{S_0}{2}$ (h – in m; S_0 – in kPa);

$\mu = \frac{2h}{S_0}$, but not more than 3.

Scheme 12. Hanging coverings of a cylindrical form



$$\mu_1 = 1; \qquad \mu_1 = \frac{l}{b}.$$

Fig. 10.12. Typical examples of snow hanging on roof eaves

References

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Control questions

1. What are the design values of snow load?
2. What coefficients are introduced when calculating the snow load?
3. What do the reliability coefficients of the values of snow load depend on?
4. What is the effect of applying the rules of DBN on snow load?
5. Perform an analysis of interannual variability of snow load.
6. What are the features of determining the snow load according to European standards Eurocode?
7. Does the absolute height of the terrain affect the snow cover?
8. How does the melting of snow on roofs affect the amount of snow load?
9. Give examples of schemes of snow loads on the roof of different configurations.

LECTURE 11. NATURE AND DESCRIPTION OF WIND LOAD

11.1. Atmospheric air movement

The Earth's atmospheric air is in constant motion. The only source of energy that causes the movement of the atmosphere is the Sun. Uneven heating of the Earth's surface, which in turn heats the air, creates a difference in atmospheric pressure. Cold air is denser, so it goes down and creates a zone of high pressure, warm air, on the contrary is less dense and, rising up, creates an area of low pressure. Wind is the movement of air relative to the earth's surface from areas of high pressure to areas of low pressure.

The energy emitted by the Sun is almost completely absorbed by the earth, which emits energy in the form of efficient radiation of the earth's surface. The atmosphere, mostly transparent to solar radiation but closed to the radiation of earth's surface, absorbs the earth's heat and partially radiates it back to the earth's surface. The atmosphere consists of four main layers: the troposphere, stratosphere, mesosphere and thermosphere. The first layer – the troposphere – is the thinnest, ending at an altitude of about 12 km above the Earth. This is the warmest layer, because the sun's rays are reflected from the earth's surface and heat the air. As it moves away from the Earth, the air temperature drops to about 55 ° C in the upper troposphere. The systems of weather and wind climate around the world are in the troposphere. They arise as a result of joint action on the atmosphere of solar radiation and the rotation of the Earth around the Sun.

The general circulation of the atmosphere is called a system of large-scale air currents over the globe, ie, such currents that are commensurate in size with most parts of the continents and oceans. In the general circulation of the atmosphere, with all the variety of its continuous changes, there are some persistent features that are repeated from year to year. To describe these features the so-called simplified model of atmospheric circulation is used, which assumes that the wind transfers heat from the equator to the poles and moisture from sea to land. This model qualitatively illustrates the influence of the distribution of air temperature in the atmosphere on the occurrence of winds, without taking into account a number of factors that actually determine the circulation of the atmosphere and its temperature.

The Earth's rotation and friction transforms the thermal circulation system into a three-cell meridional model (*Fig. 11.1*). In this case, each part of the planet has certain predominant (dominant) winds. Dominant winds are associated with the main processes of atmospheric circulation. In the area of the equator, where solar heat is most intense, hot air rises, creating an area of low pressure – the equatorial zone of calm. Rising air cools and is carried north and south before descending back to the surface in the subtropics (30º north and south latitude). In these places, high-pressure zones are formed «horse latitudes» – an area between 20º and 30º north and south latitudes (in the old days, dead horses were thrown overboard in this strip, so the latitudes got their current name).

From here the air is directed back in the direction of the comfort zone, forming two trade wind zones. Outside the tropics, the main prevailing winds are

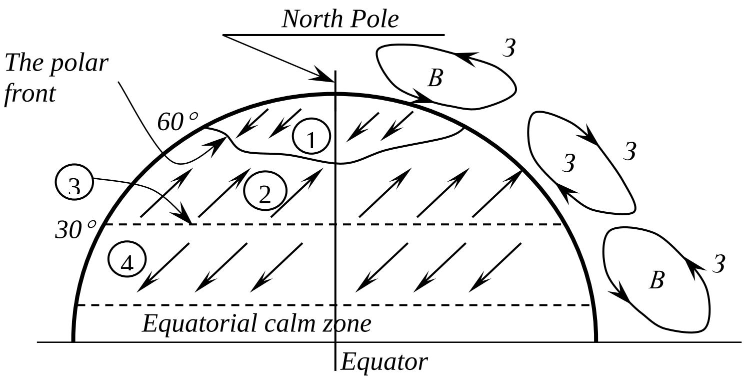

Fig. 11.1. Three-cell meridional circulation model
1 – Polar easterlies;
2 − westerlies;
3 – horse latitudes;
4 – trade winds.

westerly winds of temperate latitudes. They are formed by air moving from the horse's latitudes in the direction of the poles. These warm winds eventually meet the eastern polar winds along a boundary called the polar front. A cyclone with cool, unstable weather is installed along it. The polar front is not located in a straight line, it is curved in places where cold air penetrates in the direction of the equator, and warm goes to the pole. These movements create a vortex that is amplified by fast winds (jet currents) that blow the upper part of the troposphere.

It should be noted that in fact the three-cell meridional model of air circulation is complicated by the influence of the following factors:

- seasonal specifics, ie changes in the position of zones and the intensity of pressure in them, caused by the annual course of the sun to the north and south of the equator;

- geographical physical properties that cause differences and uneven distribution of water and land on the surface of the globe: in summer the air over the ocean is colder than over land, because the ocean surface heats up more slowly than the mainland; therefore, in summer the air is directed from the ocean to the land by the force of the baric gradient; in winter the air is colder over land and the air moves from land to ocean.

The movement of atmospheric air can be described as a superposition of interconnected flows, characterized by scales from 1 mm to thousands of kilometers. To analyze these movements, they are classified on a horizontal scale:

- microscale movement has a size of less than 20 km with a time scale of 1 hour;
- synoptic movement (from the Greek «synoptikos» – able to see everything) has a characteristic size of more than 500 km, a time scale of two days or more. Synoptic movements include the occurrence and movement of cyclones and anticyclones, air masses and atmospheric fronts. Synoptic movements, together with planetary ones, are macro-scale phenomena;
- mesoscale movement has intermediate parameters.

The variety of manifestations of the general circulation of the atmosphere especially depends on the fact that the atmosphere is constantly huge waves and vortices, which develop and move differently. This formation of atmospheric perturbations – cyclones and anticyclones – is the most characteristic feature of the general circulation of the atmosphere.

Anticyclone is a zone of high atmospheric pressure with descending air flows and a maximum in the center (1050 ... 1070 gPa). The diameter of the anticyclone can be thousands of kilometers. The anticyclone is characterized by a system of winds blowing clockwise in the Northern Hemisphere and counterclockwise in the Southern Hemisphere, by cloudy and dry weather with light winds. Some anticyclones occur in cold regions. The air here is dense, which creates an increase in pressure at the surface. Similar anticyclones develop in winter over central Canada and Siberia. As a rule, their depth does not exceed 3 km.

Cyclone (from the Greek «kyklon» - one that spins) is an area of low atmospheric pressure with a minimum in the center. The diameter of the cyclone is several thousand kilometers. It is characterized by a system of winds blowing counterclockwise in the Northern Hemisphere and clockwise in the Southern Hemisphere. The weather during cyclones is mostly cloudy with strong winds. A cyclone is formed when cold air follows warm air, and thus two fronts are formed inside the cyclone. The warm front separates the approaching warm air from the cold. In this case, warm air rises above the layer of cold dense air in front. In the ascending cooled air, water vapor condenses and forms clouds. The warm front is followed by the cold front. Along this front, cold air penetrates under a layer of warm air, forcing it to rise. Therefore, the cold front also carries cloudy, rainy weather. Often in front of the cold fronts a line of squalls develops, which can be accompanied by very intense thunderstorms and storms, sometimes the temperature changes. Mostly cyclones move in an easterly direction with speeds of about 20 km / h in summer and 50 km / h in winter.

Local circulations, such as breezes on the coasts of the seas, mountain-valley winds, glacial winds, etc., are distinguished from the general circulation of the atmosphere. These local circulations over time and in certain areas overlap with the general circulation.

11.2. Devices and equipment for research of parameters of an air stream

Accurate estimation of wind load requires reliable information about wind characteristics, such as speed, direction, frequency of winds of different speeds, speed distribution by height, amplitude and duration of gusts, etc. These data are studied in a special section of climatology – wind climatology, which provides the designer and compiler with all the necessary information to make correct and reasonable decisions about the value of wind loads used in the design of buildings and structures. The necessary information is obtained on the basis of wind observations and statistical data processing of meteorological stations, which receive information using instruments and control and measuring equipment.

Constant meteorological observations of the wind began to be conducted from the second half of the XIX century throughout Russia and Ukraine. At that time, the Wild weather vane was widespread (*Fig. 11.2*), which was used to determine data on wind speed and direction. Despite the ease of obtaining data, the Wild weather vane has a number of disadvantages: with increasing wind speed, the accuracy of measurements decreases sharply. In addition, we must take into account the subjective characteristics of the observer. With the development of technology more advanced devices began to use: cup and wing anemometers, anemorumbometers, anemorumbographs, anemometers with strain gauges (for measuring wind speed in gusts), "Holtzmann hurricane meter", ultrasonic anemometers, etc. (*Fig. 11.2*). These mechanical wind speed meters have become widespread due to their relatively high resistance to atmospheric factors: changes in temperature, pressure and precipitation. The only exceptions in this regard are ice deposits and frost, which lead to both changes in characteristics of meters (due to changes in their aerodynamic and dynamic properties), and to complete failure. In this regard, when conducting observations of wind speed in areas where the probability of the above-mentioned deposits is high, heated anemometers are used.

Mechanical anemometers are characterized by insufficient speed, which leads to distortion of measurement results in the registration of wind gusts, which significantly affect high-rise buildings. In this regard, there is a need for faster meters based on different physical principles. Among the potentially suitable for measuring wind speed are several methods: ultrasonic Doppler, laser Doppler, kinematic and thermoanemometric. The high cost of such meters, as well as the complexity of protecting their electronic transducers from climatic action (precipitation, temperature and humidity) limits their scope only to the conditions of the aerodynamic laboratory. An example of the implementation of the first of these methods is a highly computerized system Doppler SODAR: an installation for measuring horizontal and vertical wind speeds, as well as its direction.

Due to the relative cheapness and relatively high meteorological and operational characteristics, the thermoanemometric method has become widespread in the study of turbulent wind flows. An example is the TAIK-3M thermoanemometer manufactured by SKTB "Turbulence". Foreign researchers mainly use the thermoanemometric equipment of two companies, DANTEC and TSI, to measure the air flow rate.

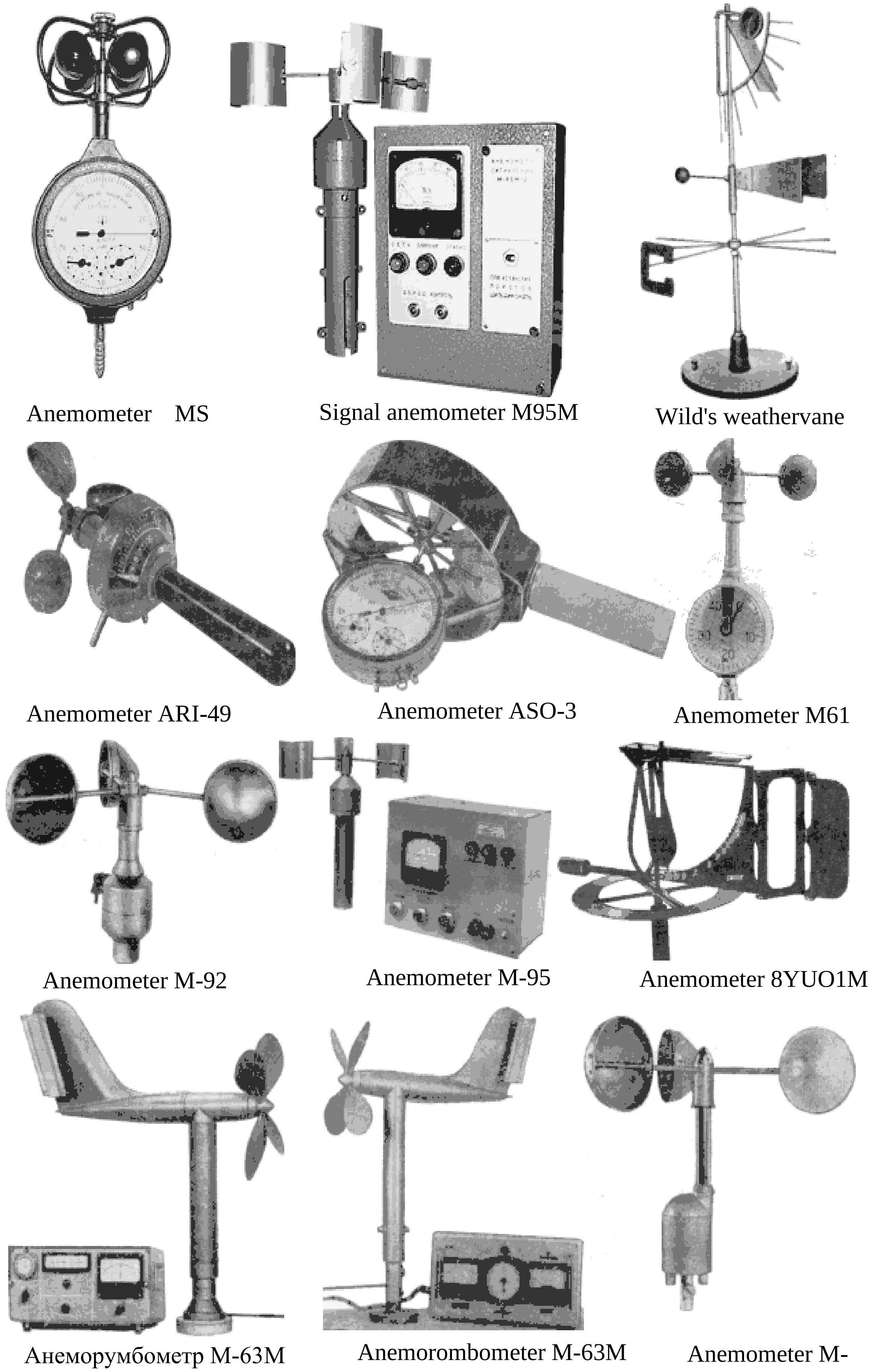

Fig. 11.2. Wind speed meters used in meteorology

Instruments for measuring wind speed at meteorological stations and posts are usually installed at a height of 8…12 m from the ground. Abroad, the starting height is 10 m (32.8 feet).

Measurement information is recorded using analog recorders (recorders, loop oscilloscopes, VCRs and tape recorders) and using digital information collection and processing systems based on specialized controllers and personal computers.

11.3. Meteorological Service of Ukraine

In the former USSR, from the century before last until 1936, weather information for climatological purposes and weather forecasts were obtained from meteorological stations, which conducted observations 3 times a day. From 1936 to 1965, regular wind observations at the meteorological stations of the Main Directorate of the Hydrometeorological Service of the USSR began to be conducted four times a day (at 3, 9, 15, and 21 o'clock) with a two-minute averaging period. Since 1966, observations have been conducted 8 times a day with a ten-minute averaging. Wind observations are also conducted by the Main Geophysical Observatory named A.I. Voeikov, Central Aerological Observatory, Central Altitude Hydrometeorological Observatory, Institute of Experimental Meteorology, regional Research Hydrometeorological Institutes, hydrometeorological centers, airports of various departments. Detailed materials on wind for each geographical area are collected in [8]. Data on wind speed and its recurrence are also given in the monograph R.I. Kinash [4].

There are currently about 240 meteorological stations in Ukraine. The method of collecting information about the parameters of wind flow used on them is focused on the use as primary measuring transducers of wind speed mechanical meters – anemorumbometers and weather vanes Wild (as backup) [7]. Mechanical speed meters (cup and wing anemometers) are basic and abroad. When using the anemorumbometer M-63M-1 (see *Fig. 11.2*), standard for Ukraine, the following parameters are measured on the network of climatic meteorological stations:

- maximum wind speed between measurements (ie with eight-term organization of wind speed observations in Ukraine – for the last 3 hours);
- average wind speed in 10 minutes;
- maximum wind speed in 10 minutes;
- average wind direction in 2 minutes.

In the case of measurements using the Wild weather vane (most of the data on wind speed at meteorological stations in the former USSR obtained with its help), the following parameters are recorded:

- maximum wind speed in 2 minutes;
- average wind speed in 2 minutes;
- average wind direction in 2 minutes.

As an example, in *Fig. 11.3* typical graphs of changes in wind speed are presented that are measured using an anemorumbometer and thermoanemometer.

The averaging intervals of 2 and 10 minutes are due, firstly, to a subjective reason – when determining the wind direction on the anemorumbometer (with selsen sensor) and speed on the weather vane, the operator visually averages the readings of the anemorumbometer or weather vane indicator. Secondly, it is determined by the parameters of the standard analog integrator of the device M-63M-1. Thus, at domestic meteorological stations discrete measurements are made, which give information about the averaged local parameters of wind current at the location of the meteorological station. The same situation is happening abroad.

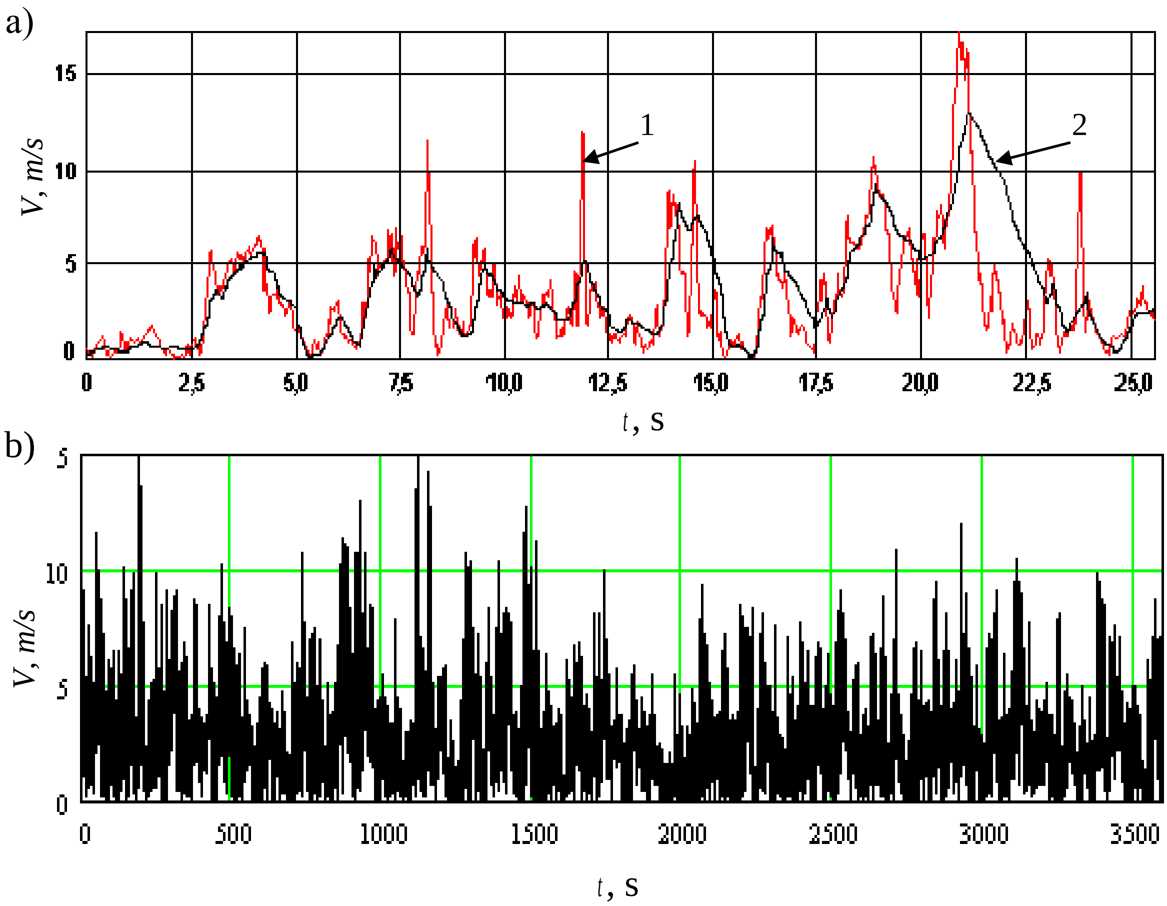

Fig. 11.3. Graphs of wind speed change:
a) comparative implementation with a length of 25 s; b) hourly implementation:
1 - measured by thermoanemometer; 2 - measured with an anemorumbometer

11.4. The spectrum of Van der Hoven

Since changes in wind speed and direction are mainly caused by changes in temperature, it can be expected that the wind has cycles corresponding to annual, meteorological (time about four days) and daily insolation cycles. This is clearly seen in the graph of the spectral density of wind speed (energy spectrum of the longitudinal component pulsations of wind speed) obtained by Van der Hoven [3] (*Fig. 11.4*).

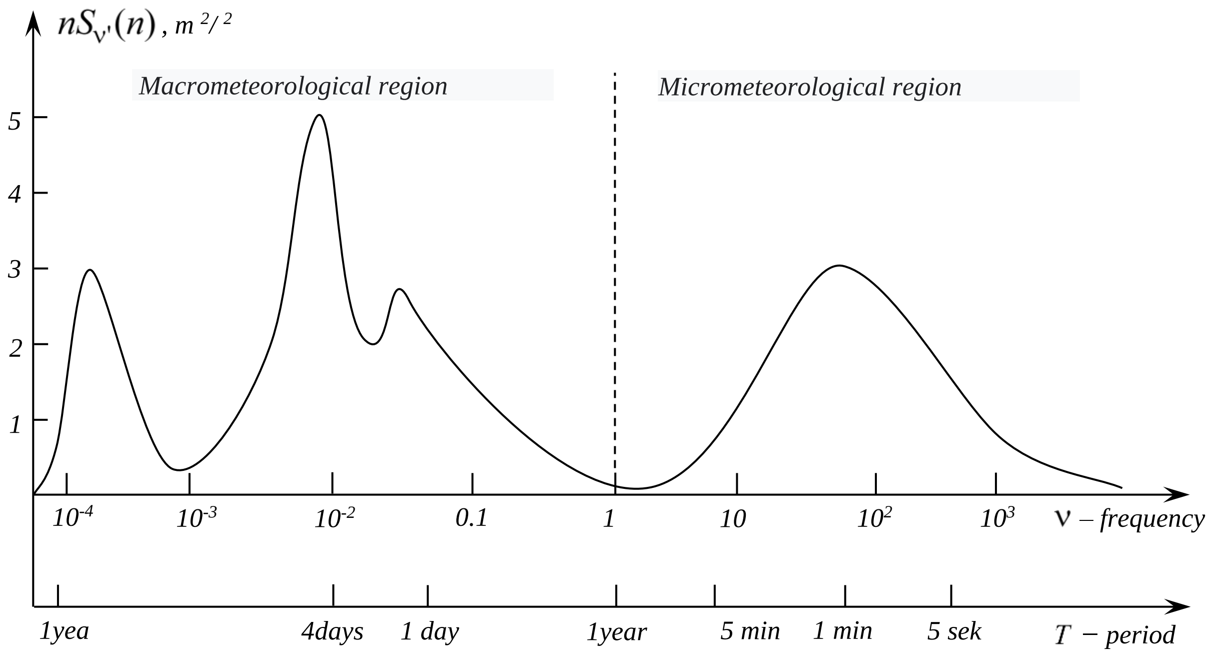

Fig. 11.4. The ripple spectrum of the velocity longitudinal component of Van der Hoven

The graph has two energy maxima:

a) left – synoptic, with a period of four days and partial highs at T = 1 year and T = 10 hours;

b) right – turbulent, with a period of T = 1 min, caused by friction of air flow on the earth's surface.

Between these two maxima at T = 5 hours...10 min there is a wide minimum, called spectral "failure". This feature of the wind spectrum allows:

a) divide the wind spectrum into two parts:

• micrometeorological (high frequency) with periods in seconds and minutes;

• macrometeorological (low frequency) with periods of tens and hundreds of hours;

b) present the wind load in the form of two components:

• static (average) component, which corresponds to the average wind speed;

• dynamic (pulsating) component, depending on the pulsating (turbulent) speed component;

c) find an averaging interval of longitudinal wind speed that significantly exceeds the period of turbulent pulsations, and at the same time not too large that long-period oscillations affect the average speed values obtained over this interval.

Hence the validity of a separate study of the mean and pulsation components of wind load and the choice of averaging time of 2, 5 or 10 minutes.

11.5. Vertical wind speed profile

Influence of friction in the boundary layer. Boundary layer of the atmosphere is adjacent to the earth's surface layer of the atmosphere to a height of about 1000 m, the properties of which are mainly determined by the dynamic and thermal actions of the earth's surface. The wind regime inside the boundary layer of the atmosphere is important for the design of industrial and civil buildings and structures.

The force of friction in the boundary layer of the atmosphere is caused by the fact that air flows over the rough earth's surface, and the speed of air particles directly in contact with the earth's surface slows down. Particles with reduced velocity in the process of turbulent exchange are transferred to the upper layers, and from above they are replaced by particles with higher velocity, which in turn slowdown in contact with the earth's surface. Thus, due to turbulence, the decrease in velocity is transmitted upwards to a sufficiently powerful layer of the atmosphere.

The presence of friction forces leads to the fact that the wind speed changes with height. This effect is so great that at the earth's surface (at the height of the anemometer) the wind speed is about twice less than the speed of the gradient (geostrophic) wind acting above the boundary layer, calculated for the same pressure gradient. In addition, wind speed is strongly influenced by the terrain, as well as natural and man-made obstacles located in the direction of its flow.

Currently, there are two proposals to describe the dependence of wind speed on altitude, known as logarithmic and power laws.

Logarithmic law. The profile of the average wind speed at the logarithmic law is written in the form:

$$\frac{\bar{v}(z_2)}{\bar{v}(z_1)} = \frac{ln(z_2/z_0)}{ln(z_1/z_0)}, \tag{11.1}$$

where $\bar{v}(z_1)$ and $\bar{v}(z_2)$ are the average wind speeds at altitude z_1 and z_2 respectively; z_0 is the parameter of the roughness of the underlying surface.

If the standard height of the anemometer (z_1= 10 m) is taken as the height z_1, then expression (11.1) can be rewritten as:

$$\bar{v}(z) = \bar{v}_{10}\,\frac{ln(z/z_0)}{ln(10/z_0)}. \tag{11.2}$$

This view can be found in the works of a number of researchers of wind load [7]. Graphical interpretation of formula (11.2) is shown in *Fig. 11.5* when varying $\bar{v}_{10}$ and z_0.

