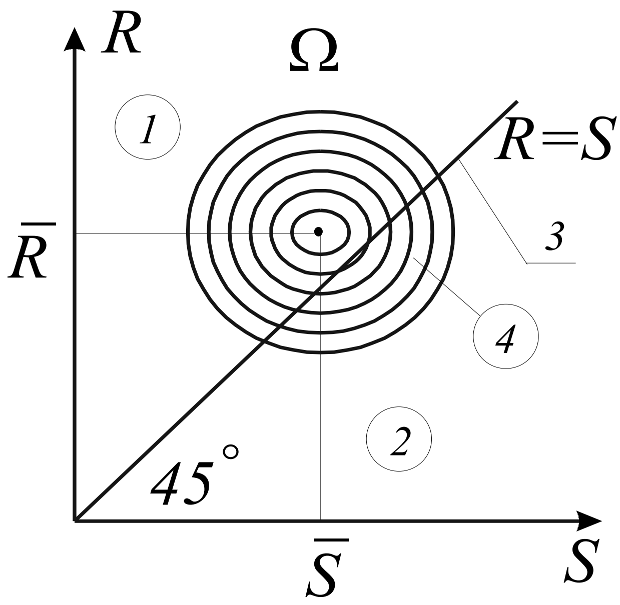
MINISTRY OF EDUCATION AND SCIENCE OF UKRAINE
NATIONAL UNIVERSITY «YURI KONDRATYUK POLTAVA POLYTECHNIC»

SERGII PICHUGIN, LINA KLOCHKO

MODERN PROBLEMS OF RELIABILITY IN CONSTRUCTION

MANUAL

for students of specialty 192 "Construction and civil engineering"



Poltava 2021

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Сучасні проблеми надійності в будівництві: навчальний посібник для студентів спеціальності 192 «Будівництво та цивільна інженерія» / С.Ф. Пічугін, Л.А. Клочко. – Полтава: Національний університет «Полтавська політехніка імені Юрія Кондратюка», 2021. – 147 с. (англ. мовою).

Навчальний посібник призначений студентам 5-го курсу (10 семестр), які навчаються англійською мовою за освітнім рівнем «магістр». Посібник охоплює важливі розділи дисципліни «Сучасні проблеми надійності в будівництві», а саме: основні положення теорії надійності, аварії будівельних об’єктів, ймовірнісний опис навантажень і міцності матеріалів та оцінювання надійності елементів будівельних конструкцій. Розділи посібника містять текстову частину з поясненнями щодо теми розділу, її ймовірнісних аспектів, основних формул і завершується числовими прикладами, які студенти зможуть використовувати при виконанні практичних розрахунків надійності будівельних конструкцій, освоєнні навчального матеріалу, підготовці до модульного тестового контролю та семестрових екзаменів та заліків.

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INTRODUCTION

The discipline "Modern problems of reliability in construction" is taught to students of the 5th year (10th semester), studying at the educational level "master with the specialty 192 "Construction and Civil Engineering" at the Department of Building Structures of the National University “Yuri Kondratyuk Poltava Politechnic".

Such a training course is the only one in Ukraine’s higher educational institutions, it is mainly based on the monograph Guide “Reliability calculation of building structures” (S.F. Pichugin. – Poltava: ASMI Publishing House, 2016), and also in previous editions of the author's monographs devoted to the problem of reliability in construction and methodological recommendations on this problem. Prof. S.F. Pichugin reads this discipline for a long time, using the pedagogical experience and the results of the reliability studies of building structures that have been carried out at the National University “Yuri Kondratyuk Poltava Politechnic" for many years. The course presentation is closely related to the regulatory documents in force in Ukraine, which the author took part in developing, regarding loads and impacts, ensuring reliability and structural safety, designing steel structures, etc.

The manual covers the main sections of the discipline "Modern problems of reliability in construction", namely: basic concept of reliability theory, accidents in construction, probabilistic description of the strength of materials and assessing the reliability of elements of building structures.

The material given in the manual is intended to facilitate the assimilation by students of sections of the discipline referred to practical classes. Each section contains a textual part with explanations regarding the topic of the section, its probabilistic aspects, basic formulas and ends with numerical examples that students can use when performing practical calculations of the reliability of building structures, mastering of educational material, preparation for modular test control and semester exams and tests.

Authors

LECTURE 1. RELIABILITY PROBLEM AND ITS VALUE FOR MODERN CONSTRUCTION

1.1. Introduction
1.2. The problem of reliability in construction
1.3. Poltava Scientific School of Reliability

1.1. Introduction

The discipline "Modern problems of reliability in construction" is taught to students of the 5th year (10th semester), studying at the educational level "Master" in the specialty 0901 "Construction and Civil Engineering" at the Department of Metal, Wood and Plastic Structures of the National University «Yuri Kondratyuk Poltava Polytechnic».

The need to ensure a high level of reliability of buildings and structures is obvious, since their failure, including possible accidents and destruction, leads to large economic losses, and sometimes to disasters with human casualties and dangerous environmental consequences. Reliability as the most important technical and economic parameter of construction projects determines the technical level and competitiveness of construction products.

This lecture course systematically, from a unified standpoint, based on modern probabilistic methods, outlines the methodology for calculating the reliability of building structures and the results of its application for a wide range of construction projects. As far as the authors know, a similar course in English has not been published before in Ukraine.

The course of lectures focuses on practical calculations of reliability. This is motivated by the fact that a thorough and objective reliability assessment necessarily requires the numerical values of reliability indicators, such as the probability of failure and uptime, failure rate and the like. In addition, the regulatory documents of Ukraine in the field of reliability of construction projects, introduced in recent years, regulating the quantitative standards of reliability indicators (probability of failure, safety characteristics), which must be determined during design. The technical literature lacks practical recommendations on this issue, and this course aims to correct this shortcoming.

To simplify the development of the material, the lectures provide some information from probability theory and mathematical statistics. Of the four known components of reliability (reliability, durability, maintainability and preservation), lectures focus on assessing the reliability of structures, to a lesser extent, assessing their durability. This is entirely justified, because for construction projects, failure-free (and associated durability) is the main component, while maintainability and safety are of subordinate importance.

This training course is copyright, the only one in higher educational institutions of Ukraine. This course is taught for a long time, using the pedagogical experience of the KMDiP department and the results of studies of the reliability of building structures, for many years they have been carried out at PoltNTU (for more details, see paragraph 1.3 below). The course is closely related to the normative documents in Ukraine in development, which were attended by university teachers, on loads and impacts, ensuring reliability and structural safety, design of steel structures, etc.