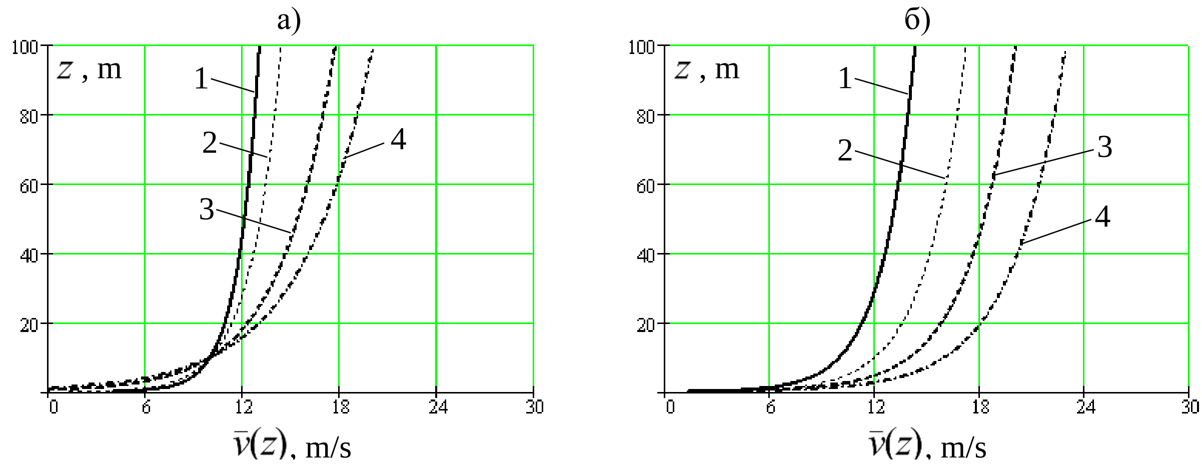

Fig. 11.5 Vertical profiles of wind speed at the logarithmic law:

a) for different types of underlying surface (at $\bar{v}_{10} = 10$ m / s):

1 − z_0 = 0,005 m; 2 − z_0 = 0,05 m; 3 − z_0 = 0,5 m; 4 − z_0 = 1,0 m;

b) for different wind velocities (at z_0 = 0,05 m):

1 − $\bar{v}_{10}$ = 10 m/s; 2 − $\bar{v}_{10}$ = 12 m/s; 3 − $\bar{v}_{10}$ = 14 m/s; 4 − $\bar{v}_{10}$ = 16 m/s

Logarithmic law has long been considered by meteorologists the best description of strong wind profiles in the lower atmosphere (in the surface layer). In particular, in the wind codes of the USSR [2] the profile of high-speed wind pressure is adopted by the logarithmic law

$$q_{0j} = q_0 \frac{ln^2(z_j/z_0)}{ln^2(10/z_0)}, \tag{11.3}$$

where z_0 is the roughness parameter taken to the mark of 40 m – 0,075 m, above – 0,05 m.

Meanwhile, according to research by M.V. Zavarina [7], the logarithmic law well approximates the wind profile at speeds not exceeding 8 m / s.

Degree law. Historically, the first-degree law was proposed in 1916 in the form

$$\bar{v}(z_2) = \bar{v}(z_1)(z_2/z_1)^{\alpha}, \tag{11.4}$$

where z_1, z_2 are heights above the ground; α is the index of degree, depending on the roughness of the terrain.

Strictly speaking, the degree α depends not only on the roughness of the terrain, but also on wind speed, stratification and turbulence of the atmosphere.

Taking in the formula (11.4) as z_1 a standard installation height of the anemometer (z_1= 10 m), we obtain

$$\bar{v}(z) = \bar{v}_{10}(z/10)^{\alpha}. \tag{11.5}$$

Fig. 11.6 shows the characteristic vertical profiles of wind speeds for three types of terrain and different values $\bar{v}_{10}$.

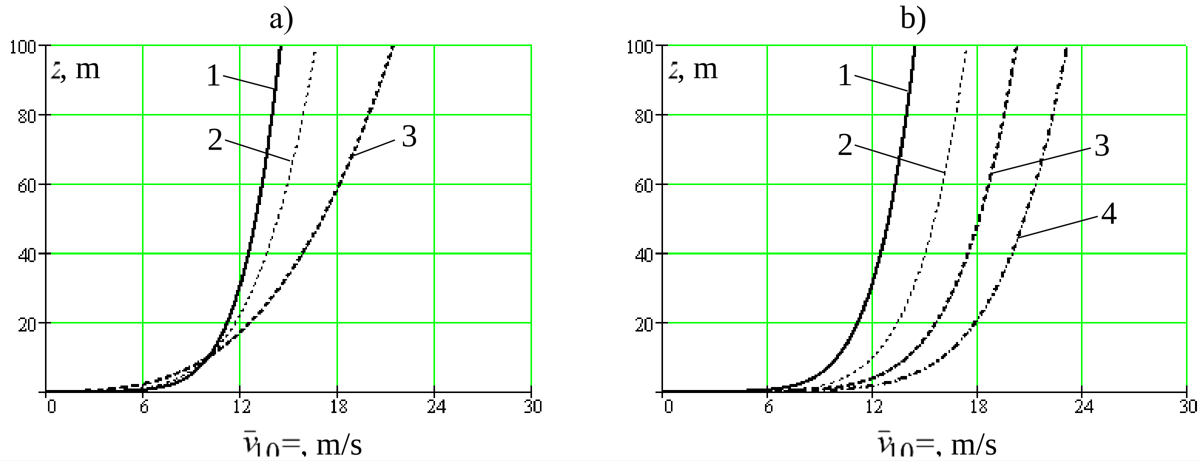

Fig. 11.6. Vertical profiles of wind speed at the degree law
a) for different types of underlying surface (at $\bar{v}_{10}$ =10m/s):
1 − α =0,16; 2 − α =0,22; 3 − α =0,33;
b) for different wind velocities (at α =0,16):
1 − $\bar{v}_{10}$ = 10 m/s; 2 − $\bar{v}_{10}$ = 12 m/s; 3 − $\bar{v}_{10}$ = 14 m/s; 4 − $\bar{v}_{10}$ = 16 m/s

Due to the quadratic transition from wind speed to wind pressure, a dependence similar to expression (11.4) is proposed for speed pressure:

$$q(z_2) = q(z_1)(z_2/z_1)^{\chi}, \tag{11.6}$$

where $q(z_2)$ and $q(z_1)$ are the velocity pressures at height z_2 and z_1; $\chi = 2\alpha$ is degree indicator (for open flat terrain for most of the territory of the former USSR, the average value of the degree indicator is $\chi = 0{,}25$).

The degree law is most widespread in Eastern Europe, in particular, it describes the vertical profile of wind speed and velocity in SNiP [2].

Comparison of logarithmic and degree laws. Analysis of numerous observational materials (performed on masts by aerological methods) showed that the profiles of average wind speeds up to a height of 300 m are somewhat more accurately approximated by the degree function than by the logarithmic one. In the surface layer of the atmosphere (up to a height of 15… 20 m) the profile of the average wind speed is quite well described by the logarithmic function. In addition, the logarithmic law is now considered by Western meteorologists to be the best description of strong wind profiles in the lower atmosphere, according to the power law is rarely used in Western countries in the practice of meteorological research [6].

The conclusion as to which law best corresponds to the experimental data does not have an unambiguous answer. In each case, for a specific area and a given altitude range, a satisfactory description of the vertical wind speed profile can be achieved both on the basis of logarithmic and power law. However, in connection with the use of logarithmic law by Eurocode [9], as well as the transition to this law of Ukrainian codes [1], it can be considered justified to use the logarithmic law in the practice of construction design.

Table 11.1

Roughness parameter values for different surface types [6]

Surface type	z_0,*sm*
Sand	*0.01−0.1*
Sea surface	*0.0003−0.5*
Snow cover	*0.1−0.6*
Mown grass (~ 0.01 m)	*0.1−1.0*
Low grass (steppe)	*1.0−4.0*
The field is plowed under steam	*2.0−3.0*
Tall grass	*4.0−10.0*
Dwarf plants	*10.0−30.0*
Small forests (average height of trees 15 m, one tree per 10 m^2)	*90.0−100.0*
Suburbs with sparse buildings	*20.0−40.0*
Cities, suburbs with solid buildings	*80.0−120.0*
Centers of large cities	*200.0−300.0*

Surface roughness of the surrounding area. Surface roughness is a set of natural and artificial features of the terrain that characterize the average and maximum height of irregularities and their width, the average distance between them and so on.

The influence of this factor on the wind speed in the logarithmic law (11.2) is estimated by the roughness parameter z_0, ie the value proportional to the average size of the irregularities rising above the underlying surface. The wind speed at altitude z_0 is zero. Typical values z_0 for different types of surfaces are given in *Table 11.1*. With the degree law, the roughness of the underlying surface affects the value of the parameter α. Values α vary widely; over flat open terrain they are significantly smaller ($\alpha = 0{,}14\ldots0{,}16$) than over rugged and heterogeneous terrain: for suburbs $\alpha = 0{,}22\ldots.0{,}28$, for the centers of large cities $\alpha = 0{,}33\ldots.0{,}40$.

11.6. Issues of construction aerodynamics

Aerodynamics is a branch of aeromechanics in which the laws of motion of a liquid or gas (in particular, air) and the forces arising on the surface of a gas-flowing body are studied. The main tasks of aerodynamics are to determine the forces acting on the gas-flowing body, the distribution of pressure on its surface and velocities in the gas surrounding it. Aerodynamics of buildings and structures refers to the aerodynamics of bad-optical bodies.

The theory of flow around bodies with simple geometric parameters is the basis for solving the laws of aerodynamics of more complex forms. When considering them, a universal parameter is used – the Reynolds number:

$$Re = \frac{vL}{\nu}, \tag{11.7}$$

where v is the characteristic flow Velocity; L is the characteristic size of the flow; ν is kinematic viscosity of air. For air at normal atmospheric pressure and temperature 20ºC $\nu = 1.512 \times 10^{-5}\,\text{m}^2\,/\,\text{s}$.

Consider first a rigid plate with sharp edges (plate thickness not more than 0.01… 0.02 of its width), placed in a stream of real atmosphere. The plate is an elementary body, the study of whose behavior facilitates understanding of the mechanism of phenomena occurring with more complex bodies. At the angle of attack $\alpha = 90°$ and Reynolds numbers up to *Re* = 1000, ie at very low flow velocities, the resistance of the plate strongly depends on the Reynolds number, in which the characteristic size of the plate *h* means its width. The evolution of the flow of a plate with sharp edges is shown in *Fig. 11.7* and includes the following stages.

• $Re \approx 0{,}3$. The flow wraps around sharp corners and follows inseparably along both the front and back sides of the plate contour (*Fig. 11.7, a*).

• $Re \approx 10$. Increasing the velocity of the air flow (hence the Reynolds number) leads to the disruption of the flow jets at the corners of the plate, as well as the formation of two large symmetrical vortices behind it. The vortices remain attached to the back of the plate (*Fig. 11.7, b*).

• $Re \approx 250$. A further increase in the Reynolds number leads to the destruction of symmetric vortices and their replacement by the correct sequence of vortices, which alternately form at the upper and lower edges of the plate and are carried downstream (*Fig. 11.7, c*).

• $Re \geq 1000$. At these values, the Reynolds number is dominated by the forces of inertia. Large isolated vortices have little opportunity for their formation, and instead of them a *turbulent* associated jet is formed behind the plate. The two outer edges of the plate form a "shear layer" consisting of long chains of the smallest vortices. They are located in the part of the associated jet, which is adjacent to the smooth flow (*Fig. 11.7, d*).

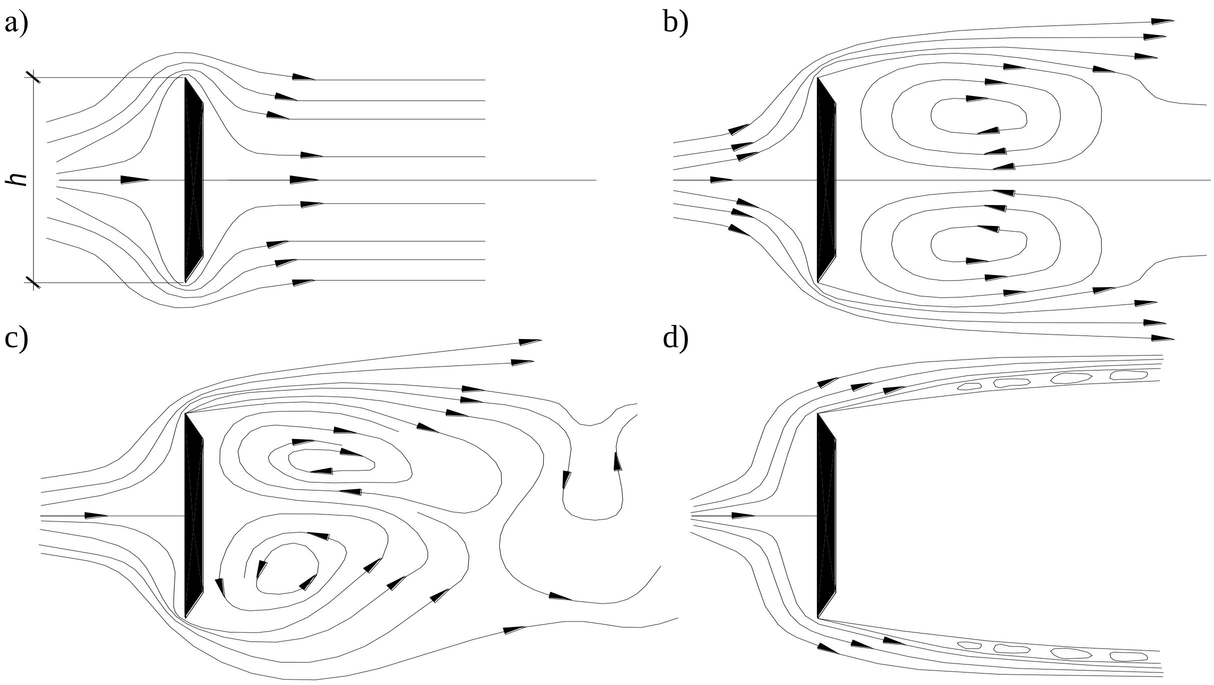

Fig. 11.7. Flow of a plate with sharp edges:
a) – Re ≈ 0.3; b) – Re ≈ 10; c) – Re ≈ 250; d) – Re ≥ 1000

Consider further the case of a flat flow surrounding a circular cylinder. As in the previous case, an increase in the Reynolds number will be equated with an increase in air flow velocity. Let's analyze five possible ranges of Reynolds number change.

• $Re \approx 1$. The flow which is provided at the approach to the cylinder laminar, remains connected to the cylinder on all its perimeter (*Fig. 11.8, a*).

• $Re \approx 20$. The flow remains symmetrical, but there is a separation of the flow and the formation of large vortices in the accompanying jet, which are located near the rear surface of the cylinder (*Fig. 11.8, b*).

• $30 \leq Re \leq 5000$. Properly alternating vortices break away from the cylinder, forming a clearly defined "vortex path" downstream (*Fig. 11.8, c*). Behind the cylinder is a stable system of vortices arranged in a checkerboard pattern, which move downstream at a speed slightly less than the speed of the surrounding air. In this range of Reynolds numbers, the flow of the accompanying jet is quite smooth and regular.

• $5000 \leq Re \leq 20000$. A continuous laminar flow of the cylinder is before the break point. In the separated stream, a spatial picture of movements is observed, and in the accompanying stream there is a transition to turbulent flow (*Fig. 11.8, d*).

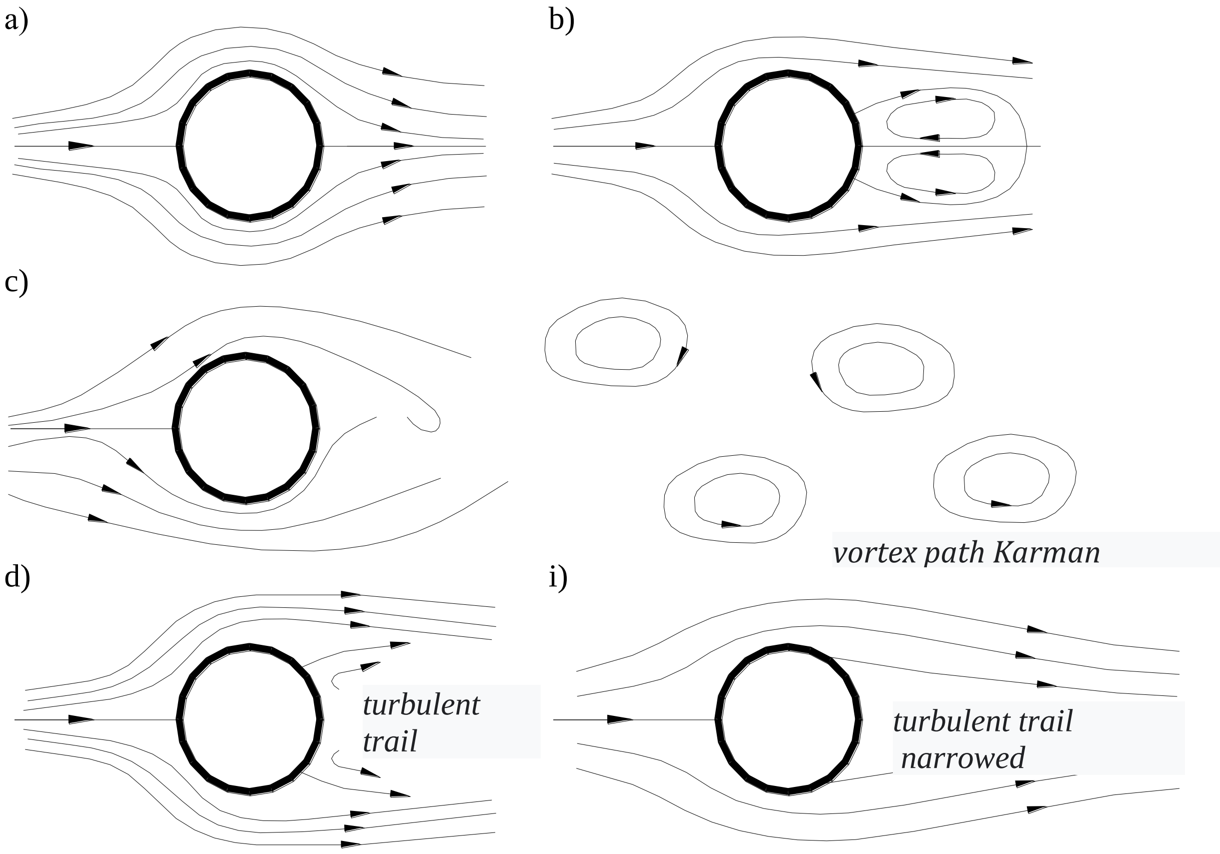

Fig. 11.8. Flow of the circular cylinder:
a) − Re ≈ 1; b) − Re ≈ 20; b) − 30 ≤ Re ≤ 5000; d) − 5000 ≤ Re ≤ 200000; i) − Re ≥ 200000

• $Re \approx 200000$. The associated jet is noticeably narrower, and the failure of the vortices is more accidental. With increasing velocity at $Re \approx 4 \cdot 10^6$, the vortex failure becomes regular again, despite the fact that the associated jet now retains a significant degree of turbulence (*Fig. 11.8, i*). The highest value of *Re*, to which experimental studies of this phenomenon were conducted, is approximately 10^8.

It should be noted that other bad-optical bodies, especially triangular, square, rectangular and other prisms of correct and incorrect geometric shape, cause similar phenomena of vortex failure [51].

The resulting associated jet is affected not only by the poor optical frontal surface of the body, but also by the length of the body in the direction of flow and its overall shape. When comparing the flow of square and rectangular prisms (*Fig. 11.9*) it is seen that the square cross section (with a sufficiently large *Re*) causes the separation of the flow, accompanied by the appearance of a wide turbulent associated jet, while more elongated rectangular shape (depending on length to width)) the separation of the flow can occur in the front corners, followed by the downstream restoration of continuous flow and another separation of the flow at the trailing edge.

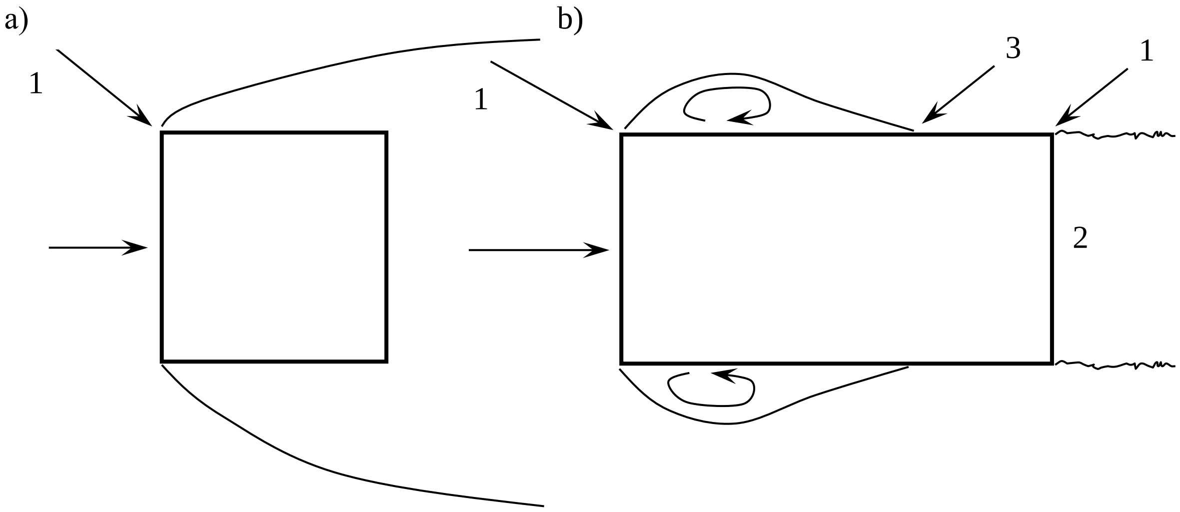

Fig. 11.9. Flow around square (a) and rectangular (b) obstacles:
1 - separation; 2 - associated jet; 3 - reconnection

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Control questions

1. How is the circulation of atmospheric air?
2. The role of cyclones and anticyclones in the formation of air movement.
3. What devices are used for wind measurements?
4. How are meteorological measurements of wind organized?
5. What is the shape of the van der Hoven spectrum?
6. How is the vertical profile of wind speed described?
7. How is the wind flow of structures?

LECTURE 12. STANDARDIZATION OF WIND LOAD

The requirements of DBN [1] (Chapter 9) apply to buildings and structures of simple geometric shape, **the height of which does not exceed 200 meters**.

When determining the wind load for buildings and structures of complex structural or geometric shape (including cable and hanging surfaces, shells, antenna sheets), steel lattice masts and towers, etc., as well as for buildings and structures over 200 meters high the special dynamic calculations for determining the effect of the pulsating component of the load should be performed and in necessary cases blowing of models in a wind tunnel should be carried out.

12.1. Standardization of wind load according to DBN

Wind load is a variable load for which two calculated values are set:

- limit design value;
- operational design value.

The wind load on the structure should be considered as a set of:

a) the normal pressure applied to the outer surface of the structure or element;

b) friction forces directed tangentially to the outer surface and related to its horizontal (for shed or wavy roofs, roofs with lanterns) or vertical (for walls with loggias and similar structures) projection;

c) normal pressure applied to the interior surfaces of buildings with breathable enclosures, with openings or permanent openings.

The set of these forces can be represented in the form of normal pressure due to the total resistance of the structure in the direction of the x and y axes and conditionally applied to the projection of the structure on a plane perpendicular to the corresponding axis.

12.2. Design values of wind load

The limit design value of wind load is determined by the formula

$$W_m = \gamma_{fm} W_0 C \, , \qquad (12.1)$$

where γ_{fm} is the reliability factor for the maximum calculated value of wind load;

W_0 is the characteristic value of wind pressure (in Pa);

C is the coefficient determined by formula (12.3).

The operational design value of wind load is determined by the formula

$$W_e = \gamma_{fe} W_0 C, \tag{12.2}$$

where γ_{fe} is the reliability factor for the operational design value of wind load, determined according to *Table 12.5*.

The characteristic value of wind pressure W_0 is equal to the average (static) component of wind pressure at a height of 10 m above the ground, which can be exceeded on average once every 50 years.

The characteristic value of wind pressure W_0 is determined depending on the wind region on the map (*Fig. 12.1*) or in Annex E [1] (5 regions instead of 3 according to SNiP [2].

Note. If necessary, W_0 may be determined by statistical processing of the results of term wind speed measurements.

12.3. Coefficients of wind load calculation method

The coefficient C is determined by the formula

$$C = C_{aer} C_h C_{alt} C_{rel} C_{dir} C_d , \tag{12.3}$$

where C_{aer} is the aerodynamic coefficient;

C_h – coefficient of building height;

C_{alt} – coefficient of latitude;

C_{rel} – relief coefficient;

C_{dir} – direction coefficient;

C_d – dynamic coefficient.

Aerodynamic coefficients C_{aer} take into account the nature of wind blowing. According to the codes of DBN aerodynamic coefficients C_{aer} are determined according to paragraph 12.7 (Annex I [1]) depending on the shape of the structure or structural element and may be:

- coefficients of C_e, which should be taken into account when determining the wind pressure normally applied to the outer surfaces of the structure or element and attributed to the unit area of this surface;

- coefficients of friction C_f, which should be taken into account when determining the friction forces directed tangential to the outer surface of the structure or building and related to the area of its horizontal or vertical projection;

- coefficients C_i to be taken into account when determining the wind pressure normally applied to the interior surfaces of buildings with permeable barriers, with openings or permanent openings;

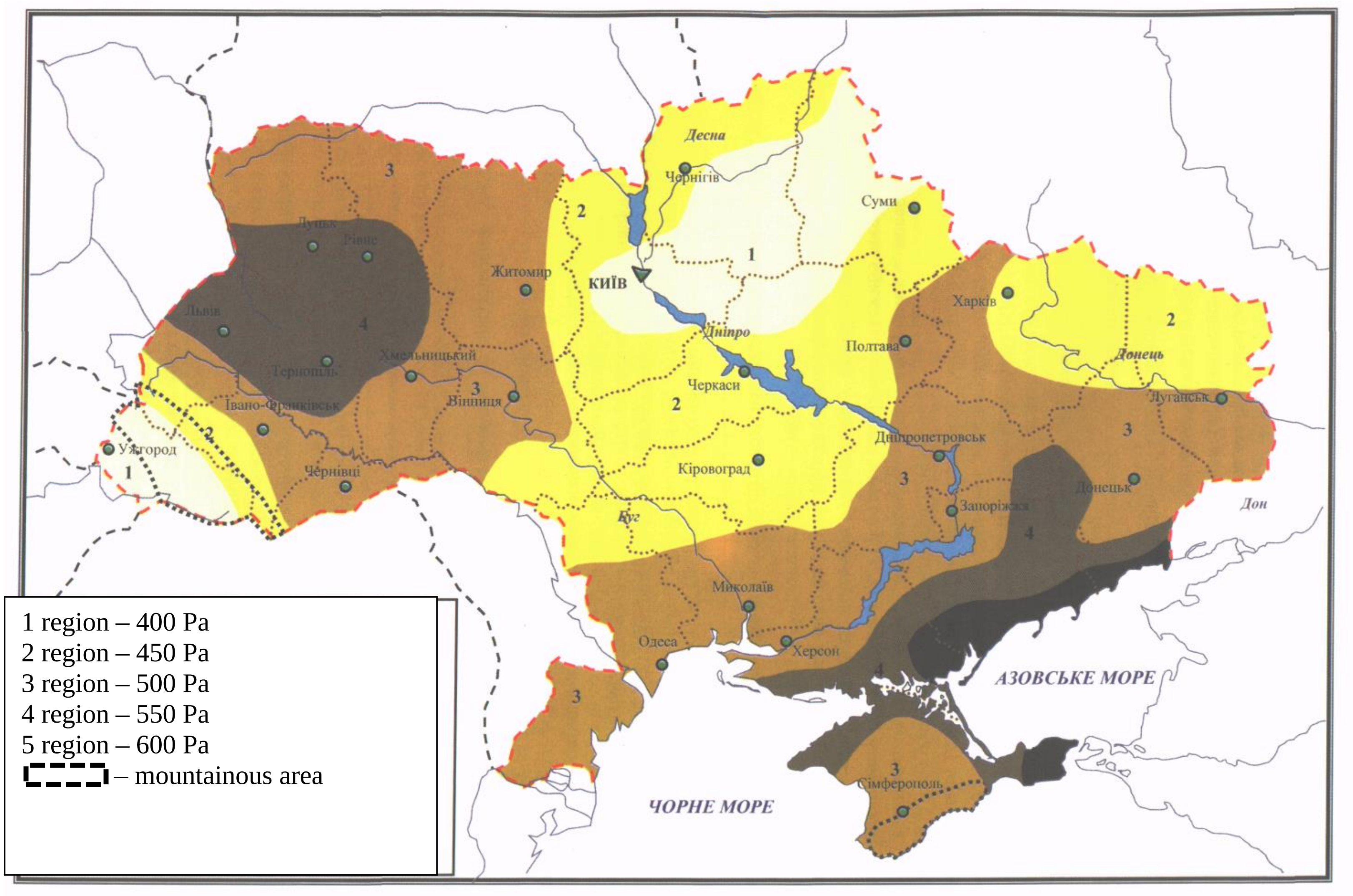


Fig. 12.1. Map of zoning of the territory of Ukraine by characteristic values of wind pressure

- drag coefficients C_x, which should be taken into account for individual elements and structures when determining the component of total body resistance that acts in the direction of wind flow and is related to the projection area of the body on a plane perpendicular to the flow;
- coefficients of transverse force C_y, which should be taken into account for individual elements and structures when determining the component of total body resistance that acts perpendicular to the wind flow and is related to the plane of projection of the body on the flow plane.

The coefficient of building height C_h takes into account the increase in wind load with height. In the DBN codes, the coefficient C_h takes into account the height of the structure or its part under consideration above the ground surface (Z), the type of surrounding terrain, it is determined according to *Table 12.1* for buildings and structures, the senior period of natural oscillations of which does not exceed 0.25s, and according to *Table 12.2* for all other buildings and structures.

Table 12.1

Coefficients of building height (for $T \leq 0{,}25$ s)

Z(m)	*C_h for terrain type*			
	I	*II*	*III*	*IV*
≤ 5	0,90	0,7	0,40	0,20
10	1,20	0,90	0,60	0,40
20	1,35	1,15	0,85	0,65
40	1,60	1,45	1,15	1,00
60	1,75	1,65	1,35	1,10
80	1,90	1,75	1,50	1,20
100	1,95	1,85	1,60	1,25
150	2,15	2,10	1,85	1,35
200	2,30	2,20	2,05	1,45

Table 12.2

Coefficients of building height (for $T > 0{,}25$ s)

Z(m)	*C_h for terrain type*			
	I	*II*	*III*	*IV*
≤ 5	1,40	1,20	0,90	0,60
10	1,80	1,50	1,20	1,00
20	1,95	1,85	1,55	1,40
40	2,25	2,20	2,00	1,95
60	2,45	2,45	2,25	2,15
80	2,65	2,60	2,45	2,30
100	2,70	2,70	2,60	2,40
150	2,95	3,00	2,90	2,60
200	3,10	3,20	3,20	2,80

Intermediate values of the coefficient C_h should be determined by linear interpolation.

The types of terrain surrounding the building or structure are determined for each calculated wind direction separately:

I - open surfaces of seas, lakes, as well as flat plains without obstacles exposed to wind on a section of at least 3 km;

II - rural areas with fences, small buildings, houses and trees;

III - suburban and industrial zones, long forests;

IV - urban areas where at least 15% of the surface is occupied by buildings with an average height of more than 15 m.

When determining the type of terrain, the structure is considered to be located on the terrain of this type for a certain calculated wind direction, if in this direction such terrain is at a distance of 30Z at full height Z <60 m or 2 km – at higher altitudes.

Note. If the building is located on the border of different types of terrain or there are doubts about the choice of terrain type, you should take the type of terrain that has a higher value of the coefficient C_h.

The coefficient of altitude C_{alt} takes into account the height H (in kilometers) of the location of the construction site above sea level and is calculated by the formula

$$C_{alt} = 2H \ (H > 0{,}5 \text{ km}); \ C_{alt} = 1 \ (H \leq 0{,}5 \text{ km}). \tag{12.4}$$

Note. Formula (12.4) is used for objects located in mountainous terrain, and gives an approximate value in the safety margin. In the presence of the results of meteorological observations of the wind conducted in the area of the construction site, the characteristic value of wind load is calculated by statistical processing of the results of term measurements of wind speeds and C_{alt}=1.

The relief coefficient C_{rel} takes into account the microrelief of the area near the terrain of the construction site and is assumed to be equal to one, except when the construction site is located on a hill or slope. In these cases, the wind speed may increase (*Fig. 12.2*).

The relief coefficient should be taken into account when the building is located on a hill or slope at a distance from the beginning of the slope of not less than half the length of the slope or one and a half height of the hill.

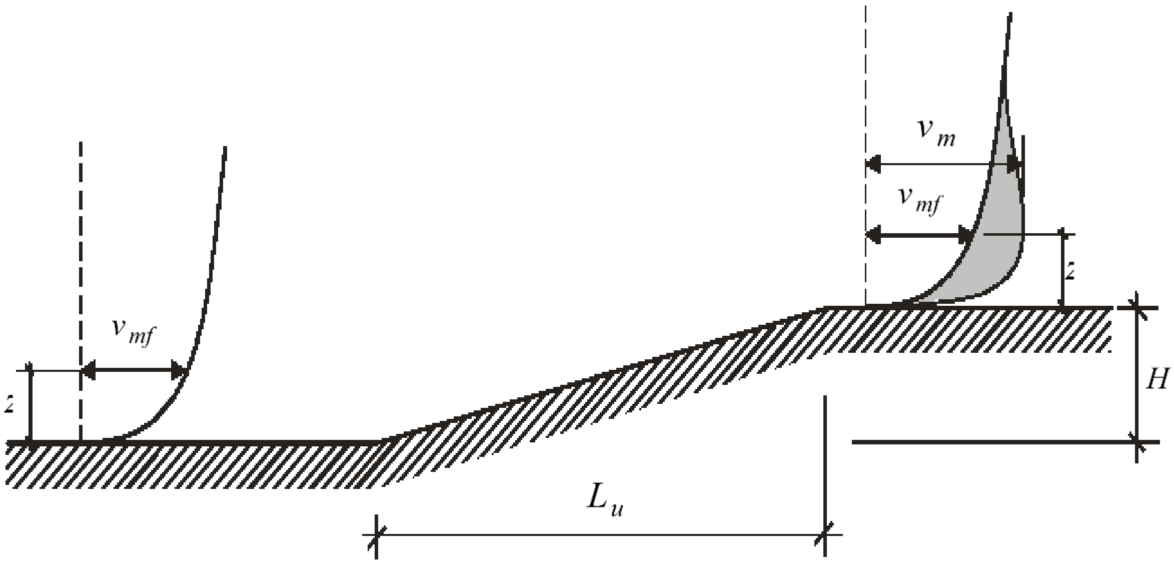


Fig. 12.2. Increasing wind speed over the slope
v_{mf} – average wind speed over the plains;
v_m – average wind speed in the place of change of relief
.

The relief coefficient C_{rel} is determined by formulas

$$C_{rel} = 1 \quad at \quad \varphi < 0{,}05;$$
$$C_{rel} = 1 + 2S\varphi \quad \text{at} \quad 0{,}05 < \varphi < 0{,}3;$$
$$C_{rel} = 1 + 0{,}6 \quad \text{at} \quad \varphi > 0{,}3. \tag{12.5}$$

Formulas (12.5) indicate:
φ – slope on the leeward side;
S – coefficient determined by *Fig. 12.3* for slopes and in *Fig.12.4* for hills.
In *Fig. 12.3* and *12.4* the following notations are given:
φ is H / L slope on the leeward side;
L_u is projection of the length of the leeward slope on the horizontal;
L_d is projection of the length of the windward slope on the horizontal;
H is height of the hill or slope;
X is the horizontal distance from the structure to the top;
Z is the vertical distance from the ground to the structure;
L_e is the effective length of the leeward slope ($L_e = L$ at $0.05 < \varphi < 0.3$; $L_e = 3.3H$ at $\varphi > 0.3$).

The direction coefficient C_{dir} takes into account the unevenness of the wind load in the wind directions and is usually assumed to be equal to one. C_{dir} values other than one may be taken into account for special justifications only for open plains and with sufficient statistics.

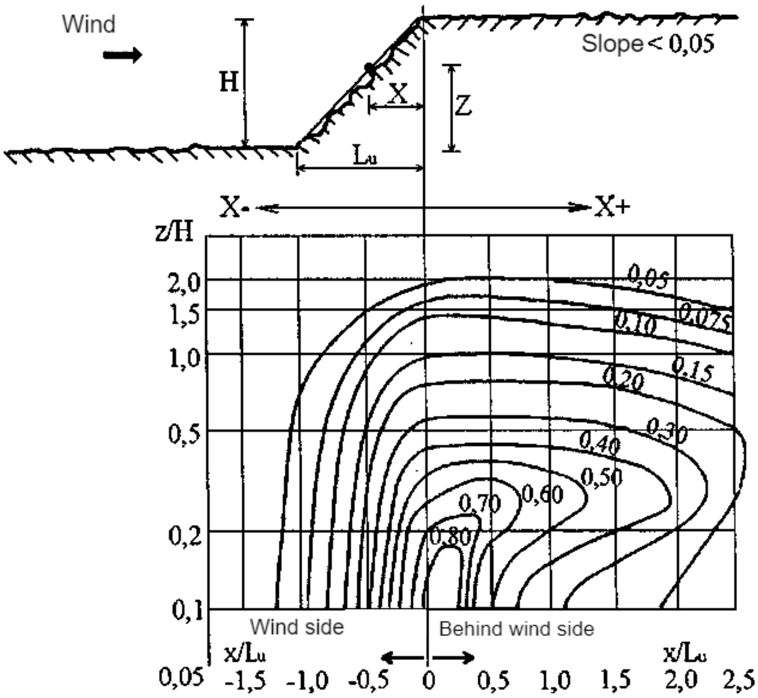


Fig. 12.3. Coefficient *S* for slopes

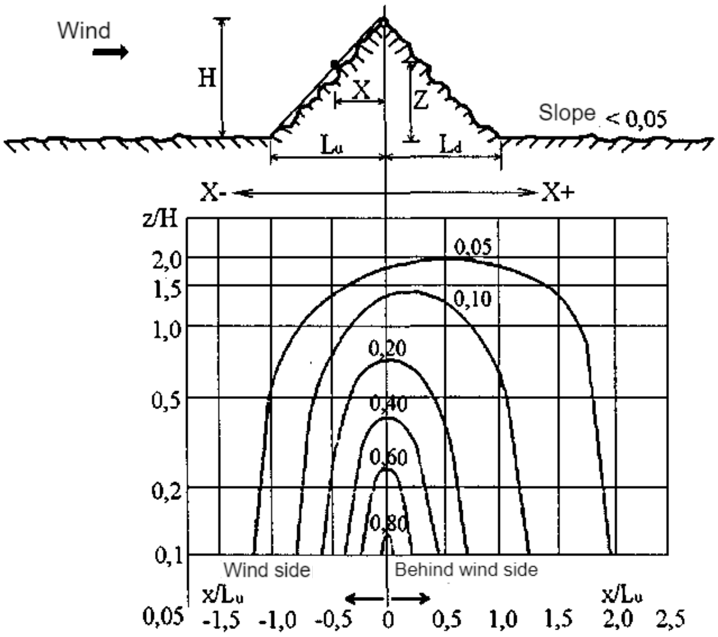


Fig. 12.4. Coefficient *S* for hills

The dynamic coefficient C_d takes into account the influence of the pulsating component of wind load and the spatial correlation of wind pressure on the structure. For buildings and structures with a higher oscillation period not exceeding 0.25 sec, C_d = 1. For the main types of buildings and structures, the oldest period of oscillations of which exceeds 0.25 sec, the values of C_d are determined according to the graphs given in the DBN (*Fig. 9.5... 9.10* [1]). Examples of graphs are shown below in *Fig. 12.5… 12.7*. The width and diameter shown in the figures are taken in a cross-section perpendicular to the wind flow. The C_d value should be taken from the left curve of the corresponding graph.

In cases where C_d > 1,2, it is necessary to perform a special dynamic calculation, which determines the effect of the pulsating component of wind load.

Notes. Values C_d <1.0 take into account the low probability of simultaneous increase in pulsation pressure at all points of the structure.

To test the strength of enclosing structures that are exposed to direct wind and have an area of less than 36 m^2, you should take $C_d \geq 1,0$.

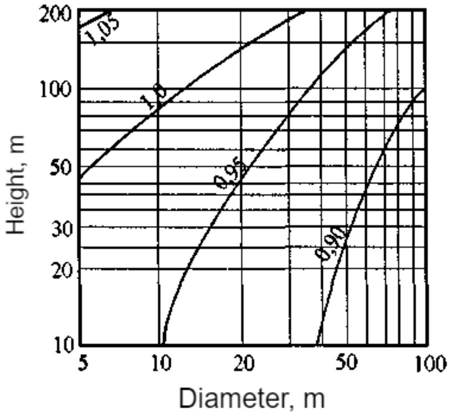


Fig. 12.5. C_d coefficient for stone buildings and buildings with reinforced concrete frame

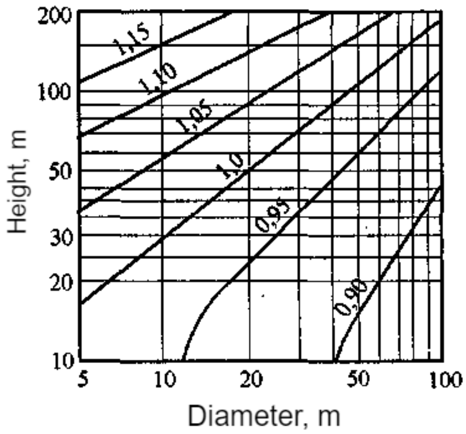


Fig. 12.6. C_d coefficient for buildings with steel frame

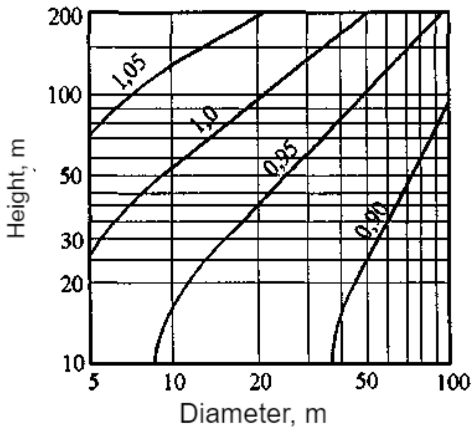


Fig. 12.7. C_d coefficient for buildings with steel-concrete frame

10.3. Reliability coefficients according to the values of wind load

The reliability coefficient γ_{fm} for the limit design value of the wind load is determined depending on the specified average recurrence period *T* according to *Table 12.3*.

Table 12.3

The values of the reliability coefficient γ_{fm}

T, years	***5***	***10***	***15***	***25***	***40***	***50***	***70***	***100***	***150***	***200***	***300***	***500***
γ_{fm}	*0,55*	*0,69*	*0,77*	*0,87*	*0,96*	*1,00*	*1,07*	*1,14*	*1,22*	*1,28*	*1,35*	*1,45*

Intermediate values of the coefficient γ_{fm} should be determined by linear interpolation.

For objects of mass construction, it is allowed to take the average recurrence period *T* as equal to the established service life of the structure T_{ef}.

For objects with a high level of responsibility, for which the technical task sets the probability *P* of not exceeding (providing) the limit value of the wind load during the specified service life, the average recurrence period of the limit value of the wind load is calculated by the formula

$$T = T_{ef} K_p, \qquad (12.6)$$

where K_p is the coefficient determined according to *Table 12.4* depending on the probability of *P*.

Table 12.4

The values of the coefficient K_p

P	***0,37***	***0,5***	***0,6***	***0,8***	***0,85***	***0,9***	***0,95***	***0,99***
K_p	*1,00*	*1,44*	*1,95*	*4,48*	*6,15*	*9,50*	*19,50*	*99,50*

The reliability coefficient $\boldsymbol{\gamma_{fe}}$ for the operational design value of the wind load is determined according to *Table 12.5* depending on the proportion of time η during which the conditions of the second limit state may be violated.

Table 12.5

The values of the coefficient γ_{fe}

$\boldsymbol{\eta}$	***0,002***	***0,005***	***0,01***	***0,02***	***0,03***	***0,04***	***0,05***	***0,1***
γ_{fe}	*0,42*	*0,33*	*0,27*	*0,21*	*0,18*	*0,16*	*0,14*	*0,09*

Intermediate values of the coefficient γ_{fe} should be determined by linear interpolation.

The value η is accepted according to the design codes or is set by the design task depending on their purpose, responsibility and consequences of exceeding the limit state. For objects of mass construction, it is allowed to accept η = 0,02.

12.4. Probabilistic substantiation of wind load codes

The development and publication of the State Codes of Ukraine DBN B.1.2-2006 "Loads and loadings" [1] in terms of wind loads was preceded by many years of work by Ukrainian researchers, including A.V. Perelmuter and M.A. Mikitarenko (VAT UkrNIIProektstalkostruktsiya named after V.M. Shimanovsky), V.A. Pashinsky, S.F. Pichugin, A.V. Makhinko (Poltava National Technical University named after Yuri Kondratyuk), R.I. Kinash (Lviv Polytechnic State University) and others [3, 4, 5].

The results of term measurements of wind speed and direction performed by anemorumboometers at 195 meteorological stations of Ukraine during 1970-1990 were used for statistical research and normalization of wind load. In total, a representative sample of more than 12 million results of term wind observations was used to normalize Ukraine's wind load. Wind measurements are carried out at a standard height of 10 m 8 times a day, during which wind speed measurements are performed with ten-minute averaging, which makes it possible to determine the wind pressure (*Pa*) according to the known formula:

$$W_0 = 0{,}61 v_m^2 \tag{12.7}$$

The obtained results indicate significant territorial variability of wind load, which differs significantly from its overly generalized standardization of SNiP [2], according to which almost the entire territory of Ukraine belonged to II (specified load W_0 = 0.3 kPa, design load 0.42 kPa) and III (W_0 = 0.38 kPa, design load 0.53 kPa) wind regions. More detailed territorial zoning of Ukraine according to the characteristic values of wind load includes five territorial regions with characteristic values from 0.4 to 0.6 kPa (*Fig. 12.1*). From the map of *Fig. 12.1* it can be seen that the lowest values of wind load are observed in the central and north-western regions of Ukraine, as well as in Transcarpathia. High wind loads are realized in the Carpathians, Prykarpattia and coastal areas.

Comparison of wind zoning according to DBN [1] with SNiP [2] reveals a relatively small difference in the design velocity pressures. For the central regions, parts of the Crimea, Lviv, Odessa, Kherson and Luhansk wind load is less than in the SNiP [2]. In the Azov Sea, on the contrary, the wind load is much higher. On average in Ukraine, map *12.1* underestimates the wind load by 4%. At the same time, for 34% of observation points the wind load is reduced by 15… 25%, and for 12% of meteorological stations its increase by 25… 65% is required [5].

For the transition from the base recurrence period of T = 50 years to other values of T (in years), the dependence, generalized for the territory of Ukraine, was justified for the reliability factor for the design value of wind load:

$$\gamma_{fm} = 0{,}56 + 0{,}12\, ln\, T. \tag{12.8}$$

In the text of the DBN, this relationship is given in tabular form (*Table 12.3*).

In the codes of DBN [1] it is accepted that the operational design value of wind load W_e depends on the proportion of time η during which it may be exceeded. According to the data of 195 meteorological stations of Ukraine, the operational design values of wind load W_e were calculated, depending on the geographical area and the share of the service life of the structure η [5]. This made it possible to justify the corresponding coefficient

$$\gamma_{fe} = 0{,}358[-lg(\eta)]^{3/2}. \tag{12.9}$$

You can also use the corresponding DBN table (see *Table 12.5* above), built on the formula (12.9).

12.5. European wind codes Eurocode

Eurocode codes are a set of rules and methods for designing building structures, which are adopted in most European countries. We will briefly dwell on the section Eurocode: Wind actions [6], which contains rules and methods for calculating the wind load on building structures. This section of the Eurocode provides a separate description of the mean and pulsating components of wind speed. The average wind speed at altitude *z* is determined by the formula:

$$v_m(z) = v_b \cdot c_r(z) \cdot c_o(z), \tag{12.10}$$

where $c_r(z)$ is roughness factor; $c_o(z)$ is orography factor; v_b is basic wind speed.

The basic wind speed is determined by the formula:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}, \tag{12.11}$$

where c_{dir} is direction factor; c_{season} is season factor); $v_{b,0}$ is fundamental value of the basic wind speed.

The values of the coefficients c_{dir} and c_{season} (the name speaks for itself) are recommended to be equal to one, unless there are other recommendations in the National Annexes for their definition.

The fundamental value of the base speed is the wind speed, determined with a 10-minute averaging period, regardless of the direction and season, at a height of 10 m in an open area with low vegetation. The value $v_{b,0}$ is regulated by the National Annexes of European countries, on the territory of which the system of Eurocode codes operates. An example of such rationing is illustrated in *Fig. 12.8*. In contrast to domestic design codes, which operate at wind pressure, the Eurocode normalizes wind speed. For the $v_{b,0}$ of base recurrence period, the selected period is 50 years. If it is necessary to take into account another recurrence period, the base wind speed found by expression (12.11) should be multiplied by the probability factor c_{prob}:

$$c_{prob} = \left(\frac{1-K\,ln[-ln(1-p)]}{1-K\,ln(-ln\,0.98)}\right)^{n}, \tag{12.12}$$

where *K* is the shape parameter; *n* is the degree indicator.

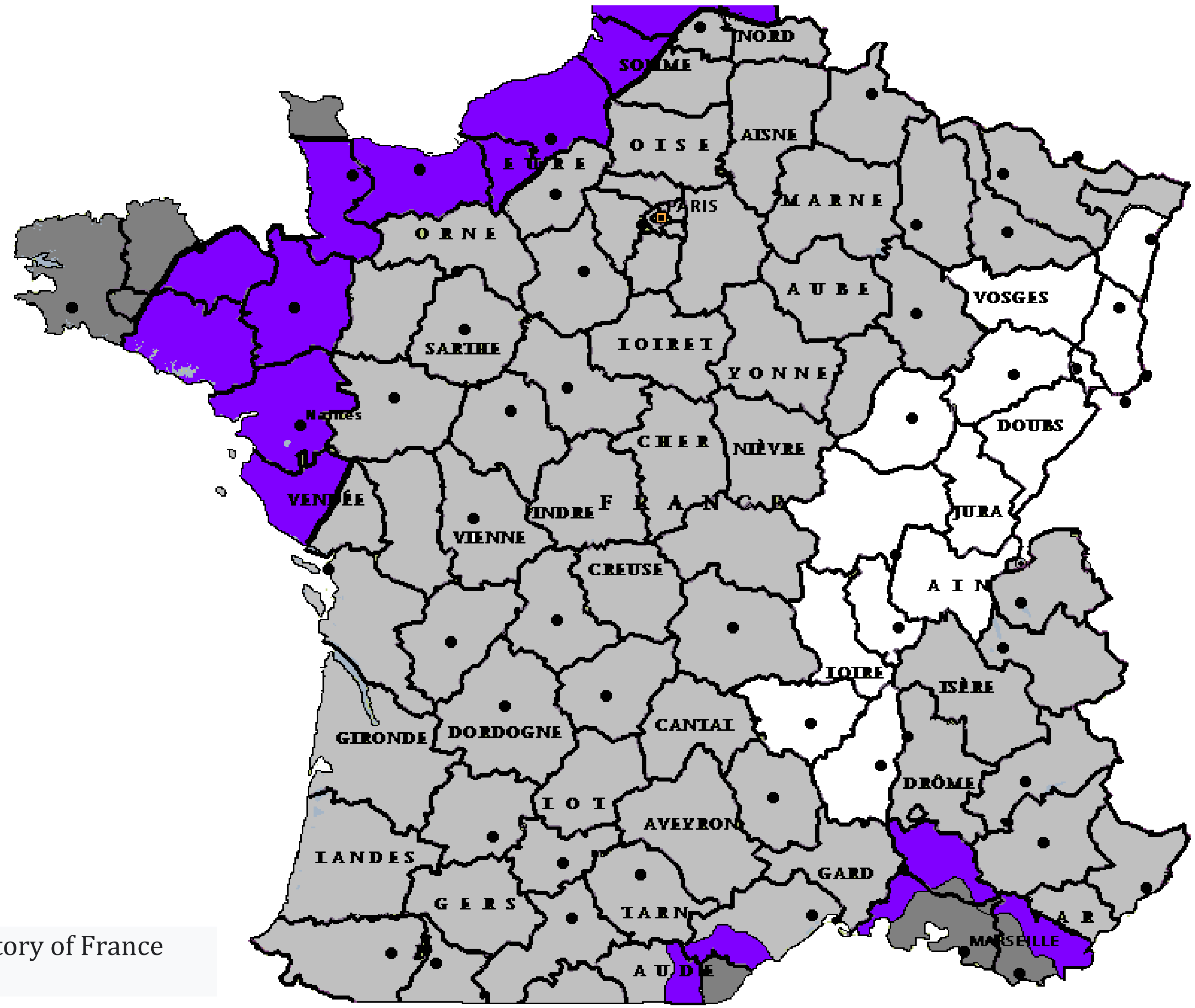
The values of *K* and *n* can be regulated in the National Annexes (for example, for the Netherlands, where each wind area has its own values of *K* and *n*). These values should be taken equal to *K* = 0.2 and *n* = 0.5 in the absence of recommendations for their definition. The parameter *p* in formula (12.12) determines the probability of annual exceedance of the wind speed level v_b, ie, $p = 1/T$, where *T* is the recurrence period of wind speed.

For example, the *Table 12.6* shows comparative data on the value of *p* and the stock ratio γ_f used in some countries.

Table 12.6

Comparative characteristics of the value of *p* and the coefficient γ_f, used in world practice

№ n/a	*Country*	*p*	γ_f
1	*Australia*	*0,05 for 50 years*	*1,0*
2	*Canada*	*0,033 during the year*	*1,3*
3	*Germany*	*0,1 during the year*	*1,5*
4	*Great Britain*	*0,02 during the year*	*1,4*
5	*Japan*	*0,01 during the year*	*1,0*
6	*Netherlands*	*0,08 during the year*	*1,5*
7	*New Zealand*	*0.05 over 50 years*	*0,93*
8	*South Africa*	*0,02 during the year*	*1,3*
9	*Switzerland*	*0,02 during the year*	*1,5*
10	*USA*	*0,02 during the year*	*1,3*
11	*Eurocode*	*0,02 during the year*	*1,5*



– 1 region

– 2 region

– 3 region

– 4 region

Region		Wind speed
1	–	24 m/s
2	–	26 m/s
3	–	28 m/s
4	–	30.5 m/s

Fig. 12.8. Zoning of the territory of France by wind speed

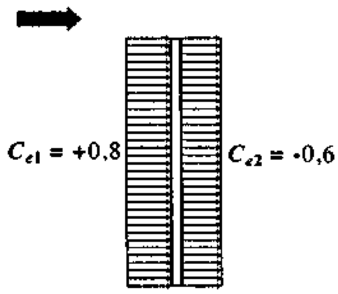
12.6. Aerodynamic coefficients Caer (extract from appendix I [1])

The aerodynamic coefficients C_{aer} are given in Annex I, where the arrows indicate the wind direction. The "plus" sign next to the coefficient corresponds to the direction of wind pressure on the surface, the "minus" sign the direction of wind pressure from the surface. Intermediate values of the coefficients should be determined by linear interpolation.