The course of lectures covers all the main sections of the discipline "Modern problems of reliability in construction", namely: a probabilistic description of the loads and strength of materials; reliability assessment of elements of building structures; reliability assessment of compressed-curved elements; reliability of statically indefinite systems. According to the curriculum, in this discipline, in addition to lectures, practical classes are also conducted and settlement and graphic work is carried out, for which a training manual has been developed [3].

1.2. The problem of reliability in construction

The obviousness of the reliability problem. The term ***“reliability”*** in the dictionary of synonyms contains enough analogues: inviolability, responsibility, fidelity, serviceability, stability, accuracy, strength, security, solidity, unwaveringness, invulnerability, fundamentality, durability, firmness, security, reliability, inevitability, inviolability, solidity, proof , capitalism, persuasiveness, break-even, certainty, credibility, incontestability, creditworthiness, testing, faultlessness, win-win. Obviously, we are talking about a widespread understanding of the concept of reliability, but such verbosity to a certain extent indicates the versatility of the general reliability problem.

If we narrow down the consideration of the issue by the construction industry, we can state that the concept of “reliability” and “unreliability” of construction objects coexist quite a long time and widely. It is clear, for example, that the shaky bridge is unreliable, which obviously cannot withstand the cargo that needs to be transported through it; the cable is unreliable, not strong enough for the load that is suspended on it; the floor beam is unreliable, allows too much deflection from the load acting on it, etc. Requirements for reliability and reliability are presented to each technical device, in particular, in construction projects.

The need to ensure a high level of reliability of equipment, buildings and structures is obvious, since their failure, including possible accidents and destruction, leads to large economic losses, and sometimes catastrophes with human casualties and dangerous environmental consequences. A detailed overview of accidents of construction sites is given in the 3rd and 4th lectures of this course. Here, for clarity, we recall only a few examples of major equipment failures and accidents:

• failure of an element worth *5$* caused a disruption in the launch of an American satellite worth $8\cdot10^6$ *$*;

• failure of relay protection in the power system of the northeastern part of the United States caused a power outage in several states and total losses of $500\cdot10^6$ $;

• an accident at a chemical plant built by the American company Union Carbide in the Indian city of Bhopal resulted in the death of 3 thousand people, 100 thousand people remained disabled;

• the accident at the Chernobyl nuclear power plant (Ukraine, 1986), the Holocaust, which became a national tragedy that created socio-economic problems that do not disappear over the years. The economic damage to Ukraine because of the Chernobyl disaster is about $180 million. Because of the Chernobyl explosion, more than 145 thousand square kilometers of the territory of Ukraine, Belarus and the Russian Federation are contaminated with radionuclides. In Ukraine, more than 2,200,000 have the status of victims. 100 thousand people were evacuated;

• destruction as a result of the earthquake at the Fukushima-1 NPP (Japan, 2011), which caused losses of more than $100,000,000,000 affected by radiation of 15 thousand people, 150 thousand people were resettled.

We emphasize that reliability as the most important technical and economic parameter of construction projects determines the technical level and competitiveness of construction products.

In favor of the importance of the reliability problem, the following obvious arguments can be added. To begin with, to increase the efficiency of construction, one of the main areas is the reduction of material consumption of structures. Further, we take into account that one of the possible solutions to this issue, which does not require additional material and financial investments, is to improve the methods of calculation and design of construction objects. Finally, in conclusion, we note that with the current rather high development of the theory of structural analysis, supported by the widespread use of computer technology, the least studied and promising direction in this area is probabilistic calculation, which allows assessing the reliability of buildings, should be quite high.

The severity of the reliability problem. The need to ensure a high level of reliability of buildings is because their failure during operation is accompanied by large economic losses associated with downtime, repairs, material and labor losses. Moreover, in such hazardous industries as, for example, nuclear power plants, gas pipelines, mines, chemical and metallurgical enterprises, transport, etc., as well as in public and residential buildings, insufficient reliability of construction sites can lead to disasters with human casualties and dangerous environmental the consequences. This problem becomes even more acute in a market economy, when the reliability and durability of construction products can decisively affect the results of competition for an order in construction.

The science of reliability in construction, as a branch of the science of reliability of technical systems, studies the patterns of changes in the quality indicators of buildings and develops methods that ensure the reliability and sufficient durability of their work at the lowest cost. Reliability, formulated briefly as “quality deployed in time”, is the most important technical and economic parameter of construction projects for industrial, public and residential purposes, to a large extent determines the technical level and competitiveness of the ability of construction products.

The complexity of the construction sites reliability problem is associated, first, with the complex nature of external loads and influences of a random nature and depend on physical, climatological, technological and other spatio-temporal factors. Certain difficulties arise when calculating reliability on real random influences of structural elements working geometrically and physically nonlinearly. Additional analytical and computational difficulties must be overcome in assessing the reliability of structural systems, especially statically indefinite structures. Existing design standards for building structures, being deterministic in form, provide, except in some cases, failure-free structures during the service life, however, they do not make it possible to quantify the level of reliability of structures laid down. The calculated coefficients of the method of limiting states that are in the design standards need to be refined by statistical methods, in particular, the reliability coefficients for loads, messages and working conditions.

Stochastic reliability problems. All factors on which the failure (failure) of construction projects depend are random. To confirm this, consider an elementary example: a beam is under load. Her failure may occur:

• as a result of structural overload;
• in cases of insufficient strength of the beam material;
• with reduced dimensions of the structure;
• as a result of a gradual change in cross-section and resistance due to corrosion and the like.

Obviously, all of the above factors are not fixed (deterministic), they have a certain scatter and random nature (this is described in detail in the 7th lecture of the course). Because of the failure of the beam under consideration, and in general construction objects, also relate to random events. Therefore, we can only talk about the probability of failures and, accordingly, the probability of failure-free operation of the system (object) for a given period of time. The previously mentioned indicates the need to involve in the solution of problems the reliability of building structures of probability theory and mathematical statistics.

Directions for solving the problem. It must be recognized that in the material world it is impossible to create anything absolutely reliable. To confirm, we give indicative data on the reliability of Soviet rocketry since its heyday *(Table 1.1)*.