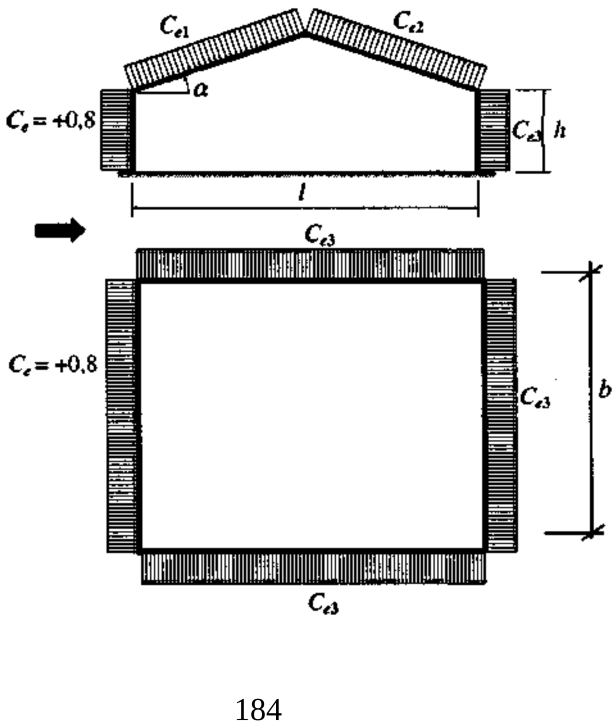
In cases not provided for in Annex I (other forms of construction, taking into account with due justification other directions of wind flow or components of total body resistance in other directions, etc.), aerodynamic coefficients may be taken from reference and experimental data or based on the results of blowing models of structures in wind tunnels.

Scheme 1. Flat solid structures are located separately

Vertical surfaces and those that deviate from the vertical by no more than 15 °



Scheme 2. Buildings with gabled roofs

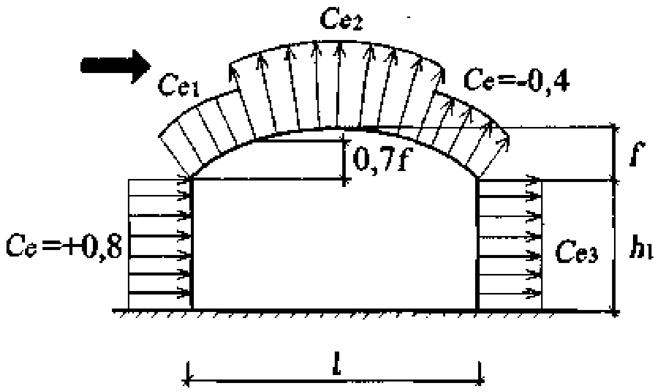


Coefficient	α***, deg***	***The values of C_{e1}, C_{e2} at h_1/l, equal to:***			
		0	***0,5***	***1***	***≥2***
c_{e1}	*0*	*0*	*-0,6*	*-0,7*	*-0,8*
	20	*+0,2*	*-0,4*	*-0,7*	*-0,8*
	40	*+0,4*	*+0,3*	*-0,2*	*-0,4*
	60	*+0,8*	*+0,8*	*+0,8*	*+0,8*
c_{e2}	*≤60*	*-0,4*	*-0,4*	*-0,5*	*-0,8*

b/l	***The value of C_{e3} at h_1/l, equal to:***		
	≤0,5	***1***	***≥2***
≤1	*-0,4*	*-0,5*	*-0,6*
≥2	*-0,5*	*-0,6*	*-0,6*

***Note**. When the wind is perpendicular to the end of the building, for the entire roof C_e = - 0.7.*

Scheme 3. Buildings with vaulted and close to them in outline coatings

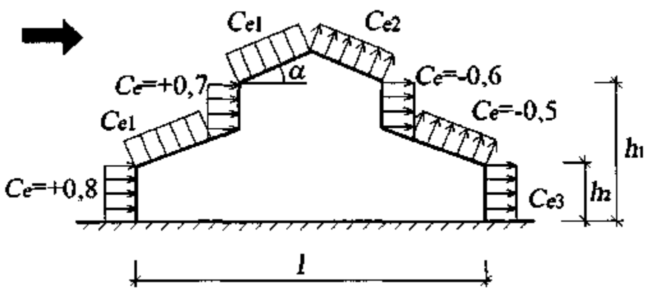


Coefficient	***h_1/l***	***The values of C_{e1}, C_{e2} at f/l, equal to:***				
		0,1	***0,2***	***0,3***	***0,4***	***0,5***
C_{e1}	*0*	*+0,1*	*+0,2*	*+0,4*	*+0,6*	*+0,7*
	0,2	*-0,2*	*-0,1*	*+0,2*	*+0,5*	*+0,7*
	≥1	*-0,8*	*-0,7*	*-0,3*	*+0,3*	*+0,7*
C_{e2}	*Arbitrary*	*-0,8*	*-0,9*	*-1*	*-1,1*	*-1,2*

The value of C_{e3} is taken according to scheme 2.

***Note**. When the wind is perpendicular to the end of the building, for the entire roof C_e = - 0.7.*

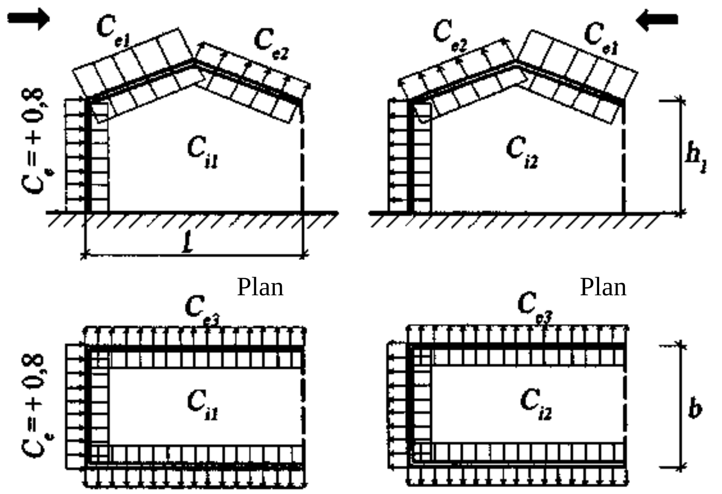
Scheme 4. Buildings with a longitudinal lantern



The coefficients C_{e1}, C_{e2}, C_{e3} should be determined according to the instructions in scheme 2.

Note. *When calculating the transverse frames of buildings with lanterns and windshields, the value of the total coefficient of drag of the system "lantern-shields" is assumed to be equal to 1.4.*

Scheme 9. Buildings that are constantly open on one side



At $\mu \leq 5\%$ $C_{i1} = C_{i2} = \pm 0{,}2$; when $\mu \geq 30\%$ C_{i1} should be taken as equal to C_{i3}, determined according to the instructions in scheme 2; C_{i2}=+0,8.

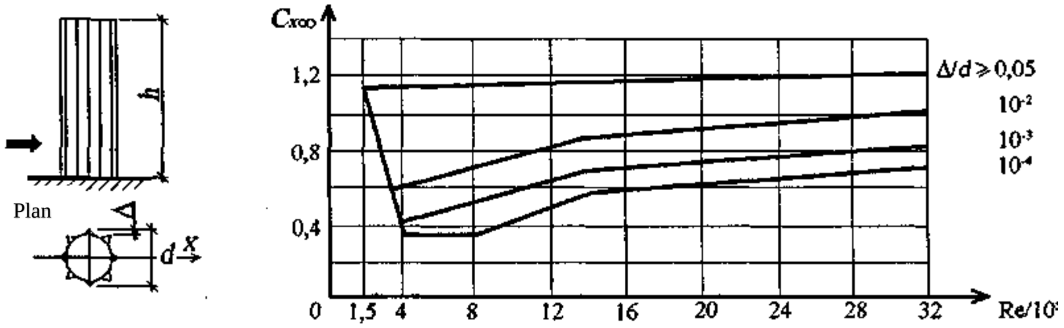
Notes.

1. Coefficients C_e on the outer surface should be taken according to the instructions in Scheme 2.

2. The permeability of the fence μ should be defined as the ratio of the total area of the openings to the total area of the fence. For a sealed building, C_i=0 should be taken. In the buildings specified in 9.3c, the characteristic value of the internal pressure on light partitions (at their surface density less than 100 kg / m^2) should be taken 0.2w_0 but not less than 0.1 kPa (10 kgf /m^2).

3. For each wall of the building, the sign "plus" or "minus" for the coefficient C_{i1} at $\mu \leq 5\%$ should be determined based on the condition of the implementation of the most unfavorable load option.

Scheme 14. Structures and their elements with a circular cylindrical surface (tanks, cooling towers, towers, chimneys), wires and cables, as well as round tubular and solid elements of through structures



$$C_x = kC_{x\infty}$$

$C_{x\infty}$ is determined by the schedule;

k is determined by *Table 1* of scheme 13.

λ_e	*5*	*10*	*20*	*35*	*50*	*100*	∞
к	*0,6*	*0,65*	*0,75*	*0,85*	*0,90*	*0,95*	*1,00*

Here λ_e = *l/b*, where *l* and *b* are respectively the maximum and minimum dimensions of the structure or its element in the plane perpendicular to the wind direction.

Notes.

1. Reynolds number $\mathrm{Re} = 0{,}88d\sqrt{W_0 k(z)\gamma_f} \cdot 10^5$, where z = h, d is the diameter of the structure. Values of Δ are accepted: for wooden structures Δ = 0.005 m; for brickwork Δ = 0.01 m; for concrete and reinforced concrete structures Δ = 0.005 m; for steel structures Δ = 0.001 m; for wires and cables with a diameter of d Δ = 0.01d; for ribbed surfaces with edges of height b Δ = b.

2. For corrugated coatings C_f = 0,04.

3. For wires and cables (including ice-covered) C_f =1,2. For wires and cables d $\geq$ 20 mm free of ice, the value of C_x may be reduced by 10%.

References

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5. Pashinsky V.A. Atmospheric loads on building structures. – K.: «Stal», 1999. – 185 p.

6. Eurocode 1: Actions on Structures. – Part 1-4: General Actions – Wind Actions. – Brussels: CEN TC 250, 2002. – 155 p.

Control questions

1. How is the wind load normalized according to the DBN?
2. What are the design values of wind load?
3. What coefficients are entered in the calculation of wind load?
4. Reliability coefficients according to the values of wind load
5. Probabilistic substantiation of wind load codes.
6. Features of European wind codes Eurocode.
7. Give examples of aerodynamic coefficients.

LECTURE 13. NATURE AND FEATURES OF CRANE LOADS

13.1. Overhead travelling cranes for general purposes
13.2. Overhead travelling cranes for special purposes
13.3. Metallurgical overhead travelling cranes
13.4. Underslung cranes
13.5. Cranes operation modes
13.6. Experimental research methods for crane loads

Hoist and transport machines are an indispensable element of any sphere of the economy. They significantly increase the productivity and the quality of work and save human resources. Technological process of most manufacturing enterprises is connected with the need of vertical and horizontal transportation of materials with a large range of weight. This mechanization together with other transport equipment is carried out by using overhead travelling cranes (bridge cranes) (*Fig. 13.2)* and underslung cranes (*Fig. 13.7*), which are special devices that move with the loads along and across the workroom.

Depending on the purpose, the overhead travelling cranes, which are installed in industrial buildings, are divided into the following main groups: cranes of general purpose, special cranes and metallurgical cranes.

13.1. Overhead travelling cranes for general purposes

The overhead travelling electric cranes are used for lifting and moving parts, assemblies and products, and also for installation operations in production shops of machine-building and metallurgical plants, on trestle bridges, in machine halls of power plants, etc. The main hoisting bodies of general purpose cranes are the main and secondary hooks, which can be moved vertically, along and across the workshops. Therefore, these cranes are also called hook cranes.

Classification of hook cranes.

- *Manual single-girder and double-girder hook cranes*, designed for lifting and moving loads during periodic work in warehouses, assembly and repair workshops, engine halls of power plants.
- *Electric single-girder hook cranes* (four-wheeled cranes), designed for lifting and moving loads in the workshops and warehouses.
- *Electric double-girder cranes* (four-wheeled cranes) that perform similar functions and are also used in open areas.
- *Electric hook cranes of large capacity* (multi-wheeled cranes), designed for moving loads of great weight in the mechanical and warehousing workshops. In addition, they can be used for installation and repair works.

Single-girder cranes. Such overhead travelling cranes (runway beam for hoist block) have become widespread for mechanization of loading and unloading operations, as well as transport operations in warehouses, installation and container sites and in industrial shops. Taking up a minimum of space, single-girder cranes are able to serve almost the entire area of the workshop. Single-girder cranes can be made with solid span structures (span beams) in the form of a pipe, I-beam, perforated structure (*Fig. 13.1*).

Fig. 13.1 Single-girder crane (with perforated beam)

For single-girder cranes with electric drive rope the electric hoists (electric hoists block) are used as crane crab; these cranes have loading capacity of 2.0 ... 12.5 tf, the span of 10.5 ... 28.5 m.

Electric hook double-girder cranes (*Fig. 13.2*). Let's look in more detail at its design on the example of a four-wheeled crane of general purpose with a load-carrying capacity of 50 tf (*Fig. 13.3*). The crane bridge consists of two span (head) beams (pos. 5) of box cross-section. Beams are made of sheet steel 5… 6 mm thick, depending on the load capacity and strengthened with corrugations. Windows are provided in the outer vertical sheet of the beam for facilitation. Perpendicular to the span beams there are the end (transverse) beams (pos. 8) also of box cross-section.

Two travel wheels (pos. 9) are attached to the end girders on the axles, one of which is driven, there are railings on top of the girders, and spring buffers (pos.10) are attached to the ends of the girders, by which the bridge thrusts against on the deadlocks (pos. 11). To one of the span beams on the suspensions (pos. 7) are attached two platforms (pos. 15) with railings. Each platform is equipped with an individual drive (pos. 14), which is consists of an electric motor and a reducer. The reducer is connected to the drive trolley by a floating shaft (pos. 13), which is connected by means of toothed couplings. There is a crossing bridge (pos. 19) between the platforms, which is also protected by railings. There are similar platforms (pos. 23) on the other span beam, which are not connected with each other by a bridge.

Fig. 13.2. Overhead travelling double-girder crane

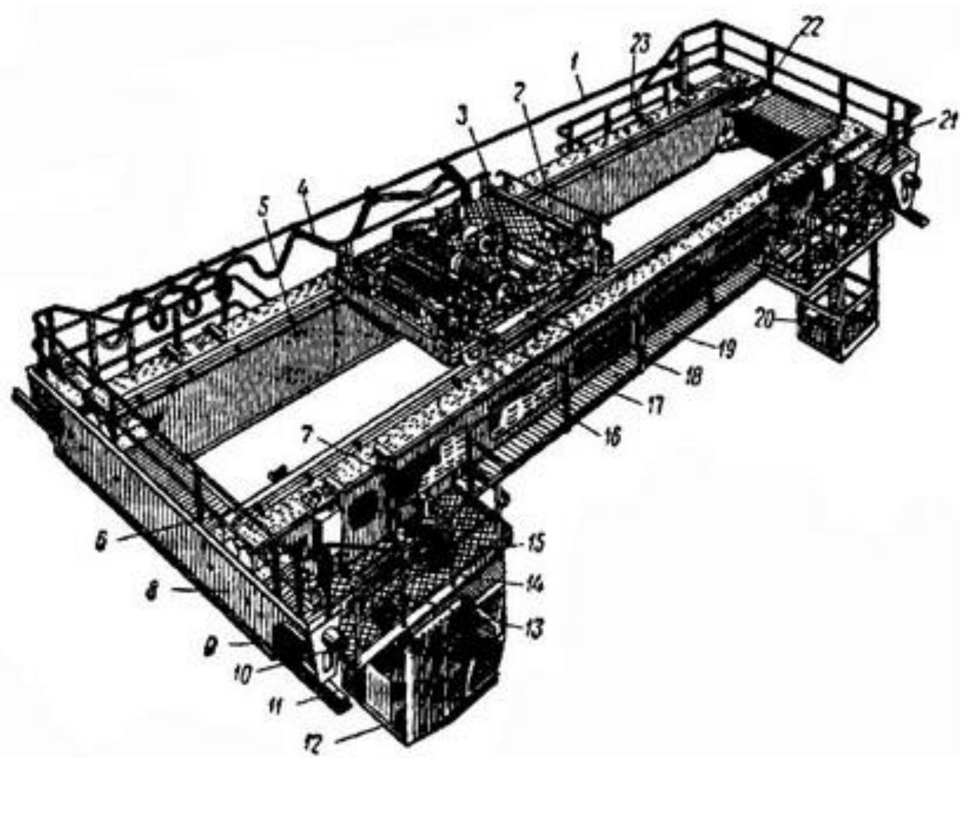

Fig. 13.3. Bridge double-girder electric crane of general purpose with a capacity of 50 tf

Rails (pos. 6) are installed on the span beams. Rails are used for crab moving. Crab is formed by the frame (pos. 18), to which four running wheels (pos. 16), two of which are driven, are attached on the axleboxes. There are the mechanisms of movement and main hoist (pos. 6) and additional hoist (pos. 17) on the frame of the crab. Polyspasts with lifting blocks (not shown in the figure) are suspended from the drums of the mechanisms. The power supply of the crane crab is provided by the flexible cable (pos. 4), which is suspended on kapron rollers to the wire (pos. 1), which is stretched along the span beam. The crane driver's cabin (pos. 12) and the trolley maintenance cabin (pos. 20) are suspended to the span beam on the side of the drive wheels. The entrance to the crane cabin through the hatch is blocked in the crane electrical scheme in such a way that when the hatch is open the crane mechanics cannot be switched on. The electrical installations and wiring are located in the void of the span beams, which excludes the use of pipes for wires. All crane mechanisms are mounted on rotating bearings.

Load capacity of two-beam four-wheeled cranes: 5; 10; 12.5; 16; 16 / 3.2; 20/5; 32/5 and 50 / 12.5 ts. Span: 10.5… 34.5 m (with a step 3 m). Lifting height up to 20 m.

Electric hook bridge cranes of large capacity (*Fig. 13.4, a*). The main hoists of such cranes are the main and additional hooks that can move vertically, along and across the workshop. Crane bridges consist of two main (span) beams and two end beams. The end beams have more complicated design, they have balancers for more uniform load transfer to crane girders. Considering the large weight of the crane and the cargo, as well as in order to reduce the load on the wheels, cranes with capacity of 80, 100, 125 t of all spans and cranes with capacity of 160 tf with the span up to 16 m are supported by balancing carts on eight wheels that are connected in pairs by balancers. Cranes with a capacity of 160 tf with a span longer than 16 m and cranes with a capacity of 200, 250 and 320 tf of all spans are supported on 16 wheels. As shown in *Fig. 13.4, b,* each main beam (pos. 5) of these cranes is rested on the balancer (pos. 3) from both sides, which is connected to the span beam by a roller (pos. 2). Balancers are also connected by rollers (pos. 4) to balance carts, each of them has two travel wheels (pos. 6). The outside wheels are driven on each side. Thus, the crane has four drive wheels. If cranes have eight wheels, the span beams rest directly on the balancing carts.

Load capacity of cranes: 80/20… 320/32, 450/100 ts. Span: 9.5… 33.5 m. Lifting height up to32 m.

13.2. Overhead travelling cranes for special purposes

Magnetic cranes. These cranes are designed for lifting and moving of ferrous metal products with magnetic properties (scrap, shavings, sheet and profile rolled products, steel casting molds, scrap metal, etc.).

a)

b)

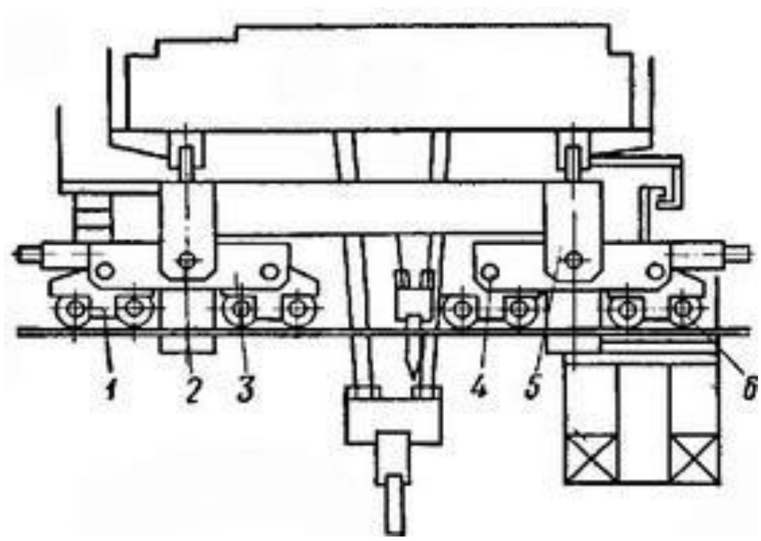

Fig. 13.4. Bridge electric crane of large capacity:
a - general view; b - end view of the crane

Magnetic cranes are equipped with electromagnets that are suspended on a hook suspension or on traverses (on a flexible or rigid suspension) which are located in a longitudinal or transverse direction to the bridge (*Fig. 13.5, a*). Traverses can be different lengths, and the number and location of magnets on the traverse can vary. When the electromagnet is removed, the magnet cranes can operate with unit loads in hook mode.

a)

б

Fig. 13.5. Special bridge cranes:
a - magnetic crane; b - grab crane

Technical characteristics of magnet cranes: load capacity - 5 ... 40 tf, lifting speed - 14 ... 20 m/min, crab moving speed - 70 ... 120 m/min.

Grab cranes. Grab cranes are designed for reloading, lifting and moving of bulk and lump materials with the help of rope or hinged electromechanical or electro- hydraulic grabs (*Fig. 13.5, b*).

The load capacity of the grab cranes is determined by the total weight of the grab and the cargo 5; 10; 15 and 20 tf (accordingly the capacity of the grabs: 1.6; 2.5; 3.1 ... 8.0; 4.0 ... 10.0 cubic meters). Spans: 10...34.5 m, lifting height up to 24 m.

Magnetic grab cranes are designed for the reloading of ferromagnetic cargoes (for example, cast iron, scrap, etc.), and also for bulk and lump cargoes. Electromagnet is used to lift ferromagnetic cargoes, and double-jawed grab is used for bulk materials. Depending on the application, these cranes are manufactured with two crabs: magnetic and grab or with one crab, which is equipped with a magnet and a grab hoist.

13.3. Metallurgical overhead travelling cranes

This is a special type of cranes, which are used in the technological process of metallurgical and machine-building plants for carrying out lifting and transporting and different technological operations. In contrast to the usual overhead travelling crane, which is serviced by a crane operator and slinger, metallurgical crane is often controlled only by the crane operator. The absence of a slinger requires full mechanization of the grabbing organs of the metallurgical crane. In order to achieve the loading of cargo, the grabbing bodies of most metallurgical cranes have **a rigid suspension**, due to which it is ensured the mechanization of the crane transport operations from the crane operator's cabin.

According to GOST 25546-82* [5], most of the metallurgical cranes are classified to the operating mode groups 7K (heavy) and 8K (very heavy). Bridges of metallurgical cranes are beam and lattice structures. Depending on their own weight and the weight of the cargo, which is lifted up, the bridges can be rested by four, eight, twelve or sixteen running wheels.

Open-hearth furnace charging cranes are designed for loading solid charge into steelmaking furnaces, for carrying out additional lifting and transporting operations during repair and maintenance of furnaces, and also for cleaning the working area of the workshop (*Fig. 13.6, a*). Operating gripper is made in the form of a trunk, which is progressively moved up and down together with the column and jiggles in a vertical plane. The crane uses this trunk to captures the troughs with the charge and transport them to the steelmaking furnace.

Foundry cranes are the main lifting and transporting equipment that are used at metallurgical enterprises for transportation, pouring and pouring of liquid metal (*Fig. 13.6, b*). On the main beams of such cranes, the main crab is moved, and on the additional beams the additional crab is moved. The main crab by means of hoist transports a ladle with molten metal, additional crab can move under the main crab and turn over of ladle for pouring liquid iron into the furnace or release the steel ladle from slag. The main load-grabbing device of foundry cranes is designed as a traverse with widely spaced lamellar single-horned hooks that are suspended to the axles.

Stripper cranes are installed in special (stripper) departments of steelmaking workshops and perform technological operations that are related to the removal of steel castings from casting molds (*Fig. 13.6, c*). The crab has a rigid suspension of the cargo, large and small pliers which are fixed in a special chuck. This chuck moves along the special guidings, that are fixed in the middle of a circular shaft. The shaft is rigidly connected to the frame of the crab.

Soaking pit cranes are used in the sections of heating wells of blooming or slabbing furnaces. They move the casting to the bottoms of the furnaces (*Fig. 13.6, d*). Castings are heated to a temperature of 1100 ... 1200 °C, and then they are transported to the casting conveyor by soaking pit cranes, which transports them to the receiving roller conveyor of blooming or slabbing. The crane is equipped with special clamping device that can rotate around its axis. During operation of the mechanism, the clamping device together with the column move along the guides that are installed in the mine.

Forge cranes are used in press-forging workshops for forging with presses, including feed of workpieces on the table and removal from the table, support and rotation of the workpiece during forging, regulation of workpiece positioning on the press table (*Fig. 13.6, e*). On the hook of the main crab a forging tilter with a hinged chain is suspended that supports the chuck in which the pin of the ingot is fixed. Forging is carried out by the upper striker of the press.

Cranes of rolling mill workshops have a special place in the group of metallurgical workshops, they are included in the non-stop technological process and are involved in the transportation and storage of rolled steel or steel billets. Claw cranes (cranes with pickups or claws) have hoisting devices in the form of traverses with claws, as well as with hoisting magnets or hooks. Claws are used to insure the cargo when the magnets work and for transportation of hot rolled products. Bridge cranes with main hook rotation mechanism are designed for transportation of coils in cold rolling workshops (*Fig. 13.6, f*). In accordance with technological process of these cranes a magnet can be suspended on the hook, a bracket or claws for transportation of coils or a special traverse with sliding claws.

13.4. Underslung cranes

The most popular in enterprises of various sectors of industry have received a single-girder underslung cranes, which are characterized by simplicity, reliability in operation and the possibility of maintenance of almost all the space, which is covered by the rafter structures. Underslung cranes with a load capacity of 0.25 ... 5 tf are mainly used in industrial buildings. There are one-, two- and three-span underslung cranes.

Single-girder underslung crane consists of a bridge, it is a girder that is attached from the lower side to the end girders and strengthened with struts to increase stiffness (*Fig. 13.7*) The mechanism of movement are driven and non-driven electric crabs. An electric hoist is used as the lifting mechanism (*Fig. 13.8*). The crane is controlled from the floor with a special remote control.

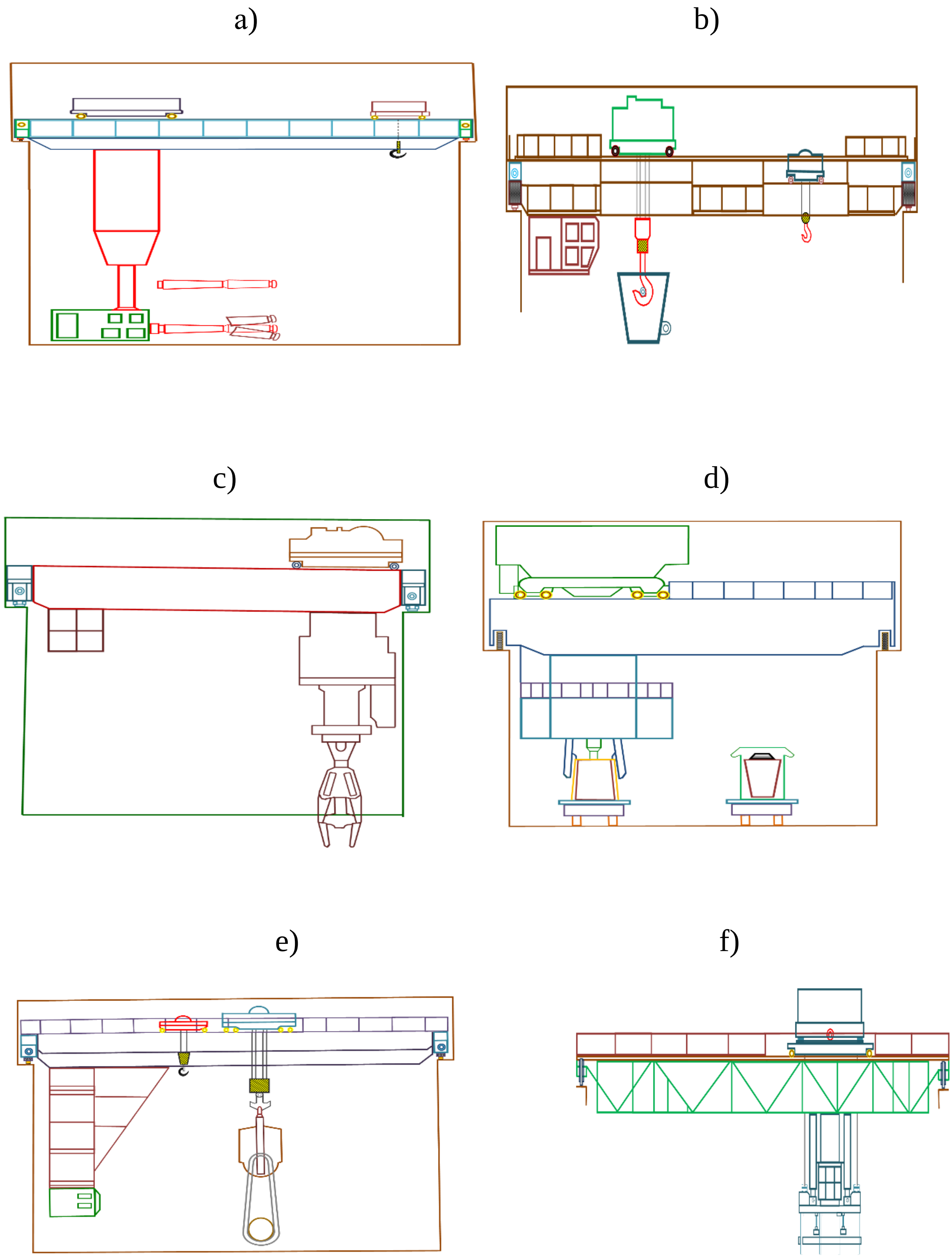

Fig. 13.6 The metallurgical cranes
a – open-hearth furnace charging crane; b –foundry crane; c – stripper crane;
d – soaking pit crane; e – forge crane; f – crane of rolling mill workshop

Fig. 13.7 Underslung crane (runway beam for hoist block)

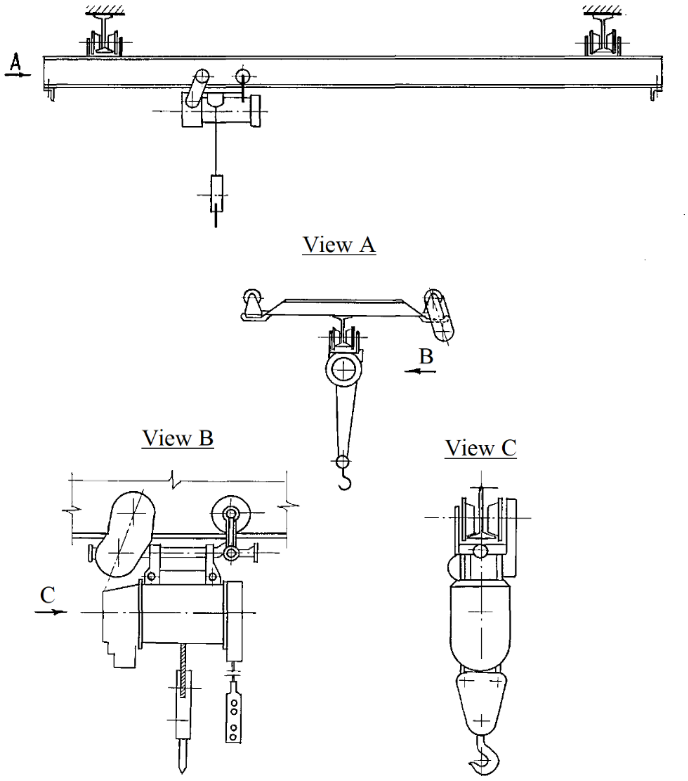

Fig. 13.8. Scheme of the single-girder underslung crane
(view A, B, C – hoist block)

Runway beams of underslung cranes are often made of rolled I-beams of a special profile. There are also runway beam of I-beams with a perforated wall, and since recently the beams of the folded section with the lower flange from a special wear-resistant brand have become widespread. The span of crane runway beams at enterprises of different industries is usually set equal 6 m, in some cases is 4 m. Special beams of folded asymmetric cross-section with 12 m span have been developed and implemented recently.

Location of runway beams in relation to the rafter structures is carried out both parallel and perpendicular to them, which provides a more flexible planning in comparison with the planning of buildings, which are equipped with bridge cranes. On some runway beams can be located a few cranes, the number of them often reaches five or more.

13.5. Operation modes

The main parameters of bridge cranes, which affect to their created loads are load capacity, span, type of drives, method of cargo suspension and group of their operation modes, which is set by GOST 25546-82 *[5].

Crane operating mode group is determined depending on the class of using and the load class in accordance with *Table 13.1*. The table also includes the appropriate crane operating modes according to the international code ISO 4301/1-86 [6]. Class of crane using is determined according to the number of crane operating cycles during its service life (*Table 13.2*).

Table 13.1

Group of crane operating mode

Class of using	***Group of crane operation mode for the load class***									
	Q0		***Q1***		***Q2***		***Q3***		***Q4***	
	G	***I***	***G***	***I***	***G***	***I***	***G***	***I***	***G***	***I***
C0	-	-	-	-	*1K*	-	*1K*	*A1*	*2K*	*A2*
C1	-	-	*1K*	-	*1K*	*A1*	*2K*	*A2*	*3K*	*A3*
C2	*1K*	-	*1K*	*A1*	*2K*	*A2*	*3K*	*A3*	*4K*	*A4*
C3	*1K*	-	*2K*	*A2*	*3K*	*A3*	*4K*	*A4*	*5K*	*A5*
C4	*2K*	-	*3K*	*A3*	*4K*	*A4*	*5K*	*A5*	*6K*	*A6*
C5	*3K*	-	*4K*	*A4*	*5K*	*A5*	*6K*	*A6*	*7K*	*A7*
C6	*4K*	-	*5K*	*A5*	*6K*	*A6*	*7K*	*A7*	*8K*	*A8*
C7	*5K*	-	*6K*	*A6*	*7K*	*A7*	*8K*	*A8*	*8K*	-
C8	*6K*	-	*7K*	*A7*	*8K*	*A8*	*8K*	-	-	-
C9	*7K*	-	*8K*	*A8*	*8K*	-	-	-	-	-

Table 13.2

Class of crane using

Class of using	***Total number of crane operating cycles during its service life***
C0	*Up to* $1{,}6\cdot10^4$
C1	*Over* $1{,}6\cdot10^4$ *to* $3{,}2\cdot10^4$
C2	*Over* $3{,}2\cdot10^4$ *to* $6{,}3\cdot10^4$
C3	*Over* $6{,}3\cdot10^4$ *to* $1{,}25\cdot10^5$
C4	*Over* $1{,}25\cdot10^5$ *to* $2{,}5\cdot10^5$
C5	*Over* $2{,}5\cdot10^5$ *to* $5{,}0\cdot10^5$
C6	*Over* $5{,}0\cdot10^5$ *to* $1{,}0\cdot10^6$
C7	*Over* $1{,}0\cdot10^6$ *to* $2{,}0\cdot10^6$
C8	*Over* $2{,}0\cdot10^6$ *to* $4{,}0\cdot10^6$
C9	*Over* $4{,}0\cdot10^6$
Note. *Cycle of crane operation consists of moving the lifting device to the cargo, lifting and moving the cargo, releasing the lifting device and returning it to the original position.*	

The working life of the cranes is set in the codes or technical conditions for the cranes of particular types. The load class of the crane is determined according to the load factor (*Table 13.3*).

Table 13.3

Load class	***Load factor*** K_P
Q0	*Up to 0,063*
Q1	*Over 0,063 to 0,125*
Q2	*Over 0,125 to 0,25*
Q	*Over 0,25 to 0,50*
Q4	*Over 0,50 to 1,00*

Load factor K_p should be calculated by the formula

$$K_P = \sum \left(\frac{Q_i}{Q_{ном}} \right)^3 \frac{C_i}{C_T}, \tag{13.1}$$

where Q_i is weight of the cargo that is moved by the crane;

C_i is number of crane operation cycles with the weight of the cargo Q_i;

Q_{nor} is nominal capacity of the crane;

C_T is number of crane operation cycles during its service lifetime,

$C_T = \sum C_i$.

For lack of data about the operating mode group in the crane passport, as well as for the lack of output data which are necessary for determining the load class and factor of using, the mode group can be determined according to the recommended list of cranes, which is given in the appendix to the code. Here is the classification of cranes (*Table 13.4*) by groups of crane operation mode which is contained in the current design codes [1].

Previously, the USSR used a different classification of modes of operation of cranes, which was regulated by the rules of Gosgortekhnadzor [5], it is illustrated in *Table 13.5*

The factors that are included in the table were defined in the following way:

- factor of yearly use: the number of working days of the crane per year divided by 360;
- factor of daily use : the number of working hours per day divided by 24;
- factor of use by load-carrying capacity: the average value of the amount of lifting cargo per shift divided by the crane capacity;
- relative duration of crane motor starts: operating time of the mechanism during the cycle divided by the full cycle time.

Table. 13.4

Overhead travelling cranes and underslung cranes (approximate list, appendix G [1])

Cranes	***Terms of use***	***Groups of crane operation mode***
Manual of all kinds	*Any*	*1K-3K*
With suspended hoists, including hinged grippers	*Repair and reloading works of limited intensity*	
With winch cargo hoists, including hinged grippers	*Engine halls of power plants, installation works, reloading works of limited intensity*	
With winch cargo hoists, including hinged grippers	*Reloading works of average intensity, technological work in mechanical workshops, warehouses of finished products of construction material enterprises, warehouses of metal products*	*4K-6K*
With grabs of two-rope type, magnetic-grapple	*Mixed warehouses, work with a variety of cargoes*	
Magnetic	*Warehouses of semi-finished products, work with various cargoes*	
Hardening, forge, soaking pit, foundry	*Workshops of metallurgical enterprises*	*7K*
With grabs of two-rope type, magnetic-grapple	*Warehouses for bulk cargo and scrap metal with uniform cargo (one or two shifts work)*	
With winch cargo hoists, including hinged grippers	*Technological cranes at round-the-clock work*	
Traverse, open-hearth furnace charging, stripper, soaking pit	*Workshops of metallurgical enterprises*	*8K*
Magnetic	*Workshops and warehouses of metallurgical enterprises, large metal bases with uniform cargo*	
With grabs of two-rope type, magnetic-grapple	*Warehouses for bulk cargo and scrap metal with uniform cargo (round-the-clock work)*	

Table 13.5

Overhead travelling cranes operation modes

Nominal crane operation modes	***Factor of yearly use K_G***	***Factor of daily use K_C***	***Factor of use by load-carrying capacity K_{LCC}***	***Relative duration of inclusions PV %***	***Quantity of inclusions per hour n_{VK}***
Light (L)	*0,25…1,0*	*0,33..1,0*		*15…25*	*60*
Medium (M)	*0,5…1,0*	*0,33..1,0*	*0,75…1,0*	*15…60*	*120*
Heavy (H)	*0,75…1,0*	*0,33..1,0*	*1,0*	*25…60*	*240*
Very heavy (VT)	*1,0*	*1,0*	*1,0*	*40…60*	*300*
Very heavy uninterrupted (VTN)	*1,0*	*1,0*	*1,0*	*60…80*	*720*

13.6. Experimental research methods for crane loads

The study of crane loads includes, first of all, experimental research, which must be carried out mainly under conditions of normal operation of active workshops. Therefore, obtaining and accumulation of experimental data of the crane loads are the task of great complexity. A great role here is played by such specific factors as uninterrupted operation of overhead and suspension cranes, large dynamical actions from the movement of the crane bridges on the rails, magnetic fields on the cranes, high temperature actions and so on. As a consequence, the methodology and equipment for research of crane loads require more stringent requirements than the equipment for laboratory studies.

Because the industry does not produce the necessary devices, various researchers developed their own methods, designed and produced devices and appliances, that in some or other way satisfied the requirements of the full-scale experiment. However, it should be admitted that up to the present time there is no universal set of equipment that can provide the researcher with reliable statistical data about the force actions on the frames of industrial buildings.

The methodological part of the full-scale test to study the crane loads includes:

a) selection of measuring and registration equipment;

b) selection and design of weighing devices (dynamometers), determining the location of their installation;

c) determination of the required volume of statistical material and duration of the experiment.

Visual method of research. The vertical load of the crane on the structures (columns or crane beams) depends on the following parameters:

a) load from the weight of the crane bridge with movement mechanism and cabins, depends on the position of the crane on the crane runway beam;

b) the load from the crab with its movement mechanism, depends on the position of the crane and the location of the crab on the bridge;

c) load from the weight of the cargo on the hook, which is a function of the coordinates of the hook and the weight of the cargo.

By recording the crane's displacement and its load, and the weight of the cargo on the hook, the vertical load of the crane on the columns and crane runway beams can be determined. Considering this, the research of B.N. Koshutin [9] was carried out in operating workshops by visual method with registration of the position and all movements of the crane hook and with record of the cargo weight and places of its lifting and lowering. The registration was performed using a grid, as lines of which were used the axes of the workshop along the length, which connected the opposite columns, and across the span were used the trajectory of the trusses of the moving crane, or lines which connected the respective nodes of the rafters. The coordinates of the moving crane were recorded in the observation journal. For example, the entry 7e, 5/6aL.3, 3/4c/dP, 3/4a/b, 1/2a/b corresponds to the movement of the crane hook (*Fig. 13.9*); here it is written that from position 7e the hook moved to point 5/6a, where the crane lifted cargo (L), and at point 3/4c/d crane put down (P) cargo weight of 3 tf, after which the crab with an empty hook moved to point 3/4a/b, then hook moved to point 1/2a/b.

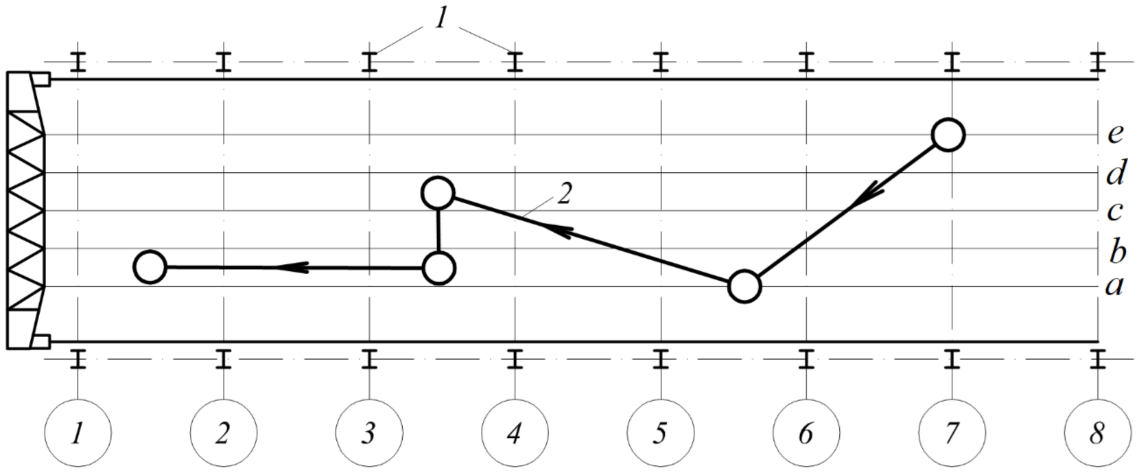

Fig. 13.9. An example of the movement of the crane hook, that was reproduced in the logbook: 1 - columns; 2 - trajectory of the crane hook

Cargo weights were determined approximately, for example, by the number of castings, by specifications, etc. For further processing, the crane hook movements simulated on the span plan with coordinate lines applied to it (*Fig. 13.9*), after which the loads on the crane runway beams and columns were calculated.

Observations according to this methodology were carried out in 21 workshops of metallurgical plants and plants of heavy machine-building. For all cranes there were statistical distributions (polygons) of cargo weight that was lifted, forces in crane bridge elements and vertical crane loads on the columns.

It should be recognized that the method of visual observations, although it gives a lot of information about the vertical actions of cranes, but it has a number of disadvantages: the subjective features of the observer, the proximity of determining the weight of crane and cargo, etc. Therefore, the results obtained by visual methods, were compared with instrumental measurements made by means of mechanical strain gauges and electric tensometers which were installed on lower flanges of crane girders. The comparison showed quite close results of the measurements which were obtained by both methods.

Use of crane runway beams as a dynamometer. In a number of works the lower flanges of crane runway beams were used as a dynamometer of the vertical load of the crane. This is due to the fact that unlike other elements of the beams, for example, the upper flanges or walls, stresses (strains) in the lower flanges of the crane runway beams in the most stressed (calculated) cross-sections are entirely determined only by the vertical loads of the cranes, regardless of any other factors. This method excludes the violation of the actual working scheme of the structures and conducting long and labor-intensive preparation work at the test sites, which makes it advantageously different from the method of using different removable dynamometers.

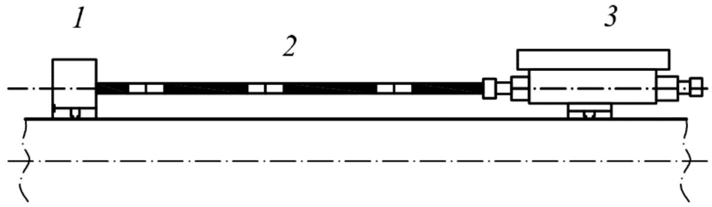

Fig. 13.10. Rabu's strain gauge based on clock type indicator:
1 - indicator; 2 - rod; 3 - fixed support

In his research O.O. Bat (TsNIISK, Moscow) used a Ruby mechanical strain gauge, the scheme of which is shown in *Fig. 13.10*. Full-scale research showed that such a strain gauge gave stable readings despite the severe temperature conditions. The strain gauge base was 50 cm, the accuracy of load determination was 20...40 kgf/cm^2 (0,2...0,4 MPa). Simplicity of measurements allowed the research to be carried out in 25 workshops of 4 plants, and continuous observations were performed for 2...6 days. In total more than 8 thousand cycles of loading of crane runway beams were recorded. This device was not widely used due to the need for a observer to be present at all times to capture data.

In the further research (Y.S. Kunin, MISI, Moscow) special strain gauges were glued on the lower flanges of the crane runway beams and their recording was carried out with a self-recording device of the H373 type with an F117/10 amplifier. The used data recorder had a wide range of speeds (from 20 to 5400 mm /hour), the record was black on a tape with a curved coordinate grid 100 mm wide. Continuous operation (up to 3 months) in the workshops of the iron and steel industry showed high reliability of the device and its ease of use. Two times in each span was carried out cargo calibration (before the start and after finishing tests) by crane runs with a known value of the cargo on the hook.

Also to measure vertical support reactions of crane runway beams there were used elements of their support parts (end support corners), on which inductive measuring devices of vertical loads were installed *(Fig. 13.11, 13.12)* (S.F. Pichugin, PoltNTU, Poltava [8]). These measuring devices were made as inductive strain gauges of a large base. The construction of the inductive strain gauge included an inductive sensor and a stand with a micrometer adjusting screw, which in turn were attached to the mortgage bars welded to the beam. The distance between the welded bars, which determines the base of the inductive strain gauges, was chosen by selection and equaled 600...700 mm.

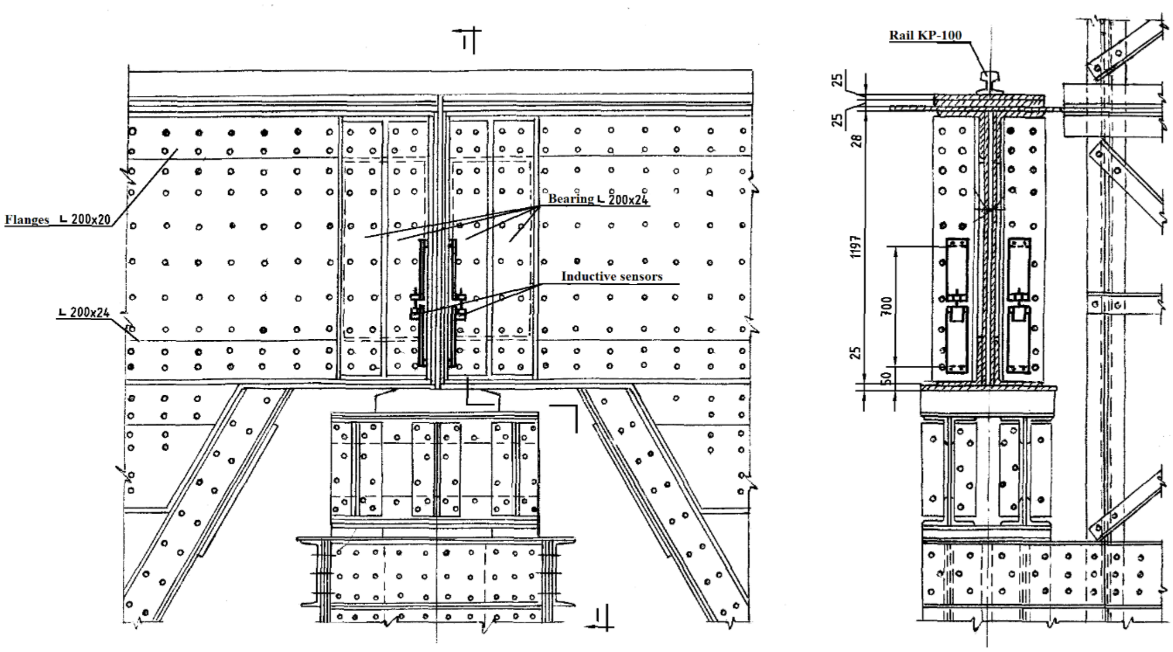

Fig. 13.11. Installation of inductive measuring devices of vertical loads

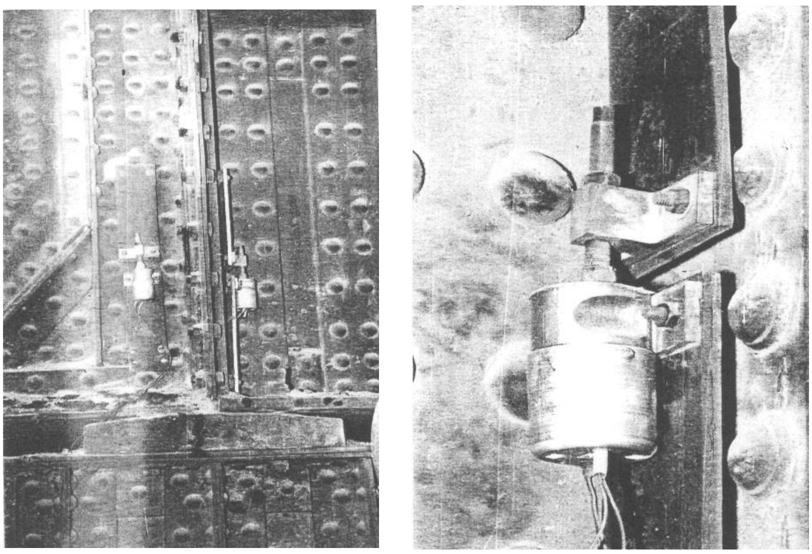

Fig. 13.12. Inductive strain gauges of vertical loading

For calibration of inductive strain gauges, special crane arrivals with different positions of the crab without cargo, with an empty and full bucket were performed.

The inductive displacement sensors were used in the tests, the principle of operation of which is as follows. When the measured displacement is transmitted to the sensor rod, the distance Δ between the disk and coils changes, which changes the inductive resistance of the coils. When each of the coils is switched to different arms of the measuring bridge, there is a current in its diagonal. Range of sensor rod movements 0,2...0,4 mm, operating voltage is variable current with voltage of 25 V of industrial frequency of 50 Hz. The signal from inductive sensors without amplification was transmitted to the loop magneto-electric oscillograph H-700, which has 12 channels, with recording on the photopaper.

Measurements on the overhead travelling crane. Measuring with the help of *strain gauge lids* allowed to determine the lateral forces directly on the crane wheels by measuring the transverse forces that act on the axis of the wheels. The design of the strain gauge is shown in *Fig. 13.13* [9].

Instead of a nut, a shaped nut with a rod (pos. 3) is screwed on the wheel axis from the outside (pos. 5), which rests on the end of the axle and which is connected to a force-measuring beam (U-section beam) (pos. 5) with strain gauges glued on it (pos. 8). The supports of this beam are bolts that attach it to the axle box (pos. 2).

This technique allows to obtain a reliable and accurate picture of the distribution and changes of horizontal actions on individual wheels, while their quantitative characteristics sometimes come out with noticeable errors. At the same time, the use of this technique is associated with great difficulties in the manufacture and installation on the overhead travelling crane, which limits the possibility of using strain gauges in full-scale conditions.

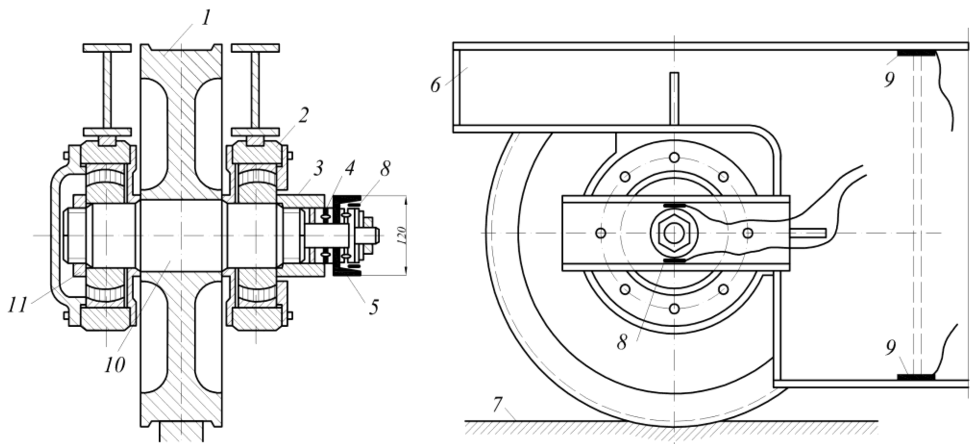

Fig. 13.13. Scheme of the strain gauge lid design:
1 - crane wheel; 2 - axle box; 3 - shaped nut with a rod; 4 - thrust bearings; 5 - dynamometer from the channel; 6 - the end beam of the crane; 7 - crane rail; 8 - strain gauges on the dynamometer; 9 - strain gauges on the flanges of the end beam; 10 - wheel axle; 11 - nut that secures the inner ring of the bearing

The the strain gauge method was used to ***measure the weight of the cargo***, the set of equipment included a sensor, a amplifier and a recording device. As a sensor, a strain gauge (load-carrying bracket) was used, the operating principle of which was analogous to the principle of measuring the force of tension of ropes and pulls. The bracket was installed directly on the rope, which was given a certain sag in the base of the bracket (*Fig. 13.14*). The transverse force, which occurs in this case, was transmitted to the beam with the strain gauges glued on it. The value of the cargo with an accuracy of 3...5% was determined by the calibration graph, which was made according to the results of the lifting of the control cargos. Signals from the strain gauge were amplified to the required value by a photocompensation amplifier of a direct current type F-117.