Reliability assessment in *Table 1.1* was defined as the ratio of the number of successful missile launches to the total number of launches. As can be seen from the table, the reliability of high-tech space technology, in the development of which multibillion-dollar funds were invested, is not one hundred percent (96.6%), the number of failures is 3,4%.

Table 1.1

Reliability of Soviet rocketry (1970-1989 yy.)

Types of launch vehicles	***Total launches***	***Successful***	***Unsuccessful***	***Reliability assessment, %***
"Proton"	*132*	*122*	*10*	*92,4*
"Soyuz"	*578*	*566*	*12*	*97,9*
"Vostock"	*92*	*91*	*1*	*98,9*
"Molniya"	*193*	*183*	*10*	*94,8*
"Kosmos"	*333*	*319*	*14*	*95,8*
"Cyclone"	*75*	*73*	*2*	*97,3*
"Zenith"	*21*	*21*	*0*	*100*
"Energiya"	*2*	*2*	*0*	*100*
Total	*1426*	*1377*	*49*	***96,6***

Obviously, because creating an absolutely reliable building structure is also fundamentally impossible. Any construction objects, even the most advanced, have ultimate reliability and allow the possibility of failure. By changing the parameters of the object, it is possible to accordingly change the probability of structural failure, bringing it to a sufficiently small value, it is considered permissible. It is this principle that underlies the creation of all technical systems, in particular building structures. The choice of an acceptable probability of failure is a technical and economic task: with an increase in reliability, the cost of the structure increases, but the losses from possible failures decrease. Based on this, you can find the optimal probability of failure, which ensures a minimum of total costs (this is described in detail in the 5th lecture of the course).

Reliability theory studies the laws of changes in the quality indicators of building structures and develops methods that ensure sufficient reliability and durability of their work with minimal cost. Reliability theory is developing in two directions:

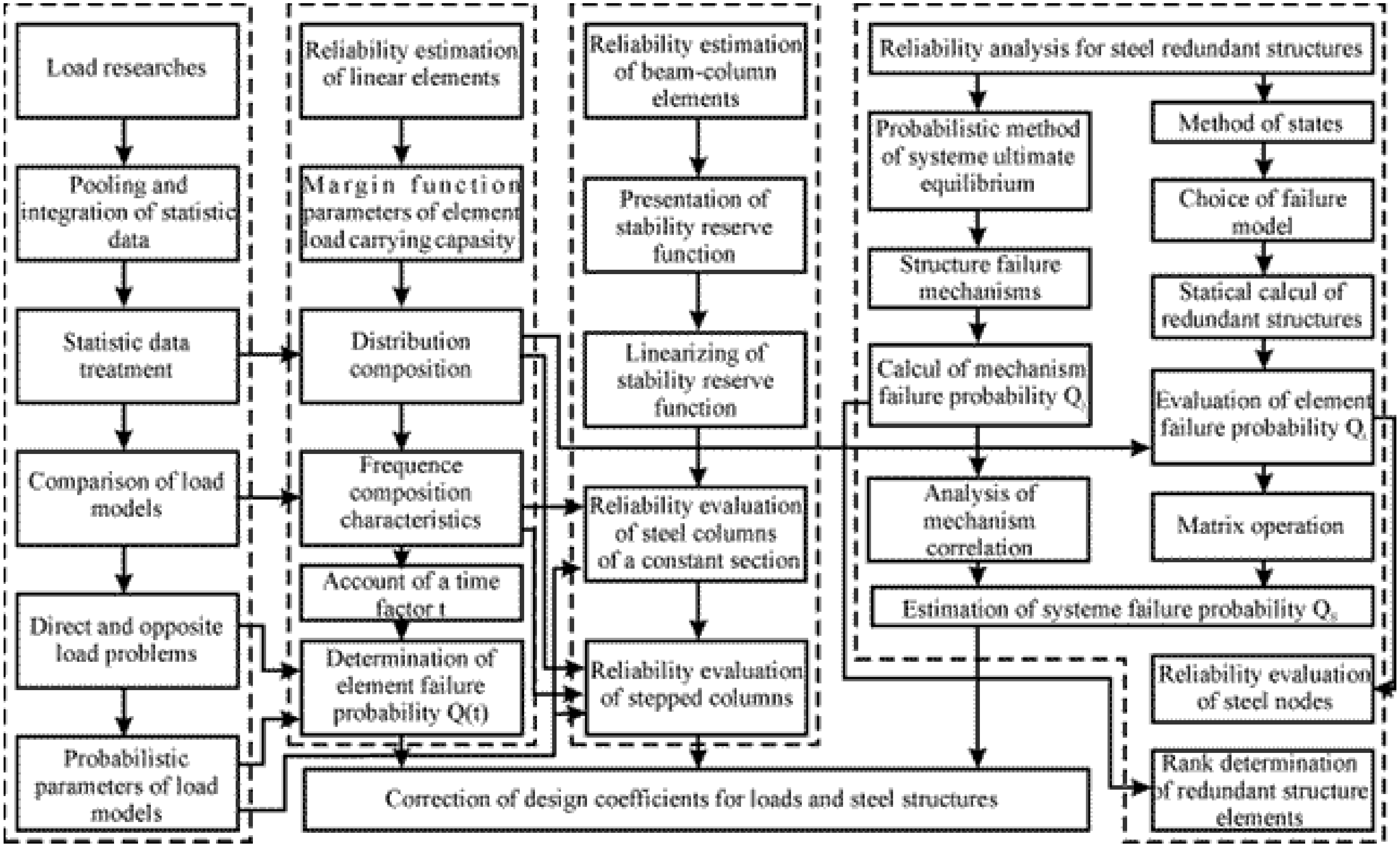

Fig. 1.1. General algorithm of reliability estimation of building structures

• studying the physics of structural failures, developing methods for predicting strength, stability, endurance, wear resistance, and the like;

• development of mathematical methods based on the use of probability theory and mathematical statistics.

We emphasize that the necessary level of reliability of building structures should be provided at all stages of the life cycle of an object, namely:

• at the stage of research and design – the implementation of a complex of calculations and the pleasure of design requirements;

• in the process of manufacturing, transportation and storage of building products – due to the implementation of requirements for the quality of materials, dimensional accuracy and technological modes of manufacturing, transportation and storage;

• during the development of the construction site and the construction of the facility, acceptance of the facility into operation;

• during operation – the competent use of the facility for its intended purpose during the specified service life, maintaining the correct operating mode, constant assessment of the technical condition, and repairs;

• during reconstruction and subsequent use in new conditions;

• at the stage of liquidation of the facility.