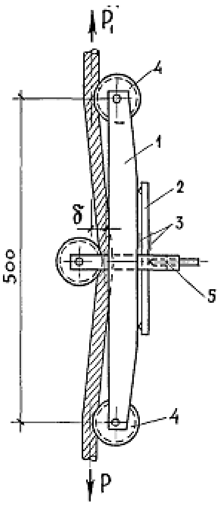

Fig. 13.14. Scheme of a load measuring bracket
1 - base beam; 2 - load measuring beam; 3 - strain gages;
4 - support rollers; 5 - tensioning device

Measurement of lateral forces on columns. For this purpose, instead of fixing the upper flange of the crane runway a special dynamometer was installed to the column, which accepted the horizontal load that is transmitted to the column from two crane beams, which are supported by the column. All available elements of fixing the beams to the columns: vertical diaphragms, brake sheets, flanges of brake trusses - were disconnected.

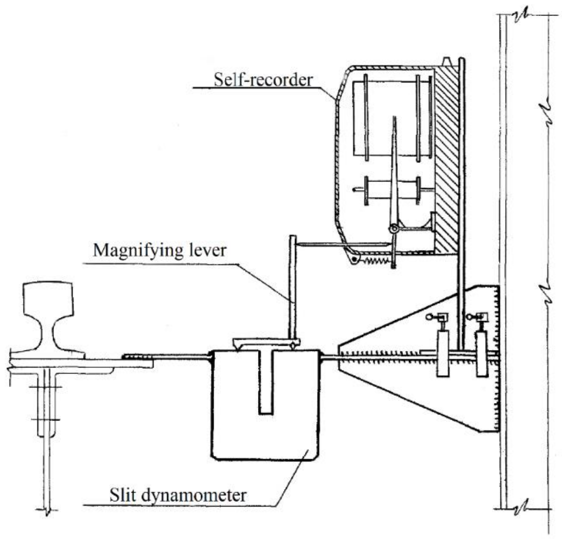

Figure 13.15. Installation scheme of the dynamometer and mechanical recorder

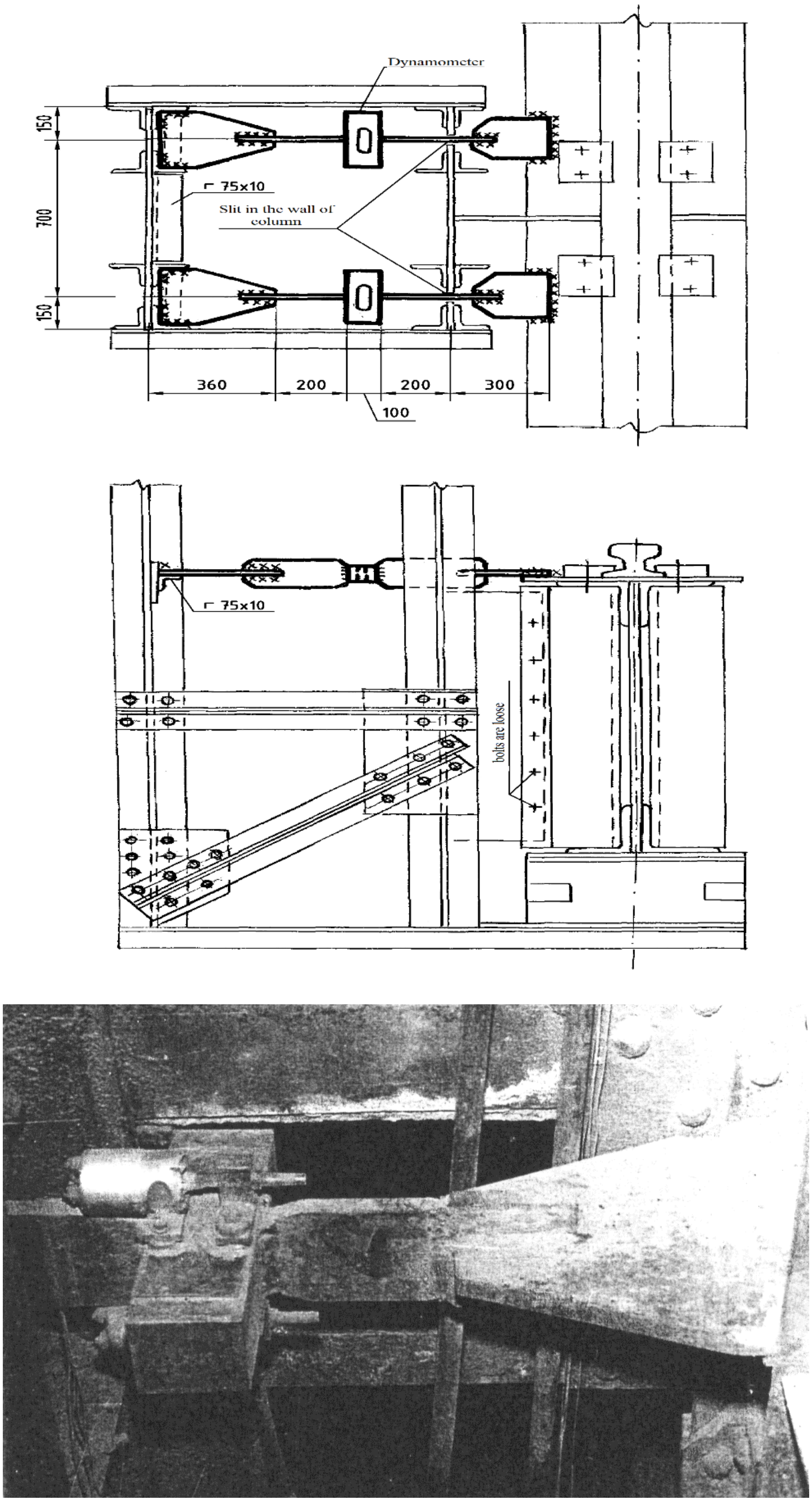

Fig. 13.16. Installation of lateral force dynamometers

Fig. 13.15 shows the installation of the slit dynamometer developed by the Test Station of the Department of Metal Structures MISI. Horizontal loads of bridge cranes that were converted into deformations of the dynamometer were recorded by a mechanical recorder (developed by S.F. Pichugin [8]). The self-recorder was attached to a slit dynamometer with the help of a special platform and clamps, on which a magnifying lever was installed. Under external load, the deformations of the slit dynamometer through the magnifying lever were transferred to the lever of the recorder and were recorded on tape.

Another variant of the slit dynamometers (author A.V. Figarovsky and S.F. Pichugin) is shown in *Fig. 13.16*. The dynamometer element was made of steel St3, had a rectangular shape, cross-section of 80x100 mm and a longitudinal groove with rounded edges. Long gusset plates with a thickness of 20 mm were welded on the transverse axis of the bar. The strain of the slit dynamometer element was transmitted to strain gauges and inductive sensors that are installed over and below the dynamometer to compensate for the possible warping. During installation of the dynamometers in the workshop, the vertical gusset plates were passed through the slots in the column walls and welded to the horizontal gusset plates, which were fastened to the upper flanges of the crane runway beams and the rear walls of the column.

During the full-scale experiment in the open-hearth workshop, the apparatus that served the dynamometers was installed on a special platform that was made on the brake grille of the crane runway beam (*Fig. 13.17*).

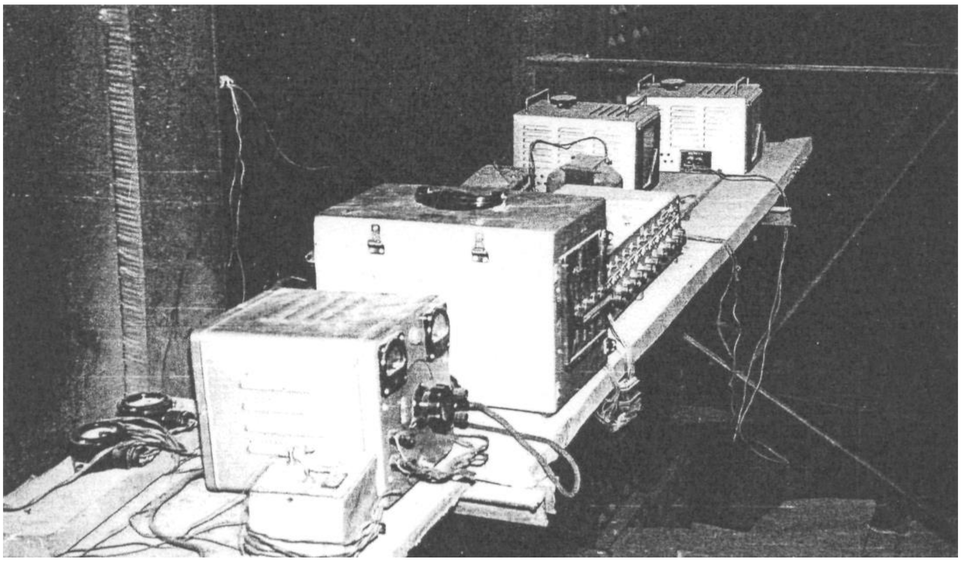

Fig. 13.17. Installation of equipment for measuring crane loads

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Control questions

1. What is the design of the overhead travelling crane?
2. How are overhead travelling cranes of general purpose classified?
3. Types of bridge cranes for special purposes.
4. Classification of metallurgical bridge cranes.
5. What is the design of underslung cranes?
6. What parameters are used to determine the operational mode of cranes?
7. Methods of experimental research of vertical crane loads.
8. Methods of experimental research of horizontal crane loads

LECTURE 14. STANDARDIZATION OF CRANE LOADS

Overhead travelling cranes are divided into:

- by the method of suspension of the cargo - cranes with a flexible and rigid suspension of the cargo;
- by the drive movement mechanism of the crane bridge - with acentral and separate drive.

Crane loads are divided into:

- vertical;
- horizontal transverse;
- horizontal longitudinal.

Loads from overhead travelling cranes and underslung cranes are variable loads, for which there are four types of design values:

- limit design values;
- operational design values;
- cyclic design values;
- quasi-permanent design values.

14.1. Limit design values of vertical crane loads

For overhead travelling cranes and underslung cranes they are defined by the formula:

$$F_m = \gamma_{fm} \psi F_0, \tag{14.1}$$

where γ_{fm} *is* reliability factor of the limit design value of the crane load (according to *Table 14.1*);

F_0 is characteristic value of the vertical load from the two most unfavorable for the impact of cranes;

ψ is combination factor of crane loads.

Characteristic values of vertical loads F_0, which are transmitted by crane wheels to the crane track beams, and other necessary data for the calculation should be taken in accordance with the requirements of national codes for cranes, and for non-standard cranes - in accordance with the data in the passports of the manufacturer.

***Note**. The crane track is defined as two beams that support one overhead travelling crane and all beams that support one underslung crane (two beams are at one-span, three are at two-span underslung crane, etc.).*

In the absence of passport data, the characteristic values of vertical pressure on the crane wheel can be determined by the following formulas (*Fig. 14.1*):

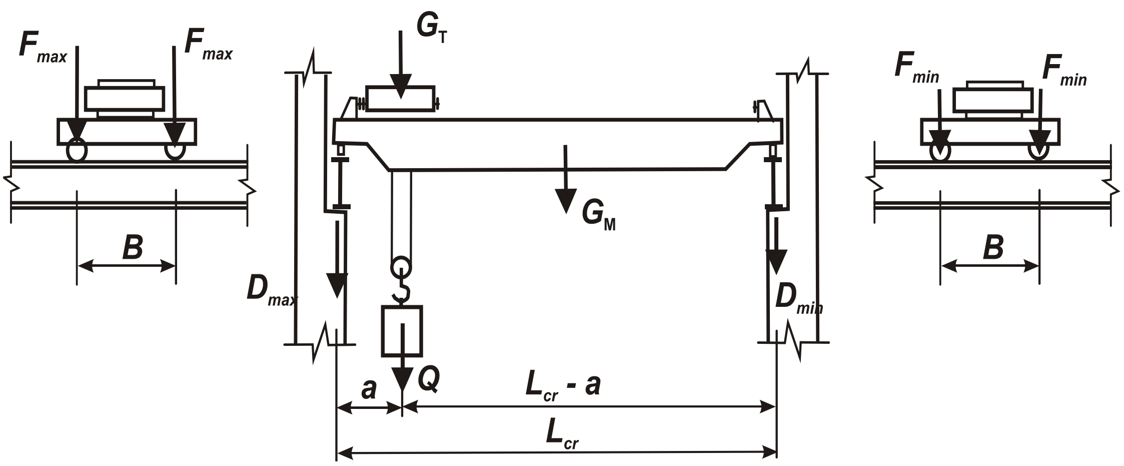

Fig. 14.1. Before determining the pressure on the wheels of the overhead travelling crane

a) the maximum pressure on the wheel on side of the crane, to which the crab with the cargo is maximally close

$$F^n_{\max} = \left[\frac{G_M}{2} + \frac{(Q + G_T)(L_{cr} - a)}{L_{cr}} \right] \frac{1}{n_0}, \tag{14.2}$$

where Q is hoist load, G_M is self-weight of the crane, G_T is weight of the crane crab, L_{cr} *is* crane span, *a is* minimum proximity of the crane hook to the crane rail axis, n_0 *is* number of wheels on one side of the crane*;*

b) the minimum pressure on the wheel on the opposite side of the crane

$$F^n_{\max} = \left[\frac{G_M}{2} + \frac{(Q + G_T)a}{L_{cr}} \right] \frac{1}{n_0}. \tag{14.3}$$

Similar result is given by the formula

$$F^{n}_{\min} = \frac{Q + G_T + G_M}{n_0} - F^{n}_{\max} \,. \tag{14.4}$$

For multi-wheeled cranes, the wheels pressures may be slightly different, and in practice, the average maximum and minimum values of the pressure on the wheels are usually taken into account.

Taking into account the factors which are given in formula (*14.1*), limit design values of the vertical loads,for example, on the transverse frame of the one-floor industrial building, are defined as

$$D_{\max} = \gamma_{fm}\psi F_{0\max} + G_{nk} = \gamma_{fm}\psi F^{n}_{\max}\sum_{i=1}^{n} y_i + G_{nk};$$

$$D_{\min} = \gamma_{fm}\psi F_{0\min} + G_{nk} = F^{n}_{\min}\sum_{i=1}^{n} y_i + G_{nk}, \tag{14.5}$$

where $\sum_{i=1}^{n} y_i$ is sum of ordinates line of influence of the supporting pressure on the column;

G_{nk} is weight of crane structures.

14.2. Limit design values of the horizontal load of overhead travelling cranes across the crane rail

Four-wheeled cranes. In terms of values and nature of horizontal loads these cranes are allocated into a separate crane group, which is susceptible to to skewing during moving. Especially this tendency happens for cranes with the ratio of the span to the base $L_{cr}/B \geq 5$. The greatest pressure is realized by four-wheel cranes in the so-called "most skewed position", when the skew of the crane bridge is limited by the flanges of the wheels which contact with the rails. The skew limitation can be performed by wheels on one side of the crane or by wheels that are on the diagonal of the crane.

The limit design value of the transverse crane load is determined by the formula:

$$H_m = \gamma_{fm} H_{01}, \tag{14.6}$$

where H_{01} is the characteristic value of lateral force from ***one crane***, the most unfavorable for the impact of the cranes that are located on one crane track or in the same section.

The characteristic value of the horizontal load of four-wheeled overhead travelling cranes that is directed across the crane runway, which is caused by skewing electric bridge cranes and non-parallel crane tracks (***lateral force***), for the crane wheel should be determined by the formula:

$$H_k^n = 0{,}1F_{\max}^n + \frac{\alpha\left(F_{\max}^n - F_{\min}^n\right)L_{cr}}{B}, \tag{14.7}$$

where F^n_{max}, F^n_{min} are characteristic values of vertical pressure on the wheel, according to more or less loaded side of the crane;

B, L_{cr} are base and span of the crane;

α is factor, which is taken equal to 0,03 at the central drive of the crane and 0,01 is at the separate drive.

Lateral forces H_k^n, calculated by formula (14.7), can be applied:

- to the wheels of one side of the crane and forces are directed to different sides (inside or outside the span of the building that is being considered), which corresponds to the limitation of the skew of the crane wheels of one side (*Fig. 14.2, a*);
- to the wheels along the crane's diagonal and forces are directed to different sides (inside or outside the span of the building that is being looked at), which corresponds to the case of limiting tthe skew of crane by wheels that are on the crane's diagonal (*Fig. 14.2, b*).

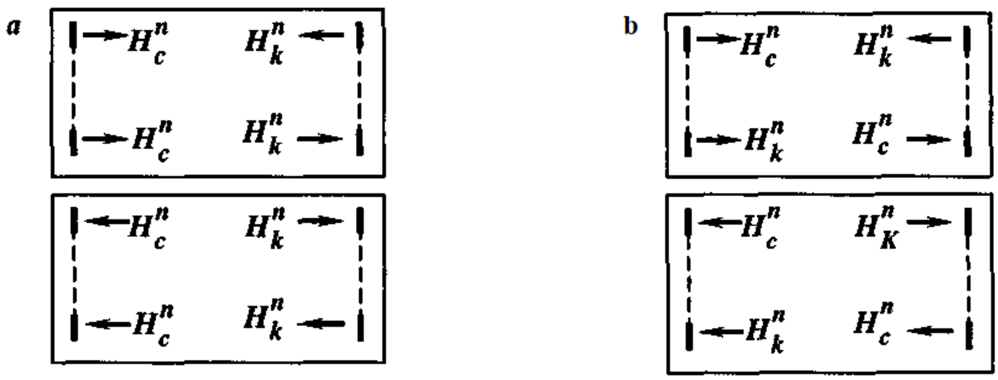

Fig. 14.2. Variants of the application of the lateral forces for four-wheeled cranes

At the same time to other wheels the applied forces are equal $H_c^n = 0{,}1F_{max}^n$ or $H_c^n = 0{,}1F_{min}^n$ (unfavorable variant is accepted), each of which can be directed both outside and inside.

For the calculation of structures it is recommended to choose from the given schemes the most unfavorable schemes of loading structures that are calculated.

In formula (14.7), the first component expresses the limit force at skewing of the wheel plane in relation to the longitudinal axis of the rail with absence of contact of the wheel body with the rail (*Fig. 14.4, b*). This is based on the experimentally confirmed proportionality factor, which links the force of transverse movement of the steering wheel and the vertical load that is acting on the wheel. The second component takes into account the effect on the lateral force by the presence of moment from the skew of the bridge in the plan. It shows the main impact of the ratio of the crane span to the base L_{cr}/B, as well as the type of drive of the crane bridge movement mechanism, because at the separate drive the skew of the bridge during the crane movement decreases in comparison with the central drive.

Experimental tests of real cranes in operating workshops have shown that the lateral forces of two adjacent cranes are insignificantly different from the lateral forces of one crane. Therefore in DBN lateral force from one four-wheeled crane on each crane track is normalized.

Multi-wheeled (with eight wheels and more) cranes. Limit design value of horizontal load of multi-wheeled cranes directed across the crane track is defined as:

$$H_m = \gamma_{fm} H_0, \qquad (14.8)$$

where H_0 is the characteristic value of lateral force from the ***two*** most unfavorable cranes that are located on one crane track or on different tracks in the same section.

Characteristic value of lateral force is assumed to be equal to 0.1 of the vertical load on the wheel. This value is applied to the wheel of multi-wheeled cranes with flexible suspension H_k^n and it is calculated when location of the crab with cargo, as equal to the passport capacity of the crane, in the middle of the bridge. The characteristic value H_k^n for multi-wheeled (eight wheels or more) cranes with a rigid suspension is assumed to be equal to 0.1 of the maximum vertical load on the wheel. It is assumed that the lateral forces of all wheels of each side of the crane have the same direction - inside or outside of the viewed span of the building (*Fig. 14.3, a, b*).

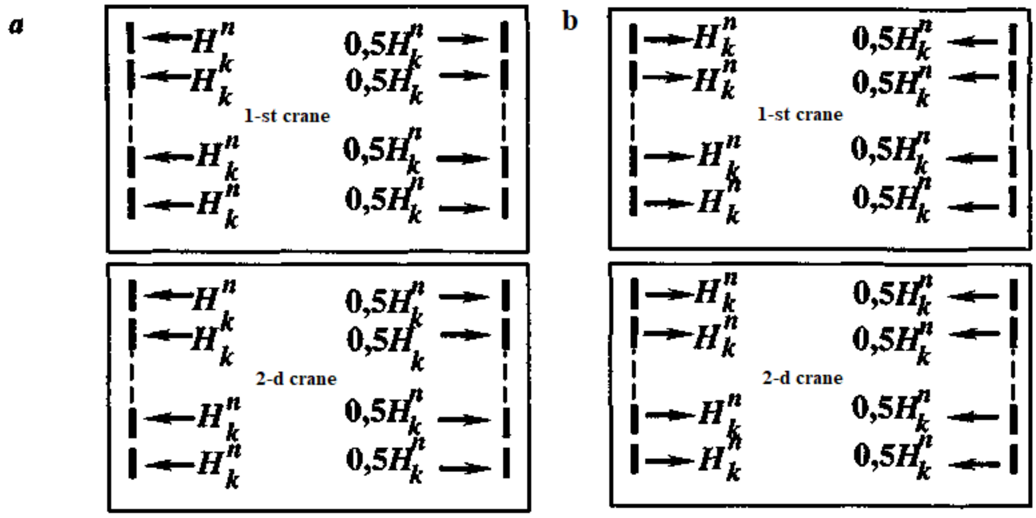

Fig. 14.3. Variants of the application of the lateral forces for multi-wheeled cranes

When determining the characteristic values of loads H_k^n it should be considered that the lateral forces of multi-wheeled cranes are transmitted to both sides of the crane track. On each side of the crane lateral forces have one direction - outside or inside, on different tracks they are directed in opposite directions (both inside or both outside). On one of the tracks the full lateral force is accepted, on the other track half of the lateral force is accepted.

14.3. Rationale for standardization of lateral forces

Large number of experimental studies have shown that the main part (about 70...90 %) of transverse loads of all overhead travelling cranes without exception is formed by the lateral forces. By the physical nature it is the friction forces that occur during transversal slide of the wheels when the crane is moving. This transverse slide is the result of mismatch between plane of the crane wheel's rotation and direction of its movement, that is skewing the wheel at an angle γ (*Fig. 10.6, a*)

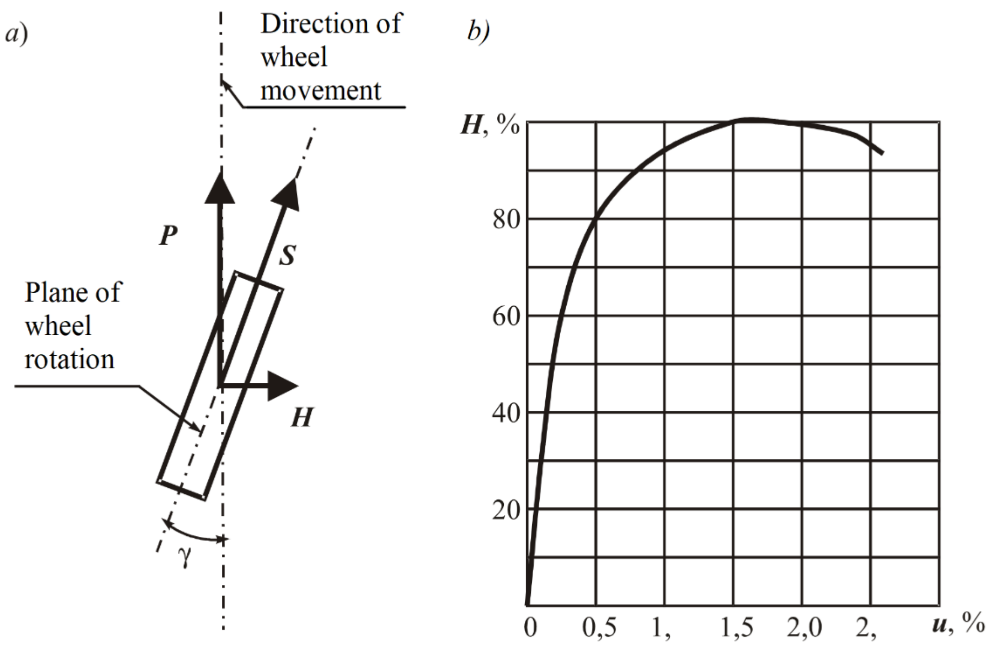

Fig. 14.4. Lateral forces of overhead travelling cranes:
a - force interaction at the point of contact of the wheel with the rail;
b - dependence between angle of wheel skew and lateral force

The general nature of the dependence of the lateral force on the angle γ is illustrated by the graph in *Fig. 14.4, b*. The value of the lateral force is plotted here along the y-axis, and the maximum value $H = F\psi$ is taken as 100%, where F is the vertical pressure on the wheel, ψ - coefficient of adhesion. The relative transverse slip of the wheel is equal to $u = tg\gamma \approx \gamma$ and is plotted on the abscissa axis. As can be seen in the graph, the force H increases up to 100% at a certain critical value $u_{cr} = 1{,}5\ldots2\%$, after which H decreases. This picture of the force interaction is observed in the absence of contact of the wheel flange with the rail.

The skewings are characteristic of any overhead travelling crane that is moving along the real crane tracks. They are generally irresistible, because they are caused by many reasons, in particular:

- skew of the bridge during its movement;
- non-parallelism of crane tracks;
- inaccuracy of installation and uneven wear of running wheels;
- different coefficient of friction of crane tracks due to uneven lubrication of tracks with fuel oil and lubricant, the presence of water and ice on the rails, contamination of tracks, etc.;
- flexibility of nodal connections of the crane bridge;
- malfunctions of the crane bridge movement mechanism, etc.

In order to illustrate the character of changes in lateral forces, there are characteristic records of these loads on columns during crane operations in *Fig. 14.5.* The main part of lateral force is the static component, that is the force of the wheels friction on the rails, which changes quite smoothly when the crane passes over the column that is being tested.

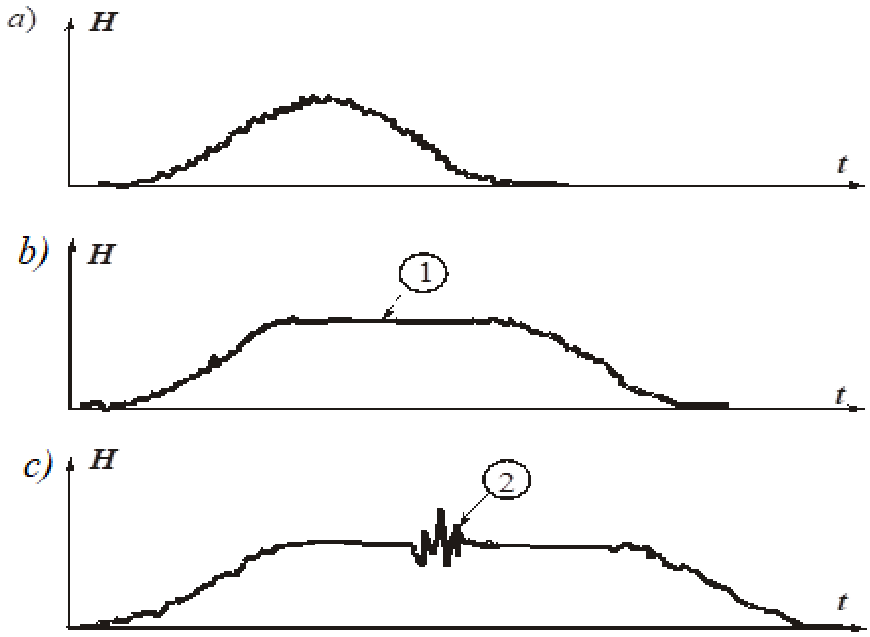

Fig. 14.5. Records of lateral forces:
a - passing without a stop; b - passing with a stop (1);
c - passing with a stop and braking (2)

A relatively small dynamical component, in the order of 10...20 %, is superimposed on the basic graph of lateral force, which has reasons analogous to dynamics of vertical crane loads. At the crane stop level of lateral force is constant, what proves its nature as force of friction, as well as static nature. During crane operations with starting and braking of the crab, a rapidly damping inertial component appears, the ordinates of which do not exceed 10…30%.

Values of lateral forces of four-wheeled cranes, which were calculated according to the formula (14.7) and were obtained experimentally, are approximately 2...5 times higher than braking forces according to SNIP [2].Therefore it can be asserted that earlier codes in Ukraine decreased horizontal loads of four-wheel cranes, especially for cranes of 1K...4K mode groups. Taking this into account, the formula (14.7), at the suggestion of the author of the manual was included in the DBN [1].

14.4. Limit design value of horizontal loads of underslung cranes across the crane rail

This value is determined by the formula:

$$R_m = \gamma_{fm} R_0. \tag{14.9}$$

where R_0 (T is another sign) is the characteristic value of transverse horizontal loads of the two most unfavorable for the impact of underslung cranes.

The specified characteristic value of the horizontal load, which is caused by braking of the crab of the underslung cranes that is directed across the crane track, should be taken as equal to 0.05 of the sum of the load capacity of the crane and the weight of the crab:

$$T_{cr}^n = 0{,}05(Q + G_B). \tag{14.10}$$

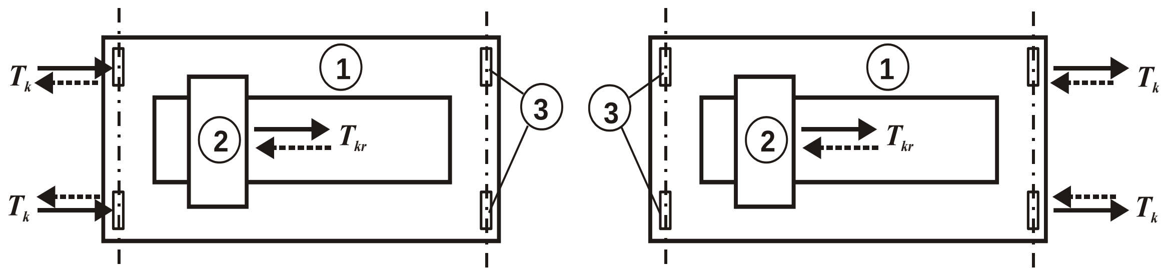

Fig. 14.6. Scheme of crane braking forces application:
1 - crane bridge; 2 - crab; 3 - crane wheels

When determining the characteristic values of the loads R_0 it is assumed that braking force is transmitted to one side (beam) of the crane track, it is distributed equally among all wheels of the underslung crane and can be directed both inside and outside of span (*Fig. 14.6*). So, the braking force, which is transmitted to one wheel of the crane, is defined as

$$T_k^n = T_{kr}^n / n_0, \tag{14.11}$$

where n_0 is the number of wheels on one side of the crane.

It should be noted that in the codes of SNiP [2] braking forces were taken into account not only for underslung cranes, but also for overhead travelling cranes.

14.5. Other design values of the crane loads

Limit design value of horizontal load of overhead travelling cranes and undercarriage cranes, which is directed along the crane track:

$$P_m = \gamma_{fm} P_{01}, \tag{14.12}$$

where P_{01} is characteristic value of the horizontal load from one crane, which is directed along the crane track.

The characteristic value of the horizontal load P_{01} which is directed along the crane runway and which is caused by braking of the electric crane bridge, it should be taken as being equal to 0.1 of the characteristic value of the vertical load on braking wheels of the viewed side of the crane.

Operational design values are determined by the following formulas:

$$F_e = \gamma_{fe} F_{01}; P_e = P_{01}; H_e = H_{01}; R_e = R_{01}, \tag{14.13}$$

where $F_{01}, P_{01}, H_{01}, R_{01}$ are corresponding characteristic values of loads from one crane, the most unfavorable for the impact of the cranes which are located on the same crane track or in the same section;

γ_{fe} is reliability factor for operational design value of the crane load, which is assumed to be equal to one.

Cyclic design values of the crane load are determined by the formulas:

$$F_{c\max} = \gamma_{fc\max} F_{01}; \; F_{c\min} = \gamma_{fc\min} F_{01}, \tag{14.14}$$

where γ_{fc} is reliability factor for cyclic design value of the crane load.

Quasi-permanent design values of the crane load are determined by the formulas:

$$F_p = \gamma_{fp} F_{01}; H_p = \gamma_{fp} H_{01}, \tag{14.15}$$

where γ_{fp} is reliability factor for quasi-permanent design value of the crane load.

14.6. Reliability factors based on the values of crane loads

Reliability factor of limit design value of the crane load γ_{fm} is determined depending on the given average period of recurrence T according to *Table 14.1.*

Table 14.1

Reliability factor of limit design value

T, years	***≥ 50***	***10***	***1***	***0,1***
γ_{fm}	*1,1*	*1,07*	*1,02*	*0,97*

Intermediate values of factor γ_{fm} should be determined by linear interpolation.

The graph in *Fig. 14.7* shows the relation of factor γ_{fm} to the period of recurrence of the crane load T. This factor is based on determination of the standard deviation from the mathematical expectation of the limit design value of the crane load at a given probability of its exceeding:

$$\gamma(T) = \sqrt{2\,ln\frac{\omega T}{2\pi Q(T)}}. \qquad (14.16)$$

Thus, with sufficient accuracy, required coefficient can be approximately determined from the ratio:

$$\gamma_{fm}(T) = \frac{1+V\gamma(T)}{1+V\gamma(T=50years)}. \qquad (14.17)$$

Here $V = \hat{X}/\bar{X}$ is variation factor of crane load.

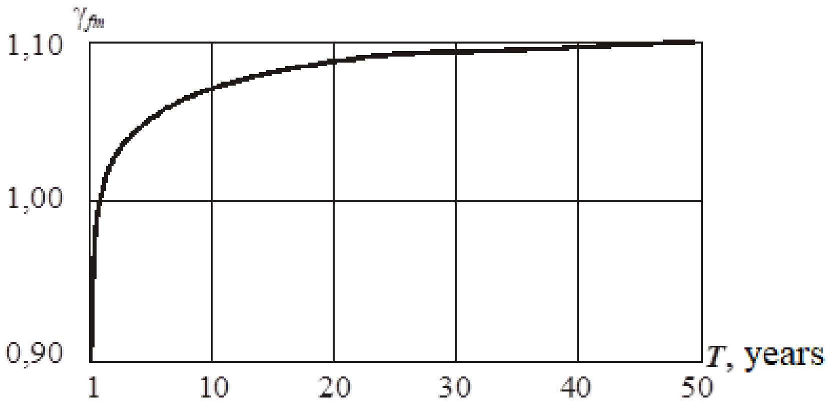

Fig. 14.7. Dependence of the reliability factor γ_{fm} from the period of recurrence

Reliability factors of the cyclic design value for the crane load are determined depending on the characteristic of capacity $g=Q/G_k$ (Q is crane capacity, G_k is weight of crab and bridge) by the following formulas

$$\gamma_{c\max} = 0{,}75\text{-}0{,}24g;\ \gamma_{c\min} = 0{,}34\text{-}0{,}24g. \qquad (14.18)$$

The number of loading cycles (per day) must be taken in a such way that it is equal to:

- n_c = 270 – for overhead travelling cranes of the 4К – 6К mode group;
- n_c = 420 – for overhead travelling cranes of the 7К mode group;
- n_c = 820 – for overhead travelling cranes of the 8К mode group.

The given number of loading cycles should be taken into account during testing of the total endurance of crane structures. During testing the endurance of the upper zone of the crane beam, these values must be multiplied by the number of wheels on one side of the crane.

Reliability factors of quasi-permanent design value of the crane load should be calculated according to the formula

$$\gamma_{fp} = F_{01}^{P}/F_{01}, \qquad (14.19)$$

where F_{01}^{P} is characteristic value of vertical load from one crane without cargo.

14.7. Local concentrated crane loads

Standardization according to the DBN [1]. Taking into account the local and dynamic impact of the concentrated vertical load from one crane wheel, the characteristic value of this load should be multiplied by the additional factor γ_{f1}. In case of calculating the strength of crane runway beam, this factor γ_{f1} is equal to:

1,6 – for the crane operation mode group 8K with rigid suspension of the cargo;

1,4 – for the crane operation mode group 8K with flexible suspension of the cargo;

1,3 – for the crane operation mode group 7K;

1,1 - for other groups of crane operation modes.

When checking the local stability of the beam walls, the value of the additional factor γ_{f1} should be taken as equal to 1,1.

Uneven pressure of crane wheels - this is one of the sources of the above factors γ_{f1}. Experiments show that the actual vertical pressures on individual crane wheels can be significantly different from the passport values. This is because an overhead travelling crane is redundant spatial structure which has a fairly high rigidity in the vertical direction (*Fig. 13.3, 13.4*).Therefore, for example, a real four-wheeled crane while driving on real tracks at certain moments can be supported in the rails at three or even two points (which are located along the diagonal of the bridge). As a consequence, the vertical load on the wheels of the bridge crane can significantly change both in the direction of increasing and decreasing.

The reasons for this effect are the possible skew of the crane structure, which was allowed during its manufacture and installation, deflections of crane girders, deformation of columns and foundations, and the main reason is the unevenness of crane tracks in real operating conditions. Some of these unevenness are allowed by the codes of installation and operation: the difference is permitted between the marks of the heads of crane rails in one line of columns 15… 25 mm, on adjacent columns - 10… 20 mm, track slopes 1/1000. However, full-scale tests of crane tracks of existing industrial buildings showed significantly larger longitudinal slopes, vertical deviations from the design position and the difference of track marks in the transverse direction up to 50… 100 mm.

A suggestion was made to take this feature into account by multiplying the wheel pressure of the overhead travelling crane by the ***non-uniformity factor***

$$\gamma_H = 1 + \Delta F / F^n, \quad (14.20)$$

where ΔF is increase the pressure on the wheel; $F^n$is maximum normal pressure.

It was found that ΔF depends on stiffness of the crane bridge from bending and torsion and the possible vertical movement of the crane wheel. Based on the analysis of this factor, the following values of the factor of non-uniformity were recommended:

$\gamma_H = 1{,}3$ – for cranes with capacity up to 5 tf;

$\gamma_H = 1{,}2$ – for cranes with capacity of 10…50 tf;

$\gamma_H = 1{,}1$ – for cranes with capacity of 75…200 tf;

$\gamma_H = 1{,}0$ – for cranes with capacity of more than 200 tf.

This factor can be considered as a component of the factor γ_{f1} given above.

14.8. Dynamic nature of crane loads

Standardization according to the DBN [1]. During the calculation of strength and stability of crane track beams and their fastenings to supporting structures the limit design values of vertical crane loads should be multiplied by the dynamic factor, which is equal to:

- at a step of columns no more than 12 m:

k_d= 1,2 - for overhead travelling cranes of the 8K mode group;

k_d=1,1 – for overhead travelling cranes of the 6K and 7K mode group;

k_d=1,1 – for all groups of crane operation modes;

- at a step of columns over 12 m - k_d= 1,1 for overhead travelling cranes of the 8K mode group.

Limit design values of horizontal loads from the overhead travelling cranes of the operating mode group 8K should be considered with dynamic factor, which is equal to k_d=1,1.

General dynamic factor. In addition to the static effect, cranes operation causes vibrations and corresponding dynamic forces of supported structures.As an example, *Fig. 14.8* shows characteristic records of crane girders deflections during the passage of bridge cranes, where the dynamic component is clearly manifested. Dynamic effect from the crane load is mainly caused by the impact effect of the crane wheels during the crane's movement along rough tracks. Some effect can cause rotation of unbalanced parts of the crane mechanics.

Full-scale experimental studies in operating production workshops of various purposes were carried out by DSc. G.A. Shapiro [6] and DSc. A.I. Kikin [5]. The obtained results, which were completed by theoretical investigations, allowed to find out that the value of the dynamic factor under impact effect of crane loading depends on number of factors, such as:

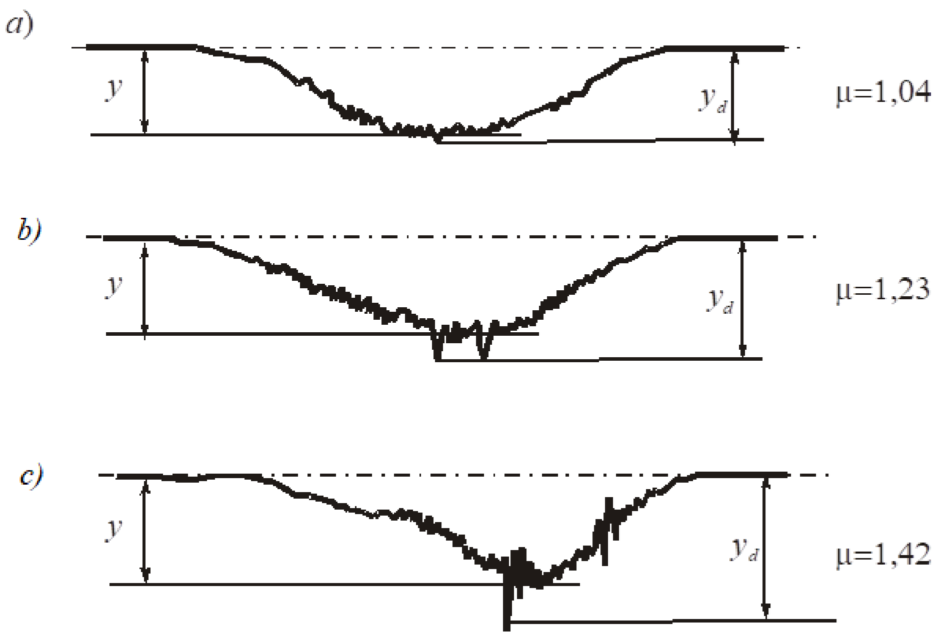

Fig. 14.8. Records of crane runway beams deflections when cranes are passing:
a - normal condition of tracks; b - presence of rails joint; c - with defect in the crane wheel

- crane speed *v*;
- height of roughnesses and difference of rails *h*;
- static deflection of crane runway beams y_S;

- crane wheel radius R;
- weight and rigidity of the crane bridge;
- rigidity of cargo suspension;
- work of a rail as an elastic gasket.

Quite wide set of factors was combined by G.A. Shapiro in the following analytical way:

$$k_d = 1 + \frac{Q}{Q_1} v\beta \sqrt{\frac{2h}{\bar{y}_S R g}}, \tag{14.21}$$

where Q_1 is weight of crane and girder, which is reduced to lumped weight by the equality of oscillation frequencies;

Q is part of the weight Q_1 (weight of one wheel) that falls from the height h;

y_S is static deflection of the beam under the action of the reduced weight Q_1;

g is gravitational acceleration;

β = 0,5…0,85 is factor that takes into account the influence of crane stiffness and suspension of cargo.

The general result of the research in this area is a scale of general dynamic factors which is more differentiated than given in the current codes (*Table 14.2*). Using the data of this table or performing the calculation according to the formula (14.21), it is possible to obtain refined characteristic of the crane influence.

As can be seen from *Table 14.2*, for cranes of light and medium mode, especially when crane runway beams have large span, as well as for cranes of heavy mode and large loading capacity, the dynamic of the vertical crane load is insignificant.

Local dynamic factor. The above described general dynamic factor is called by some researchers as the " dynamic factor of deflection or tension in the lower flange of the crane runway beams". This factor is unsuitable for the calculation of the upper part of the wall and the connection of the upper flange with wall of the crane runway beams.This is about the dynamic effect of individual crane wheels, which is taken into account by the local dynamic factor

$$k_{d,loc} = 1 + F_d / F^n, \tag{14.22}$$

where F_d is dynamic wheel pressure, F^n is normal (static) wheel pressure.

Table 14.2

Dynamic factor of crane loads

Crane operation mode	***Type of cargo suspension***	***Lifting capacity crane, tf***	***Span of crane runway beams, m***	***Dynamic factor***	
				General k_d	***Local*** $k_{d,loc}$
7К, 8К	*Rigid*	*All*	*to 12*	*1,.2*	*1,50*
			from 13 to 30	*1,15*	
			more than 30	*1,10*	
	Flexible	*30*	*to 12*	*1,20*	*1,30*
			more than 12	*1,10*	
		50 - 125	*to 12*	*1,15*	
			more than 12	*1,10*	
		150 - 225	*to 12*	*1,10*	*1,20*
			more than 12	*1,05*	
		>225	*to 12*	*1,05*	*1,10*
			more than 12	*1,00*	
1К – 6К		*≤50*	*to 12*	*1,05*	*1,05*
			more than 12	*1,00*	
		>55	*All*	*1,00*	*1,00*

The values of the local dynamic factor for cranes of various purposes are shown in *Table 14.2* (the last column). This values were determined by MIBI research.

From the above information it is clear that the physical nature of the factor γ_{f1} is determined by the unevenness and dynamic nature of the individual pressure of the wheels, as a result of which this factor can be represented as a product of the corresponding factors

$$\gamma_{f1} = \gamma_H k_{d,loc}. \tag{14.23}$$

Taking into account the above given values of non-uniformity factor $\gamma_H = 1{,}0\ldots1{,}3$ and local dynamic factor $k_{d,loc} = 1{,}0\ldots1{,}5$, it is possible to get factor values γ_{f1} for some cranes, which exceed values in acting codes. Therefore, values of this factor and values of local stresses $\sigma_{loc,y}$ in the walls of crane runway beams can be significantly underestimated, which can be one of the reasons for frequent damage of the upper zone of beams [5].

14.9. Number of cranes and combination factor

The number of cranes which are taken into account in the calculations. During the calculation of the strength and stiffness of the crane girders, the vertical load of not more than ***two*** of the most unfavorable for the impact overhead travelinf cranes or underslung cranes should be taken into account.

When calculating the strength and stability of frames, columns, foundations as well as foundations in buildings with overhead travelling cranes in several spans (in each span on one level), *the vertical loads* should be taken into account on each track no more than from ***two*** of the most unfavorable for the impact of cranes. If the cranes of different spans are combined in one section, the vertical load must be taken on each crane track no more than from ***four*** of the most unfavorable for the impact of cranes.

During the calculation of strength and stability of crane rumway beams, columns, frames, roof timbers and secondary trusses, foundations, as well as the basis, *the horizontal loads* of no more than from ***two*** of the most unfavorable for the impact of the cranes should be taken into account. In case that these cranes are located on one crane track or on different tracks in one line. For four-wheel cranes the horizontal loads should be taken into account from ***one*** crane on each crane track. Only one horizontal load (transverse or longitudinal) must be taken into account for each crane.

If there is one crane on the crane track and if the other crane will not be installed during the operation of the structure, the load on this track must be taken into account only from one crane.

Combination factor ψ, which is taken into account in formulas (14.1) and (14.5) for the loads from two cranes, is determined as follows:

ψ = 0,85 – for the crane operation mode group 1K-6K;

ψ = 0,95 – for the crane operation mode group 7K, 8K.

When taking into account four cranes, the loads of the cranes must be multiplied by the combination factor:

ψ = 0,7 – for the crane operation mode group 1K-6K;

ψ = 0,8 – for the crane operation mode group 7K, 8K.

14.10. Consideration of the crab approach limitation

During the determination the crane loads it is allowed to take into account the actual location of the crane service areas and the actual approach of the crab to the row of columns. In case, that the location and dimensions of the permanent equipment that is installed in the building do not break these limits, or that limiters (stops) of cranes movement on the tracks and limiters (stops) of crabs movement on the crane bridge are installed at the appropriate places.

If the actual approach of the overhead travelling crane crabs to the viewed row of columns, $y_{min} = y_o + pL$, exceeds the passport value y_o, then the vertical crane load on the structures of the viewed row can be adjusted by multiplying to the factor K_y, which is calculated according by the formula:

$$K_y = 1 - \frac{pL_{cr}(m_c+m_q)}{m_b} \Big/ \left(2L_{cr} + (m_c + m_q)\frac{L_{cr}-y_0}{m_b}\right), \qquad (14.24)$$

where m_c, m_b are weight of crab and bridge;

m_q is crane capacity;

L_{cr} is span of crane;

p is relative part of the crane span.

Depending on the value p and parameters of the cranes, the specified factor is in the range K_y = 0.810 ... 0.965 [7].

For the buildings that are designed and reconstructed, which have certain or established technological process, it is allowed to take into account the specific features and parameters of the operating modes and service areas of cranes.

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Control questions

1. How to determine the limit design values of the vertical crane loads?
2. How are the limit design values of the horizontal load of four-wheeled bridge cranes determined?
3. How to determine the limit design values of the horizontal load of multi-wheeled bridge cranes?
4. How to determine the limit design value of the horizontal load of underslung cranes, which is directed across the crane track?
5. How are the design values of crane loads determined?
6. How are the lateral forces of bridge cranes formed?
7. How are the reliability factors determined by the values of the crane loads?

8. What are the features of the local concentrated crane loads?

9. What is the nature of the dynamic character of crane loads?

10. How many cranes are taken into account during the calculation of the design crane loads?

11. Which value is the combination factor for the crane loads equal?

12. How is the restriction of the crane crab approaching the crane track taken into account?

LECTURE 15. ICE-WIND LOADS

15.1. Structural accidents due to ice overload

The notion of the nature and extreme values of ice loads give accidents of structures due to ice overload. Ice loads in particular affect the reliability and failure of overhead line (OHL) communications and power transmission structures. The history of construction and operation of the OHL for the last 50… 80 years contains numerous examples of destruction of structures and breakage of OHL wires, which caused great economic damage and clearly demonstrated the need to properly account for ice loads.

More than 50% of failures of OHL pylons in Ukraine are caused by ice and wind loads. The largest natural disaster in the last 50 years hit the electricity grid of Ukraine in November 2000, it paralyzed the lives of almost 5 thousand settlements in 12 regions of the south-west of the country. During the week, almost 4 million people were left without electricity, heat, gas and water. The catastrophic ice covered an area of about 226,000 km^2, and significant damage to trees, winter crops, OHL wires and electric vehicles was recorded in an area of about 118,800 km^2. More than 307 thousand reinforced concrete and more than 20 thousand steel pylons were destroyed. The intense formation of ice was the result of the interaction of cold Arctic air moving from the northeast with warm and humid air from the southwest. The formation of ice was accompanied by supercooled rain and fog at a temperature of 0… -2ºC, increased intensively for 10 ... 12 hours, its intensity exceeded that ever observed, 2 ... 4 times, the running load on the wire reached 150 N/m. The situation was aggravated by winds with a speed of 14 ... 17 m/s. The maximum diameter of ice on the measuring instruments of ice was recorded at the weather stations Zatyshshya (Odesa region) - 197 mm and Khmelnytsky - 61 mm. The total damage of Ukraine from the disaster amounted to more than 100 million euros.

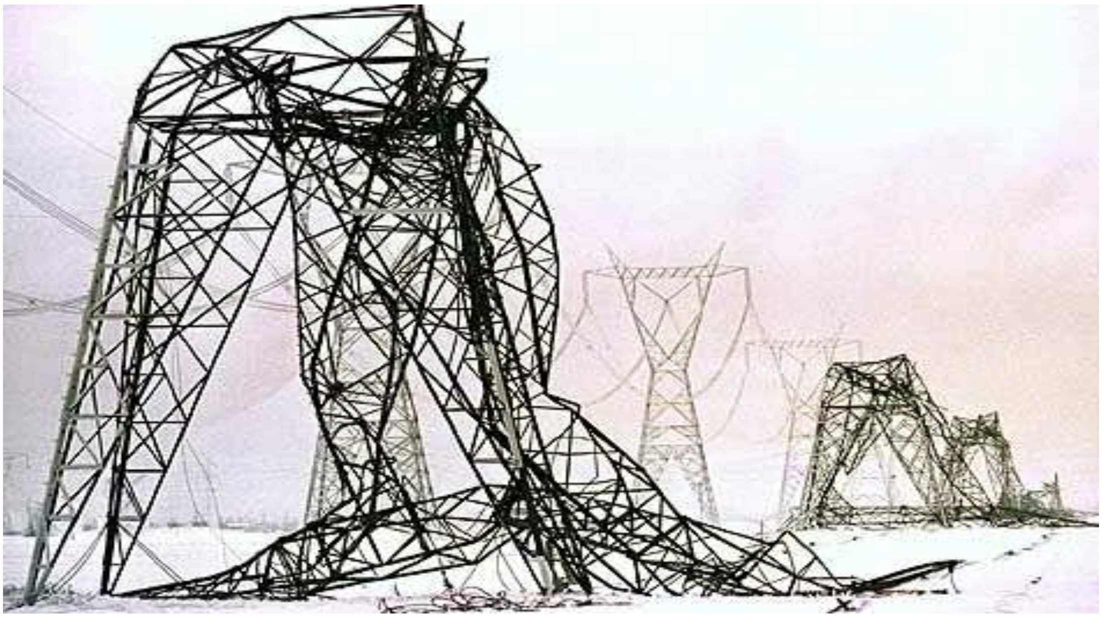
Thus, the objective assessment of ice-wind loads is very relevant, this is the subject of numerous studies, including the work of specialists PoltNTU [3, 4].

15.2. The nature of loads from ice

Ice deposits on various structures are formed as a result of:

- deposition and freezing of supercooled water droplets in the presence of fog, mist, rain;

a)



b)

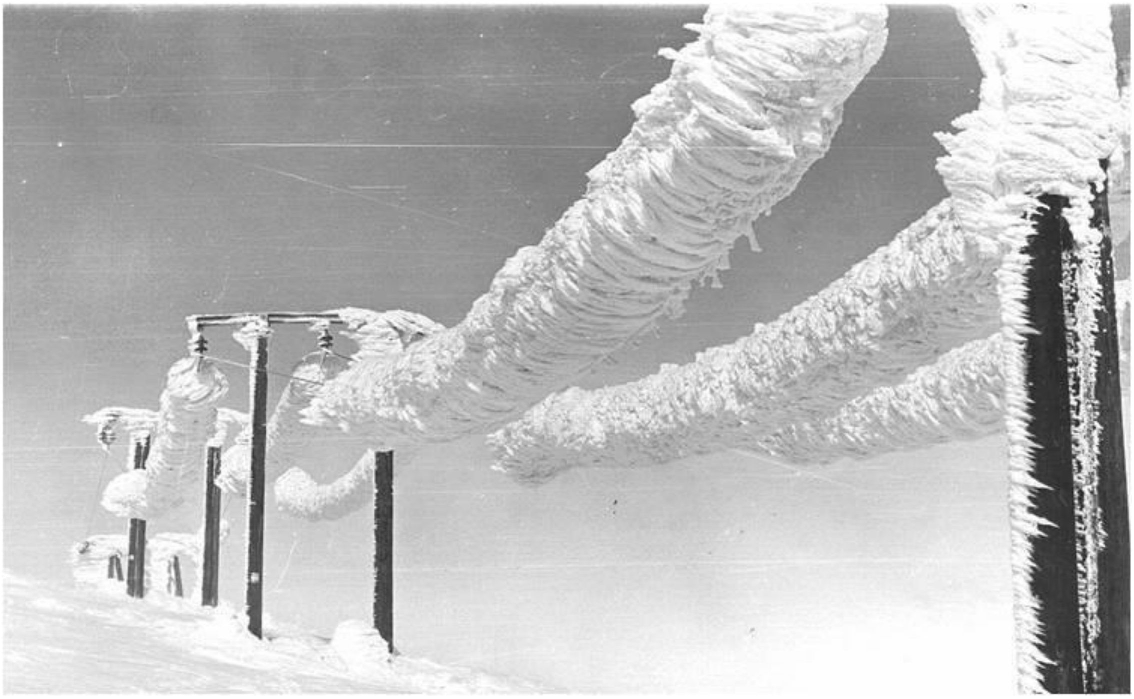


Fig. 15.1. Dangerous effects of ice:
a - destruction of the OHL support from ice load;
b - icing of electrical wires

- freezing of wet snow;
- sublimation of water vapor.