The general algorithm for solving the reliability problem of building structures is shown in *Fig. 1.1*.

1.3. Poltava Scientific School of Reliability

A study of the reliability of construction projects has been conducted at PoltNTU for a long time, starting from the 70s of the last century. A powerful scientific school has been formed here on this important issue. Some results of the work of the scientific school: 20 scientific monographs (*Fig. 1.2*), 2 doctoral and 18 master's theses, 25 textbooks and teaching aids, 30 patents for inventions. More than 400 scientific articles published in the USA, Great Britain, Canada, Italy, Norway, Switzerland, Poland and other countries (*Fig. 1.3*).

Probabilistic description of loads. Much attention was paid to this important problem. It is based on experimental researches, pooling and integration load statistic data. The results of regular snow measurements for 15…40 years at 62 Ukrainian meteorological stations have been taken as reference statistical material for ground snow load. The systematic information about the wind velocity measurements done with ten minutes average at 70 Ukrainian and NIS (New Independent States) meteorological stations were used as a initial data. The wind force values were analyzed with the help of a certain quadratic transformation of the wind velocity without the account of its direction. The crane load experimental studies were performed at several metallurgical plants in different shops from 10 to 30 years of service. Vertical and horizontal loads of bridge cranes with rigid or flexible hanged cargo were analyzed.

Fig. 1.2. Monographs on the reliability of building structures

In accordance with obtained results, the following load features were determined. Ground snow and wind loads for Ukraine are of a quasi-stationary origin. Their mathematical expectations and standards have a seasonal trend. At the same time, snow and wind frequent characteristics and normalized ordinate distributions remain constant during the season. The crane load is stationary and ergodic; its density distribution corresponds well to normal law. Taking into account the bimodal characters character of Ukrainian snow density distributions so-called polynomial-exponential law was used. Wind density distribution is well approximated by Veibull's law.

All necessary mean wind and snow probabilistic parameters of Ukrainian districts were introduced in this work. The worked out parameters give possibility to develop the reliability estimation of building structures.

The results of many years of research on the operating time of PoltNTU specialists in the field of loads were included in the collective monograph "Loads and Impacts on Building Structures", which was published in four editions in Kiev and Moscow, became a real bestseller among construction specialists both in the CIS countries and abroad, was actively sold online bookstores in the USA, Canada and other countries.

Fig. 1.3. Foreign publications on the problem of reliability

Reliability studies of building structures under different random loads were performed.

The simplest method of structure reliability estimation is realized in the technique of random values.

- Reliability estimation of reinforced concrete beams with carbon-plastic external strengthening is presented as an example.
- The analysis showed that the dead load of structures is a sum of normal random values describing the weight of every layer. It is established that decreasing coefficient of combination ψ =0,90…0,95 can be applied to a dead load in this case.

General approach to the structure reliability estimation is worked out. This made it possible to perform the analysis of building elements reliability.

- The analysis showed that steel elements have a deficient reliability if they are under 4-wheeled crane load when $X_{M1}/X_{M2} \geq 0{,}8$ i.e. in the case of the one crane load dominance. In the rest cases steel elements reliability determined on the base of general stress state criteria under crane loads is sufficient.
- The reliability of steel structures (beams, trusses) under snow load was estimated by a developed method. These structures are of different mass roofs and snow loads for all Ukrainian districts. The calculation demonstrated the lack of reliability of rafter structures. That justifies the idea of understating of snow loads of the past Code (SNiP) in Ukraine. Besides this fact validates the causes of steel truss failures. It applies to steel structures with lightweight roofs in the southern districts of Ukraine when there is much snow in winter. The increasing of snow design load for 1,5…2 times in the National Load Code (DBN) can solve the reliability problem of steel structures under snow load for our region.
- Steel elements under wind load designed in accordance with the existing code (glass elements, wind protection screens etc) are of sufficient reliability. The obtained results allow decreasing considerably the design wind load for the conditions of structure erection.
- It is recommended to introduce the increasing coefficients of combination ψ = 0,7…0,9 into the steel structure design under snow, wind and crane loads.

A general method for calculating the reliability of building structures, which are the basis of the Poltava scientific school, has been developed, summarized in the monograph “Reliability of steel structures of industrial buildings”, published in Poltava and Moscow and has become popular in the CIS and widely distributed in book and Internet networks of the CIS countries and abroad. Recently, a monograph “Calculation of the reliability of building structures” was published in Poltava, which gives a number of practical calculations of building structures used in the educational process of our university.

Problem of reliability of steel beam-column structures. Time factor, existing loads, random steel strength were taken into account during computation process of these elements. Existing steel columns of industrial buildings in a broad range of parameters were examined. All the columns were designed in accordance with existing Code. Therefore, the general conclusion is as follows: the reliability of steel columns of industrial buildings is sufficient. Besides, the reliability of lower parts of the stepped columns appeared to be much higher then upper ones.