The type of ice deposits depends on the size of water droplets and the speed of their freezing in contact with structures.

• Large drops of water, which are more often observed at temperatures close to 0ºC, freeze slowly, have time to spread and form a film of water, which when frozen forms ***ice***. The most probable temperature of ice formation is in the range of 0… -3ºC, the most probable density is 0.6… 0.9 g/cm^3. Laying on the wires of overhead power lines, ice causes them to sag, vibrate, and when the wind increases, they break.

• Small water droplets freeze at lower temperatures (below - 3ºC), and freezing occurs faster, without spreading, air bubbles remain between the ice cubes. As a result, ***granular frost*** is formed with an uneven hilly surface with individual protrusions (*Fig. 15.2*), the temperature range of its formation from - 3ºC to - 8ºC, in the mountains to - 20 ...- 30 ºC, density range - 0.1… 0.6 g/cm^3. Granular frost in its structure is close to ice and is more dangerous than crystalline. When it settles on the wires, it makes communication difficult, causes them to sag, and can also lead to wire breakage.

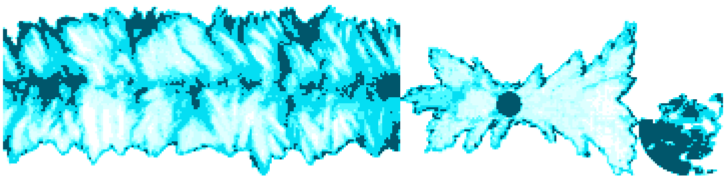


Fig. 15.2. Ice is wavy
(photo from the atlas of wire icing [5])

• In calm weather, with fine dripping fog or smoke (at high humidity) as a result of sublimation of water vapor and freezing of very small drops of water at temperatures from -10ºC to -20ºC ***crystalline frost*** is formed (*Fig. 15.3*), ie white precipitate of openwork crystal structure with a density of 0.01… 0.08 g/cm^3.

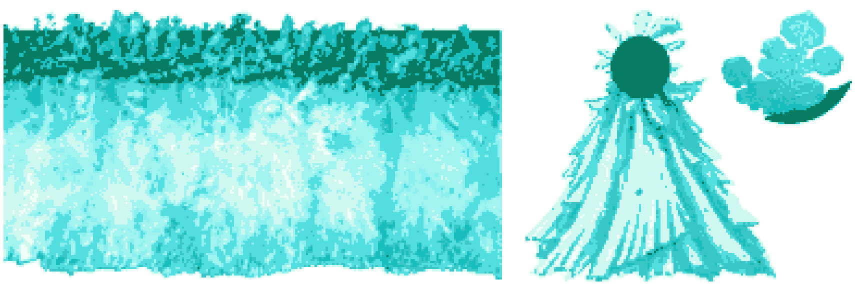


Fig. 15.3. Crystal frost (15 mm wire)

• Sticking and freezing of wet snow form ice deposits of higher density in the range of 0.10… 0.70 g/cm3 *(Fig. 15.4)*.

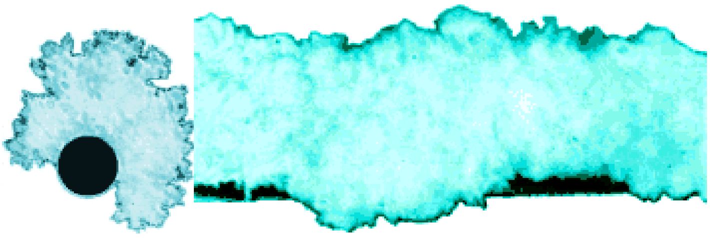


Fig. 15.4. Wet snow

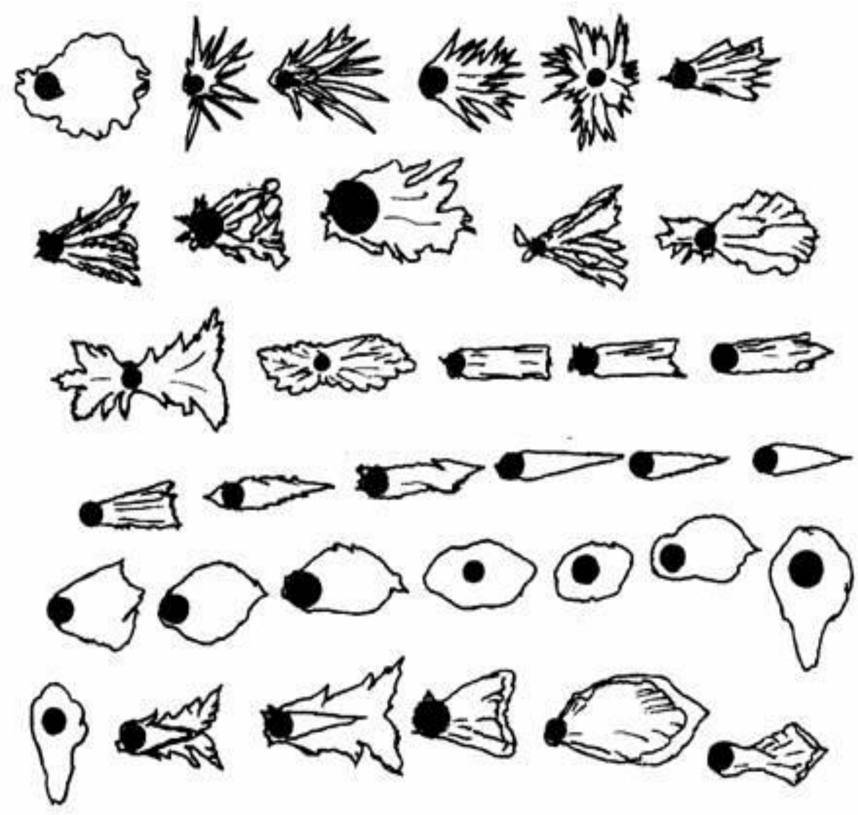


Fig. 15.5. Possible forms of ice-frost deposits on wires

Because the range of water droplets in the atmosphere is very wide, complex ice formations can occur on structures, consisting of several types or layers called ***mixed***. The average density of such a mixture can vary widely, as it depends on type of ice.

The actual forms of ice-frost deposits can be very diverse; the most characteristic of them are presented in *Fig. 15.5*.

The formation of ice deposits on structures, in addition to the above meteorological conditions (temperature, humidity, precipitation), is influenced by a number of factors:

• height position of elements;

• characteristic dimensions of the cross section of the elements: diameter, width, height;

• wind speed and direction;

• the nature of the underlying surface: steppe, pond, forest, settlement and so on

15.3. Methods of ice measuring

Different methods of ice measuring are used in meteorological and scientific research.

A. Measurements with an ice machine. At meteorological stations, most of the instrumental observations of ice deposits are carried out by this simple method. The main part of the ice machine is rigidly fixed (permanent) and removable (replaceable) rods (wires) with a diameter of 5 mm and a length of 90 cm, attached to three racks at a height of 2 m above ground level. Two wires are oriented in the meridional direction ("north-south"), two wires are oriented in the latitudinal direction ("west-east"). Removal of parameters is carried out four times a day, and at formation of ice in 2 hours. In the process of observations on a fixed wire, a large diameter (along the line of the largest deposition) and a small diameter (in a direction perpendicular to the large diameter) are measured (*Fig. 15.6*). The weight of the deposit is measured with a replacement wire.

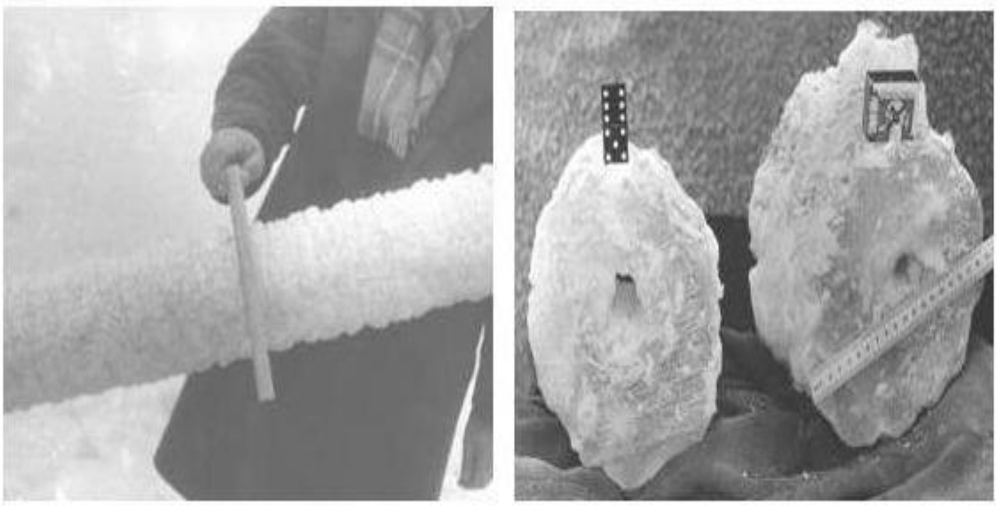


Fig. 15.6. Measuring the thickness of the ice layer

To monitor the process of deposition of ice, a part of the latitudinal and meridional wires is cleaned, having determined the type of deposition in advance. By recording the time of onset and time of cessation of deposition, the periods of its growth and destruction are determined, as well as the total duration of the period during which ice deposits were stored on wires.

B. Instrumental measurements of ice. In various industries, including gas, oil, chemical, etc., ice-frost sediment detectors have been developed and are used. The purpose of these devices is to sound an alarm signal at the beginning of ice formation. Detector examples include acoustic detectors, including the domestic CO-1 and the Swiss EW-140, which are based on an increase in the natural frequency of the membrane due to increased stiffness caused by ice. Optical signaling devices for the onset of ice formation have also been developed.


a)


b).


c)


d).

Fig. 15.7. Instrumental measurements of ice:
a - increase in ice on a rotating installation (UK); b - freely rotating rod with measuring equipment (Czech Republic); c - measuring equipment EAG 200 (Germany); d - Rosemount 872 / C3 icing detector (Canada-USA)

Devices have been developed abroad to measure the mass of ice deposits. Among them there are the vibration detector type ASOS (USA) and the ice detector (Canada), which use a sensitive element in the form of an aluminum rod

(*Fig. 15.7, d*). Ice sensors were developed in a number of countries, its allow estimating the mass of ice on a vertical rod using measuring techniques (*Fig. 15.7, a, b, c*).

B. Domestic developments. The development of NEC Ukrenergo (G.I. Grimud) of a system for monitoring ice and wind loads on overhead power lines seems to be very timely. The main component of the system was an automated ice-wind meteorological station (AGVMP). The developed post used the latest domestic and foreign achievements in this field, it includes a strain gauge of ice mass, ultrasonic anemorumbometer, temperature and humidity meters, signals from which are transmitted to a computer.

The first 4 weather stations were installed in the systems of NEC "Ukrenergo" in Odessa, Ternopil and Khmelnytsky regions. The experience of AGVMP operation in 2004-2007 revealed insufficient reliability of meteorological stations due to the complexity of the design due to the presence of moving parts. In 2007… 2008 the structure of the meteorological station was significantly modernized, the device for measuring ice and frost deposits and the system of automatic control of measured parameters were replaced.

In the future, it is planned to equip substations with such posts, and eventually directly transmission lines for continuous collection of meteorological information on the route of the transmission line and, if necessary, the issuance of alarms. The task is to create a system for monitoring climatic parameters, especially ice and wind loads, on the routes of the transmission line of NEC "Ukrenergo", which will significantly replenish the database of ice loads in Ukraine.

15.4. Determination of ice loads

According to DBN [1], ice-wind loads should be taken into account when designing overhead communication lines, contact networks of electrified transport, antenna-mast devices and other similar structures.

Ice-wind loads should be considered as a combination of the weight of ice deposits and the normal wind pressure on the ice-covered elements.

Ice-wind loads are variables, for each component of which (ice deposits and wind) limit design values are set.

The limit design value of the weight of ice deposits is determined by the formula

$$G_m = G_e \gamma_{fm}, \tag{15.1}$$

where γ_{fm} is the reliability factor for the limit value of the weight of ice deposits, determined in accordance with paragraph 15.6;

G_e is the characteristic value of the weight of ice deposits, determined by formula (15.2) for linear ice load and (15.3) for surface ice load.

The characteristic value of the linear ice load (N/m), for elements of circular cross-section up to 70 mm in diameter (wires, cables, mast extensions, cables, etc.) should be determined by the formula

$$G_e = \pi b k \mu_1 (d + b \quad k\mu_1) \rho g 10^{-3}, \qquad (15.2)$$

where b is the wall thickness of the ice, mm, determined according to *Table 15.1*, taking into account the requirements below;

k is the coefficient that takes into account the change in the wall thickness of the ice depending on height h and is taken from *Table 15.2*;

d is the diameter of the wire, cable, mm;

μ_1 is a coefficient that takes into account the change in the wall thickness of the ice depending on the diameter of the elements of the circular cross section d and is taken from *Table 15.3*;

ρ is ice density, which is taken as 0.9 g/cm^3;

g is the acceleration of free fall, m/s^2.

Table 15.1

Dependence of ice wall thickness on height

Height above the surface earth h, m	***Ice wall thickness b, mm***	
	1–3 ice regions	***4–6 ice regions and mountainous areas***
200	*35*	*Accepted on the basis of special examinations*
300	*45*	*Accepted on the basis of special examinations*
400	*60*	*Accepted on the basis of special examinations*

Table 15.2

Dependence of coefficient k on height

Height above the surface earth h, m	***5***	***10***	***20***	***30***	***50***	***70***	***100***
Coefficient k	*0,8*	*1,0*	*1,2*	*1,4*	*1,6*	*1,8*	*2,0*

Table 15.3

Dependence of the coefficient μ_1 on the diameter of the element

Diameter of wire, rope or cable, d, mm	***5***	***12***	***20***	***30***	***50***	***70***
Coefficient μ_1	*1,1*	*1,0*	*0,9*	*0,8*	*0,7*	*0,6*

Notes (to *Table 15.1–15.3*):

1. Intermediate values should be determined by linear interpolation.

2. The thickness of the ice wall on the suspended horizontal elements of circular cross-section (cables, wires, ropes) may be taken at the height of the location of their reduced center of gravity.

3. The wall thickness of ice on wires up to 10 mm in diameter should be taken as on wires with a diameter of 10 mm.

4. To determine the ice load on the horizontal elements of a circular cylindrical shape with a diameter of up to 70 mm, the wall thickness of the ice (*Table. 15.1*) should be reduced by 10%.

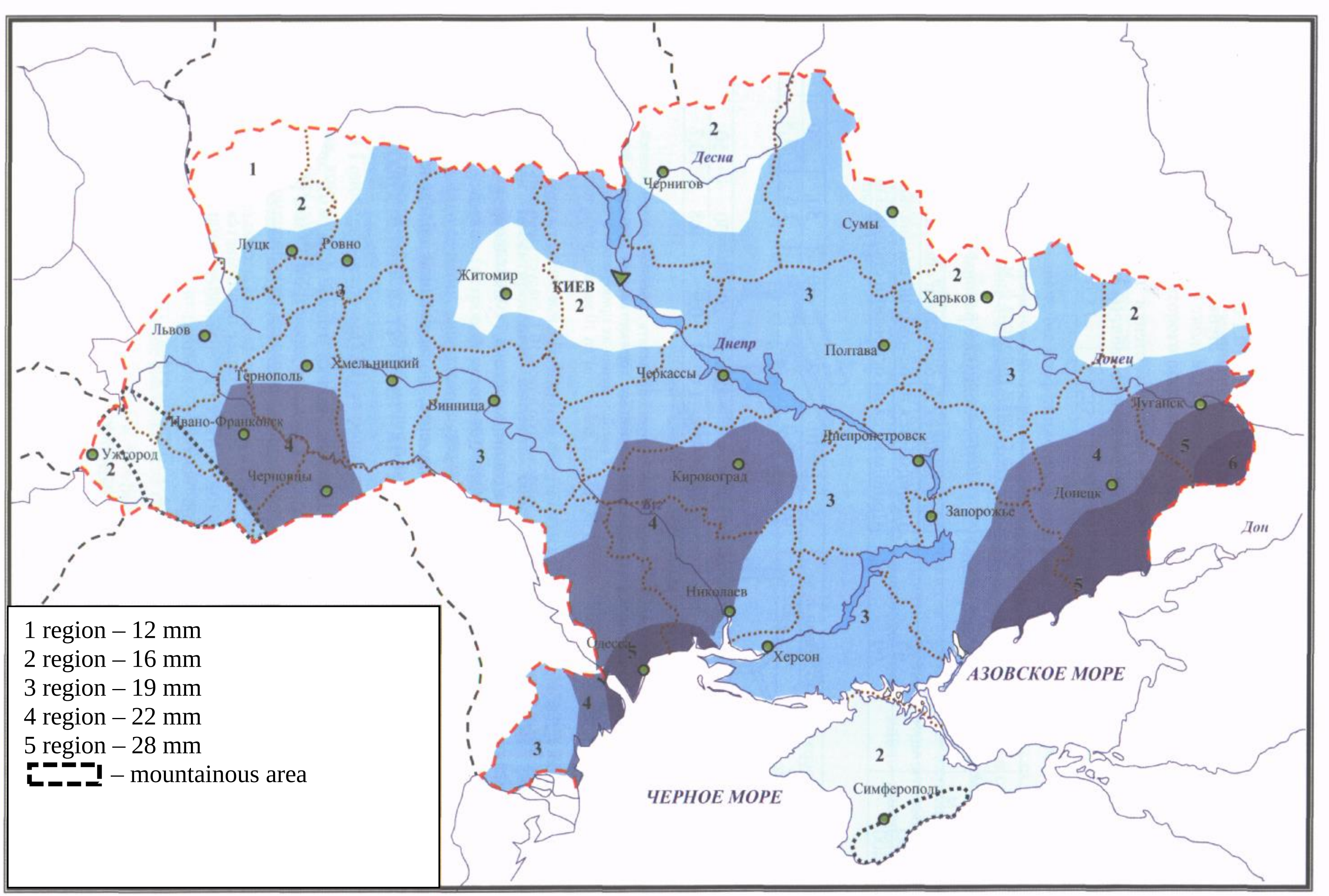


Fig. 15.8 Map of zoning of the territory of Ukraine by characteristic values of ice wall thickness

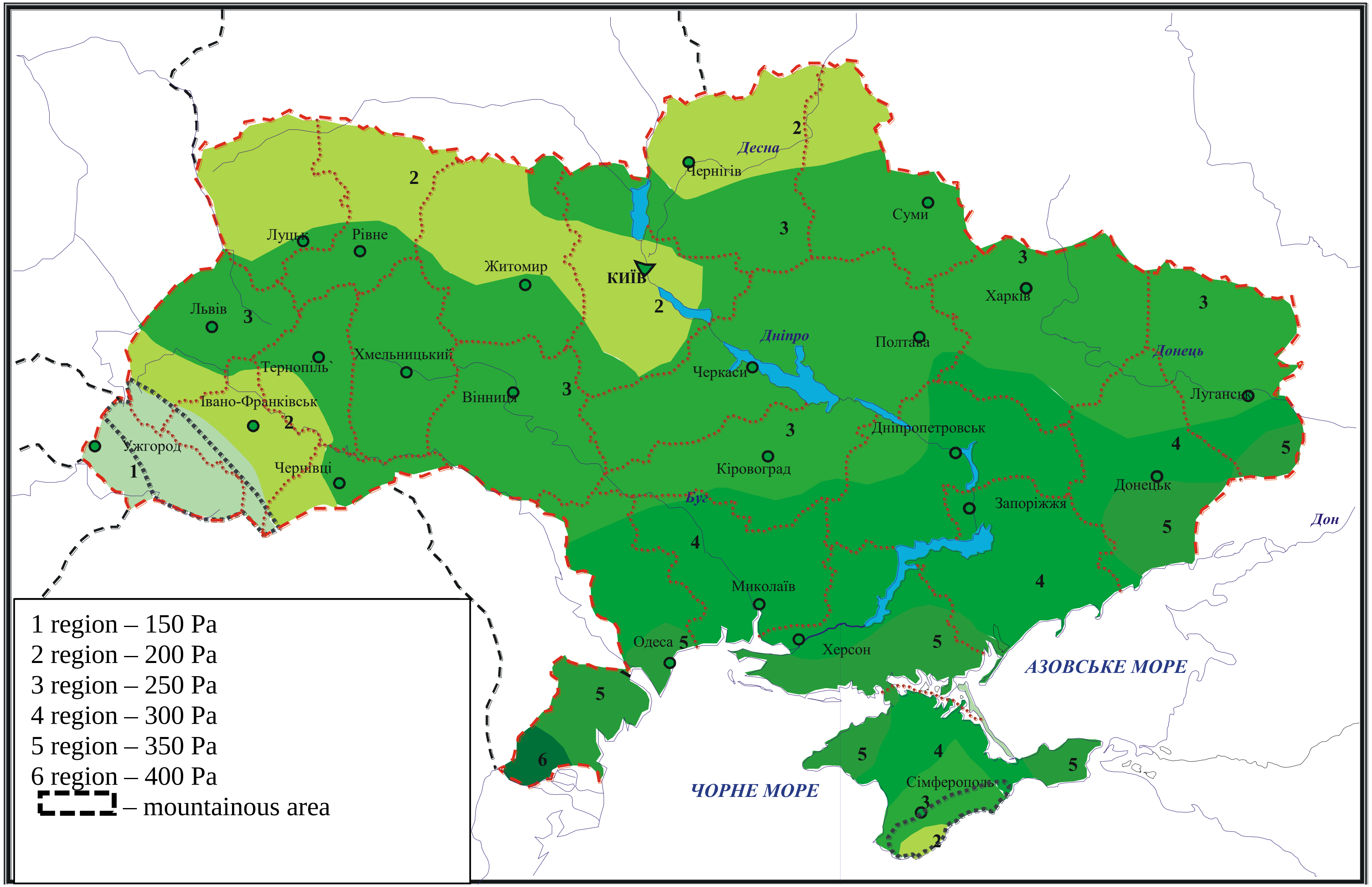


Fig. 15.9. Map of zoning of the territory of Ukraine according to the characteristic values of wind pressure during ice

The limit design value for the surface ice load (Pa) on planar elements should be determined by the formula

$$G_e = bk\mu_2\rho g, \tag{15.3}$$

where μ_2 is the ratio of the surface area of the element to be iced to the total surface area of the element. In the absence of these observations, it is allowed to take μ_2 equal to 0.6. Other notations are the same as in formula (15.2).

Characteristic value of ice wall thickness b (mm) is exceeded on average once in 50 years, on elements of circular cross-section with a diameter of 10 mm, located at a height of 10 m above the surface land. It is determined depending on the ice region on the map (*Fig. 15.8*) or Annex E [1]. Ice wall thickness b (mm) at a height of 200 m and above is taken from *Table 15.1*. For mountainous areas of the Carpathians and Crimea, as well as in very rugged areas (on top of mountains and hills, on passes, on high embankments, in closed mountain valleys, depressions, deep excavations, etc.), data on ice wall thickness should be based on special observations.

15.5. Determination of wind loads during ice

The limit design value of normal wind pressure on ice-covered elements is determined by the formula

$$W_q = W_0\gamma_{fw}, \tag{15.4}$$

where γ_{fw} is the reliability factor for the limit value of normal wind pressure on the ice-covered elements, determined in accordance with paragraph 15.6.

The characteristic value of normal wind pressure on ice-covered elements at a height of 10 m above the ground, which is exceeded once every 50 years (W_B), is taken depending on the wind region during ice on the map (*Fig. 15.9*) or Annex F [1]. For the mountainous regions of the Carpathians and Crimea, data on wind pressure during ice should be taken on the basis of special observations.

The wind pressure on the ice-covered elements is determined by formulas (12.1) and (12.3), replacing W_0 with W_B and assuming C_{rel} = 1, C_{dir} = 1 and C_{d} = 1.

15.6. Coefficients of the method of calculating ice-wind loads

The reliability coefficient for the limit value of the weight of ice deposits γ_{fm} is determined depending on the specified average recurrence period T according to *Table 15.4.*

Table 15.4

Dependence of the coefficient γ_{fm} on the recurrence period *T*

T, years	***5***	***10***	***15***	***25***	***40***	***50***	***70***	***100***	***150***	***200***	***300***	***500***
γ_{fm}	*0,46*	*0,63*	*0,72*	*0,84*	*0,95*	*1,00*	*1,08*	*1,16*	*1,25*	*1,32*	*1,42*	*1,53*

Intermediate values of the coefficient γ_{fm} should be determined by linear interpolation.

The coefficient of reliability according to the limit value of the normal wind pressure on the ice-covered elements γ_{fw} is determined depending on the specified average recurrence period *T* according to *Table 15.5*.

Table 15.5

Dependence of the coefficient γ_{fw} on the recurrence period *T*

T, years	***5***	***10***	***15***	***25***	***40***	***50***	***70***	***100***	***150***	***200***	***300***	***500***
γ_{fw}	*0,45*	*0,61*	*0,71*	*0,83*	*0,95*	*1,00*	*1,08*	*1,16*	*1,26*	*1,33*	*1,43*	*1,55*

Intermediate values of the coefficient γ_{fw} should be determined by linear interpolation.

The average recurrence period T for mass construction sites may be taken to be equal to the specified service life T_{ef} of the structure.

For objects with a high level of responsibility, for which the technical task sets the probability of *P* not exceeding (providing) the limit design value of ice-wind loads during the established service life, the average recurrence period of the limit value of ice-wind loads is calculated by the formula

$$T = T_{ef} K_p, \tag{15.5}$$

where K_p is the coefficient determined by *Table. 15.6* depending on the probability *P*.

Table 15.6

Determination of the coefficient K_p for the responsible objects

P	***0,37***	***0,5***	***0,6***	***0,8***	***0,85***	***0,9***	***0,95***	***0,99***
K_p	*1,00*	*1,44*	*1,95*	*4,48*	*6,15*	*9,50*	*19,50*	*99,50*

Intermediate values of the coefficient K_p should be determined by linear interpolation.

When determining the wind loads on the elements of structures located at a height of more than 100 m above the ground, the diameter of icy wires and cables, set taking into account the wall thickness of ice, shown in *Table 15.1*. For ice-wind regions 1 – 3 in *Fig. 15.1* and Annex E the wind load must be multiplied by a factor of 1.5.

The air temperature during ice, regardless of the height of buildings, should be taken in mountainous areas with altitudes: over 1000 m – minus 10 ° C; for the rest of the territories for buildings up to 100 m high – minus 5 ° C, over 100 m – minus 10 ° C.

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Control questions

1. Give examples of accidents due to ice and wind loads.
2. What is the physical nature of the formation of ice deposits?
3. How are ice measurements performed?
4. How are the limit values of ice load determined?
5. How is the characteristic value of ice load determined?
6. On what parameters depends the characteristic value of the ice load?
7. How to determine the characteristic value of ice wall thickness?
8. How is the wind load determined during ice?
9. What are the coefficients of the method of calculating ice-wind loads?

ПІЧУГІН СЕРГІЙ ФЕДОРОВИЧ

КЛОЧКО ЛІНА АНДРІЇВНА

ОКСЕНЕНКО КАТЕРИНА

МЕТОДИКА ГРАНИЧНИХ СТАНІВ І НОРМУВАННЯ НАВАНТАЖЕНЬ

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