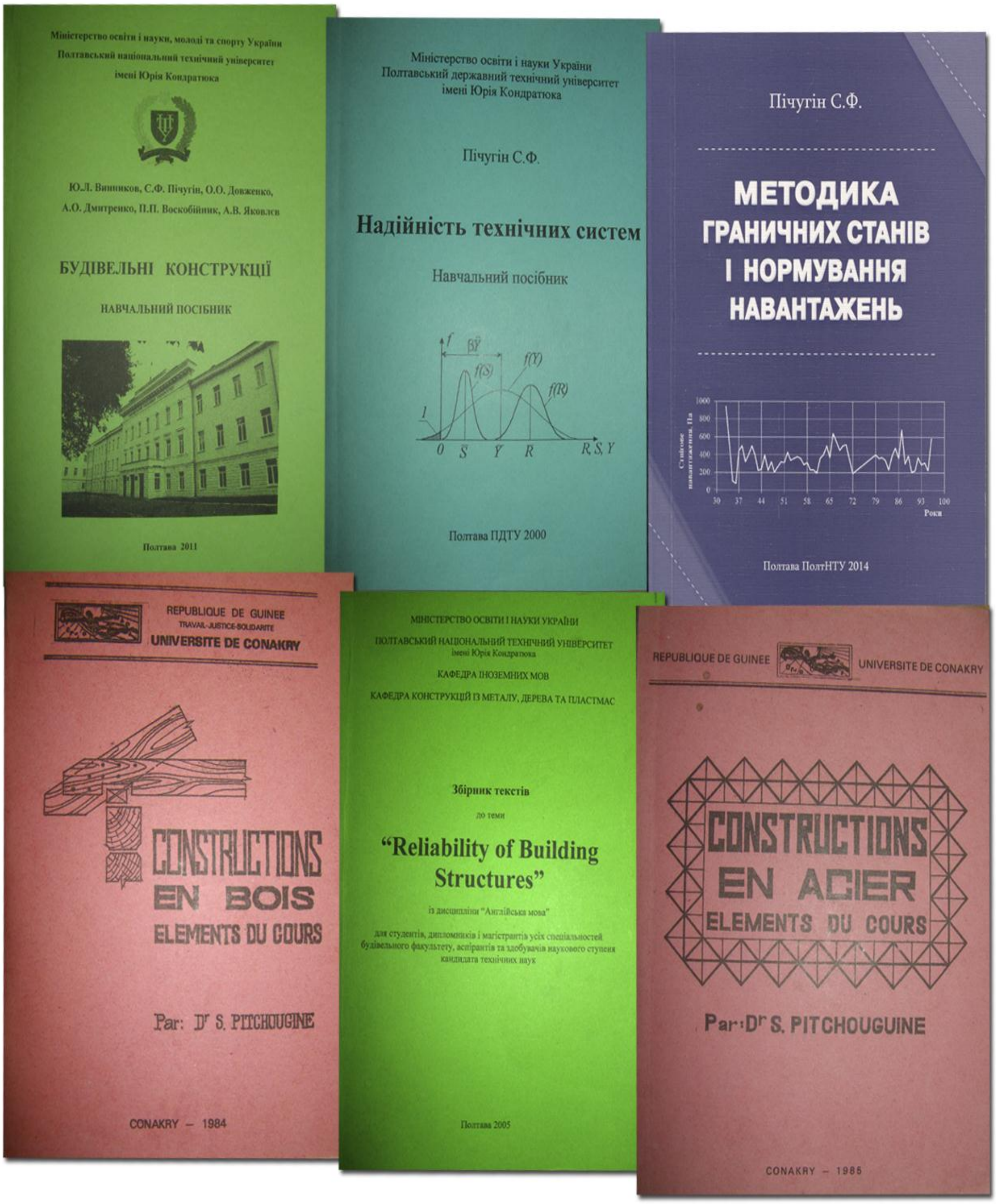

Fig. 1.4. Teaching Aids

Fig. 1.5. Normative documents developed by specialists of PoltNTU

Probabilistic estimation of steel redundant structures. Some beams and simple frames, as well as multistory and multi-span structures of industrial and residential buildings present this type of structures. Redundant structure failures occur after some member failures in the form of transmission to different workable states. These states match different designing schemes with various probabilistic parameters. Thus, the redundant structure failure estimation is a very complicated problem as depends upon the system complexity. The method of states, probabilistic method of ultimate equilibrium and logic and probabilistic method were developed for solving this problem. The estimation of a wide range of industrial redundant structures with different degree of redundancy was obtained on the base of this approach. It gave possibility to evaluate the safety level of redundant structures in comparison with separate members and statically determined structures. This level can be taken into account introducing the additional coefficient of work condition γ_C = 1,1...1,4.

Reliability assessment of objects for various purposes. The distribution of the developed probabilistic approaches to steel-reinforced concrete structures and to building materials and products proved fruitful. The general method for calculating reliability was developed, it was supplemented by taking risks into account in construction, and it was also successfully applied to steel trusses, elasto-plastic beams with spacing (rigid cables), steel beams with cut-outs and perished, beams with a corrugated wall, structural units of the supporting structures of ropeways, underground steel pipelines, high-rise supports of communication systems, frameworks of industrial buildings equipped with bridge cranes, sheet steel structures (silos), when designing a circle of steel structures tions of industrial and civil objects. However, the range of objects and structures for which there is no reliability assessment is wide enough and awaits its researchers.

Fig. 1.6. Laureate Diploma of the State Prize of Ukraine

Testing research results. In addition to the scientific publications noted above, the collective monograph "Highly efficient technologies and integrated structures in industrial and civil engineering" should be highlighted, which sets out the results of many years of research in the field of construction conducted at the Poltava National Technical University named after Yuri Kondratyuk, for which the authors (among them .F. Pichugin and A.V. Semko) were awarded the State Prize of Ukraine in the field of science and technology (2011) – the highest state award for scientific achievements (*Fig. 1.6*).

The study of the reliability of structures turned out to be a very fruitful and promising scientific direction, thanks to which the PoltNTU entered the international scientific arena and open opportunities to participate in representative international conferences, communicate with foreign colleagues, publish in international scientific publications (*Fig. 1.3*): wind engineering – in Italy (Genoa), Czech Republic (Prague), Poland (Warsaw, Lublin, Krakow) snow engineering – in Norway (Trondheim), Switzerland (Davos), Canada (anchor), for crane loads – in Poland (Krinitsa), for reliability of structures – in the UK (London), Malta (La Valletta), Lithuania (Vilnius), Belarus (Brest), Moscow (Russia), Hungary (Miskolc), Azerbaijan (Baku) and other cities.

The reliability issue is systematically included in the educational process of PoltNTU, introduced into the master's work and formed the basis of the training courses in Ukraine “Reliability of technical systems”, “Reliability of buildings and structures”, “Modern problems of reliability in construction”. The necessary teaching aids have been developed for these courses (*Fig. 1.4*).

Implementation in regulatory documents. PoltNTU research results in the field of loads and reliability were highly appreciated and were included in a number of state regulatory documents (*Fig. 1.5*), which guide all builders in Ukraine.

When the preparation for the implementation of the provisions of domestic design standards into the EU standards (Eurocodes) began, PoltNTU experts actively participated in this important work and took part in the development of national annexes to several sections of Eurocodes. Thanks to this, it became possible to use European design standards in Ukraine.

Control questions

1. Justify the obviousness of the reliability problem.
2. What is the severity of the reliability problem?
3. Why is the reliability of construction sites a complex issue?
4. Justify the stochasticity of the reliability problem.
5. What are the main components of the general algorithm for determining the reliability of building structures?
6. What issues does the Poltava School of Reliability of Building Structures solve?

LECTURE 2. BASIC CONCEPTS OF RELIABILITY THEORY

2.1. Definition of reliability
2.2. Reliability components
2.3. Types of states of building structures
2.4. Classification of failures of construction objects
2.5. Indicators of object unfaility

These concepts are defined by the relevant state standards in the field of reliability.

2.1. Definition of reliability

First, we give a general technical definition of reliability, according to DSTU and GOST.

Reliability – *the property of an object to store over time the set values of all parameters that characterize its ability to perform the desired functions in the specified modes and conditions of use, maintenance, storage and transportation.*

It can be briefly stated that ***reliability is a quality that is deployed over time.***

Building standards DBN [2] stipulate that the basic requirement that determines ***the reliability of a construction object*** is its compliance with the purpose and the ability to maintain the required operational qualities during a fixed lifetime. They include:

• guarantee of safety for the health and life of people, property and the environment;

• preservation of the object integrity and its main parts and fulfillment of other requirements, which guarantee the possibility of using the object for its intended purpose and the normal functioning of the technological process, including requirements for the rigidity of building structures and foundations, thermal and sound insulation properties of enclosures, their tightness, acoustic characteristics, etc;

• ensuring that the object can be developed (for example, completion without enhancing existing structures or increasing production for an industrial building) and adapting to changing technical, economic or social conditions;

• creating the necessary level of comfort and convenience for users and operating personnel, including the requirements for indoor climate (air exchange, temperature, humidity, light levels, etc.), as well as accessibility for inspections and repairs, the ability to replace and upgrade individual items, etc;

• limiting the degree of risk by meeting the requirements for fire resistance, safety work of safety devices, reliability of systems and networks of life support, survivability of building structures, etc.

In specific cases, this list may be refined and expanded (for example, by introducing an additional condition to the boundary of the radiation background from the building materials and articles used).

Let us explain some of the terms that appear in the definition of reliability.

Objects are complexes, structures, structural systems and individual structures, as well as their components (assemblies, elements, parts). When determining reliability in construction, the object can be, for example, a multi-element frame of a multi-storey building, as well as a separate structure and even welded or bolted connection of structures, depending on the task.

Parameters – strength, rigidity, durability and other indicators that determine the conformity of structures to their purpose and requirements of standards. These values may change over time.

When it comes to building objects that present a potential hazard, the following concepts are also important:

• *safety* – the property of a building object during its manufacture and operation, as well as in cases of disability, not to create a threat to human life and health, as well as to the environment. Safety example: braking of the passenger elevator with the help of special traps when the cable is broken);

• *survivability* – the property of an object to retain limited performance under effects not provided for in operating conditions; in the presence of some defects and damage, as well as in the failure of some components of the object. An example of survivability is the storage of the load-bearing capacity of the structure after the occurrence of fatigue cracks whose length does not exceed the specified dimensions.

2.2. Reliability components

Reliability is a complex property of a construction site that can include components such as unfaility, durability, maintainability and persistence, or a combination of the sequalities (see flow chart *Fig. 2.1*).

Unfaility is the property of an object to perform the desired functions under certain conditions over a given time interval or operating time. Similarly, the definition of a building code DBN [2] is the ability of an object to continuously maintain a working condition for a specified period of operation. An example would be any building structure that has been continuously operated for decades.

Durability is the property of an object to maintain a working condition (that is, to perform the necessary functions) until the limit state under the conditions of the installed maintenance and repair system. An example of a durable construction site is the central building of our university, built in the early nineteenth century, which is normally operated after the post-war reconstruction and numerous repairs.

Maintainability – is the ability of an object to maintain and restore its working state (ie, the state in which it is capable of performing the required functions) through maintenance and repair. An example of a repairable structure is a false ceiling structure that can be repaired and even replaced without adversely affecting the main structures of the room. At the same time, basic building structures cannot be considered repairable, since their repair and reconstruction is a time-consuming and expensive process, sometimes technically impossible (eg, monolithic reinforced concrete structures).

Persistence is the property of an object to store within the specified parameter values that characterize the ability of the object to perform the required functions during and after storage and (or) transportation. An example of structural safety is the special calculation of trusses for mounting and transport loads.

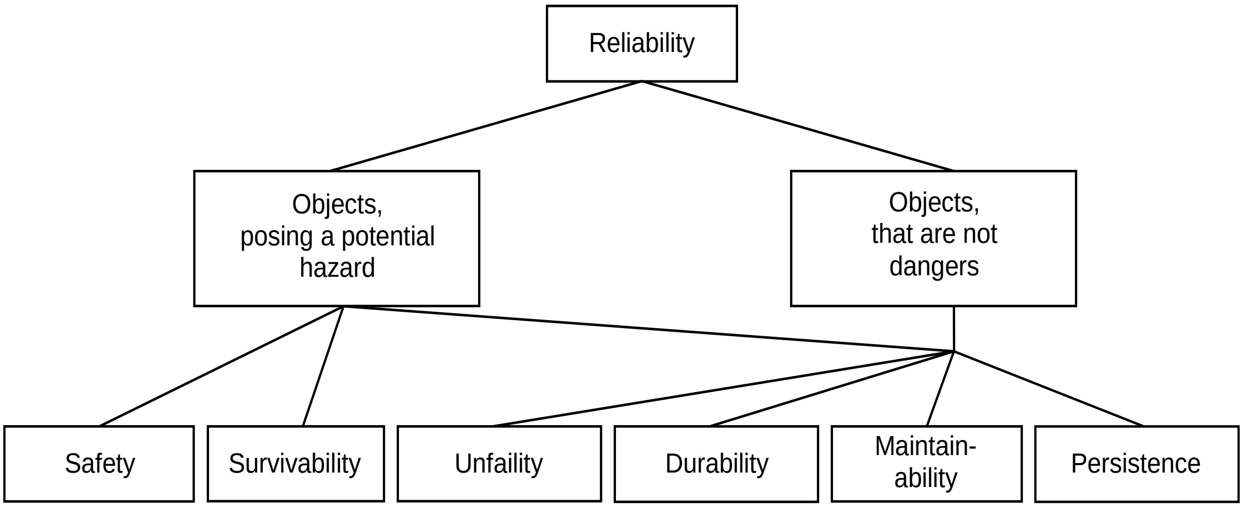

Fig. 2.1. Reliability components

Given the list of components of reliability, we emphasize that building structures must meet the following requirements:

• to accept, without destructions and unacceptable deformations, the effects that occur during their erection and during the established service life;

• have sufficient *working capacity* in normal operation during the entire established service life, namely: their operating parameters (displacement, vibration, etc.) with a given probability should not go beyond the limits set by the normative or design documentation, and their durability should be such, that deterioration of the properties of materials and structures due to decay, corrosion, abrasion and other forms of physical wear does not lead to an unacceptably high probability of failure;

• have sufficient *survivability* with respect to local destruction and the standards of emergency (fires, explosions, vehicle strikes, etc.), excluding the effects of progressive destruction when the total damage is much greater than the initial disturbance that caused them.

Reliability, including durability and survivability, is ensured by the simultaneous fulfillment of the requirements for:

• selection of materials;

• constructive and three-dimensional planning decisions;

• methods of calculation, design and quality control of works in the manufacture of structures and their construction;

• compliance with the rules of technical operation, supervision and care of structures.

2.3. Types of states of building structures.

We must first decipher the concept of an ***object function***, which has two types:

• ***given function***: execution in the object of a process corresponding to its purpose, identification of a given condition or property of the object in accordance with the requirements of normative and (or) design documentation;

• ***required function***: function or function set of an object that is considered to be a necessary condition for the object to be fit for its intended purpose.

Depending on the compliance with the above functions, a classification of the state of construction objects was developed.

Serviceable state: when the design is able to perform *all the given functions.* Building standards DBN determine the ***serviceable state*** in which the object performs all its intended functions, with the facility of regularly undergoing repair and preventive work. Example: a steel beam that has sufficient strength and rigidity, high quality welds and anti-corrosion paint.

Defective state: when a design is unable to perform at least *one of the given functions* (this condition may be due to failure, but may be without it). Malfunctions are classified similarly to faults (see section 2.4 below). Example: The steel beam mentioned above with anti-corrosion paint damage that does not affect the load beam operation.

Working state of an object is characterized by its ability to perform *all the required functions* [1]. Building Code DBN [2] complements this definition with a probabilistic aspect and defines this state as a technical state in which an object performs all its functions while maintaining a tolerable level of risk. Example: a steel beam capable of accommodating the intended load.

Inoperative state of an object that makes it unable to perform at least *one of the required functions*. Example: a steel beam with insufficient strength to perceive the intended load.

Limit state: during which further operation of the structure is unacceptable, difficult or inappropriate. Going to a limit state causes the object to be temporarily or permanently discontinued. Limit states determine the boundary between allowable and non-allowable (non-boundary) states of structures. The transition through the limit state corresponds to one of the types of failure, the limit states themselves being considered permissible. Limit states are divided into two groups.

The first group contains limit states, the transition through which makes the structures unusable for operation, and for which the boundary states may be:

• destruction of any character (viscous, fragile, fatigue);

• loss of form stability;

• loss of position stability;

• transition to a variable system;

• qualitative change of configuration;

• other phenomena where the need for discontinuation of operation (for example, perforation of the tank wall with toxic substances).

The limit states of this group may be related to the violation of the requirements of the integrity preservation or the possibility of the object existence or the non-compliance with the requirements of safety for people and the environment.

The second group contains limit states those impede the normal operation of the construction object or reduce its durability in comparison with the fixed lifetime and for which the limit states are:

• excessive movement or rotation of some points of the structure;

• unacceptable fluctuations (excessive values of amplitude, frequency, speed, acceleration);

• the formation and opening of cracks, their achievement of maximum permissible values of opening or length;

• loss of shape stability in the form of local deformation;

• damage from corrosion or other types of physical deterioration that necessitate a limitation of operation due to the shortened life of the object.

The limit states of this group may be related to the violation of the requirements for the use of the object without restrictions, the violation of the requirements for the level of comfort, convenience of staff, requirements for the appearance of structures, requirements for the possibility of development and modernization of the object in terms of its appointment.

2.4. Classification of construction objects failures

The concept of failure is fundamental in the theory of reliability.

Failure is an event that involves the loss of a building object's ability to perform the required function, which is in violation of a working state [1]. The Building Code DBN [2] interprets failure as an event consisting in the transition through one of the limit states (realization of the limit state), and supplement this definition: "A failure is considered to be the realization of such a condition of the structure, its part or an element, which results in significant economic losses or social losses." Let us clarify here that it is *damages* to consider material or financial losses because of failure, and *losses* are caused by the loss of non-material character (human life and health, cultural and spiritual values, etc.).

Defect is a malfunction of an object while maintaining its performance.

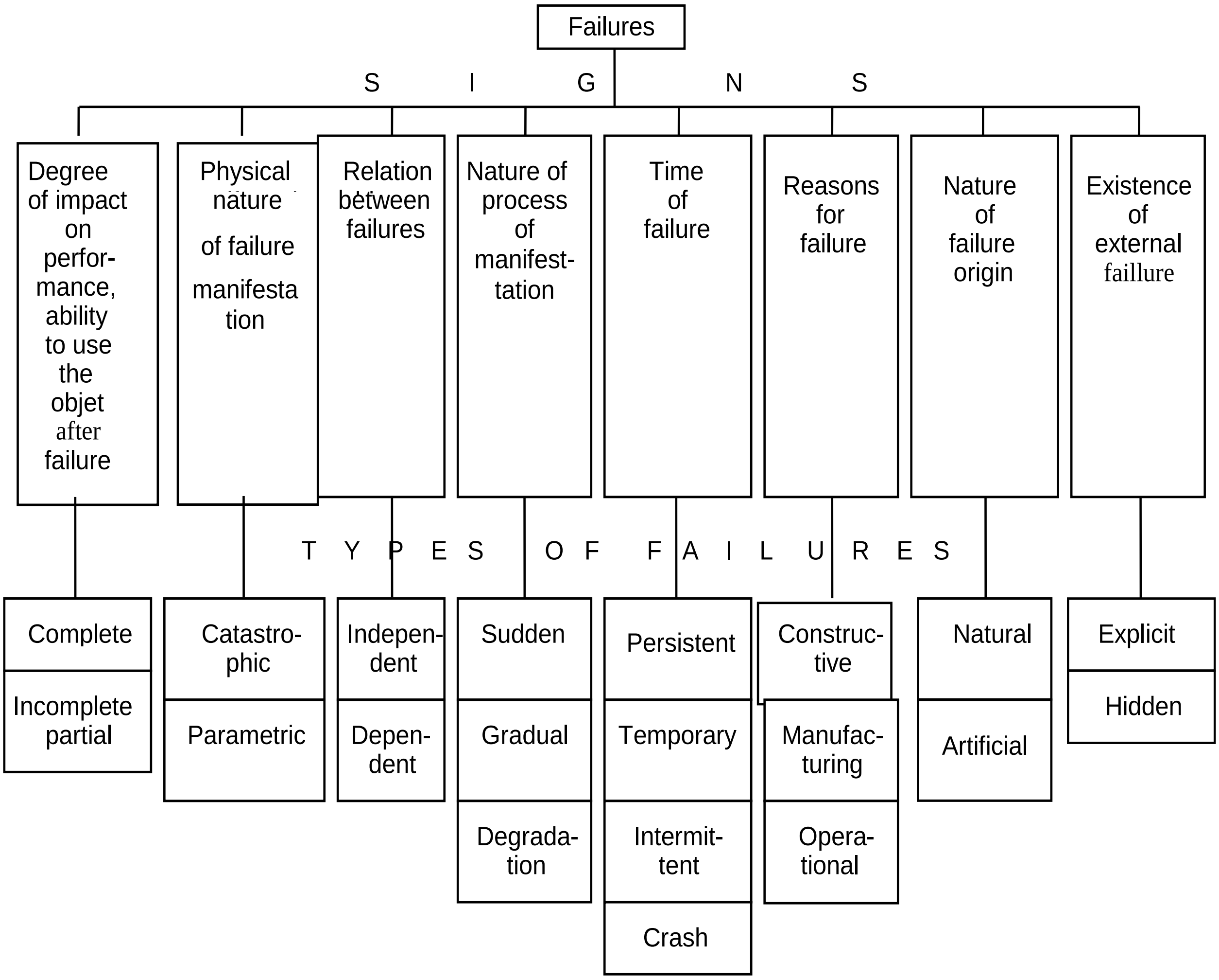

Fig. 2.2. Failures classification of construction objects

The classification of failures of construction objects is shown in *Fig. 2.2*. Let us give some explanations for the flowchart.

Complete failure – the total rejection of the object to perform any of the required functions. Example: the destruction of the frame of a industrial building under load.

Incomplete, partial failure – causes the object to fail to perform some of the required functions. Example: failure of a separate run of the roof of an industrial building while preserving its structure as a whole.

Catastrophic failures – lead to complete disability of structures, for example, to the destruction or fall of the structure. This also refers to failure, the appearance of which immediately causes losses.

Parametric failures – are found in the deterioration of the functioning of the object, for example, in excess of beams deflections of limit values by norms.

Independent failure – not caused directly or indirectly by the failure or malfunction of another object (for example, the destruction of a roof run, which is not related to the failure of other structures).

Dependent failure – caused directly or indirectly by the failure or malfunction of another object (for example, the destruction of a truss and roof elements due to a failure of the column on which the truss rests).

Sudden failure – which cannot be predicted by previous research or technical inspection – is characterized by the abrupt change in one or more object parameters (such as brittle structural failure or loss of stability of compressed structural members).

Gradual failure – occurs because of a gradual change in one or more object parameters (for example, the destruction of a steel element whose cross section has diminished over time as a result of corrosion). This also refers to ***the failure-obstacle***, after which the gradual accumulation of losses (costs) begins.

Degradation failure – is caused by processes of degradation in an object (natural processes of aging, wear, corrosion, fatigue, etc.) in compliance with all established rules and (or) rules for its design, manufacture and operation.

Persistent failures – are of a lasting nature and are eliminated by repairing or replacing a failing element.

Temporary failures – can be arbitrarily disappeared because of eliminating the cause (eg, high temperature or humidity fluctuations, abnormal accelerations and vibrations, etc.).

Intermittent failures – repeatedly occur and disappear, having the same character.

Crash is a self-executing or one-time failure, which is eliminated by a minor intervention by the operator (an example of a failure is a PC stop, which is eliminated by restarting the program).

Constructive failure – caused by imperfection or violation of the established rules and (or) rules of design and construction of the object.

Manufacturing failure – caused by the mismatch of the manufacturing facility to its design or production process standards.

Operational failure – occurs as a result of violation of the established rules and conditions of structures operation.

Explicit failure – manifested visually or by standard methods and means of control and diagnosis during preparation of the object for use or during its intended use (for example, the destruction of truss elements or cracks in concrete beams).

Hidden failure – not detected visually or by standard methods and means of control and diagnostics, but detected during technical inspection or special methods of diagnostics.

2.5. Indicators of object unfaility

Note the two fundamental features of the building structures reliability:

• time dependency, input and use of time parameters;

• failure is a random event, so a probabilistic approach to reliability tasks is required.

Here are some time concepts.

Object (service) life *is the calendar life of the construction object from the beginning or its renewal until the transition to the limit state.*

Operating time *– the duration or amount of work of a construction object*. For an object that works continuously, the operating time is measured in units of calendar time and is the same as its service life. If the object works intermittently, there are distinguished continuous and total operating hours, which are also measured in units of calendar time. If the physical wear of the structure also depends on the operation intensity of the structure, the operating time is expressed through the number of duty cycles. Example: crane beams, which work under load only when a bridge crane (laboratory of the Department of SMWiP) drives them.

Reliability indicator *is a quantification of one or more of the properties that collectively make up a construction object reliability*. Here are the main indicators of the reliability of building structures.

1. The probability of failure-free operation is the probability that during a given operating period the failure of the building structure does not occur.

Denote by: *t* – continuous operation time or total design time. The occurrence of a failure is a random event, so running until the first failure is a random variable. The probability of failure-free operation *P (t)* in the range from *0* to *t* is equal to:

$$P(t)=P(\tau>t). \tag{2.1}$$

Thus, *P (t)* is a function of the operating time *t*, usually it is considered continuous and differentiating. Quite often, *P (t)* is called the ***reliability function of structure.*** As shown in *Fig. 2.3*, it decreases monotonically: $P(0)=1; P(t)\rightarrow 0$ at $t \rightarrow \infty$.

2. The probability of failure Q (t). This is the probability that the design fails once within a specified operating time, and it is operational at the initial time. Because working and inoperative states are opposite, incompatible events, we can write:

$$Q(t)=1-P(t). \tag{2.2}$$

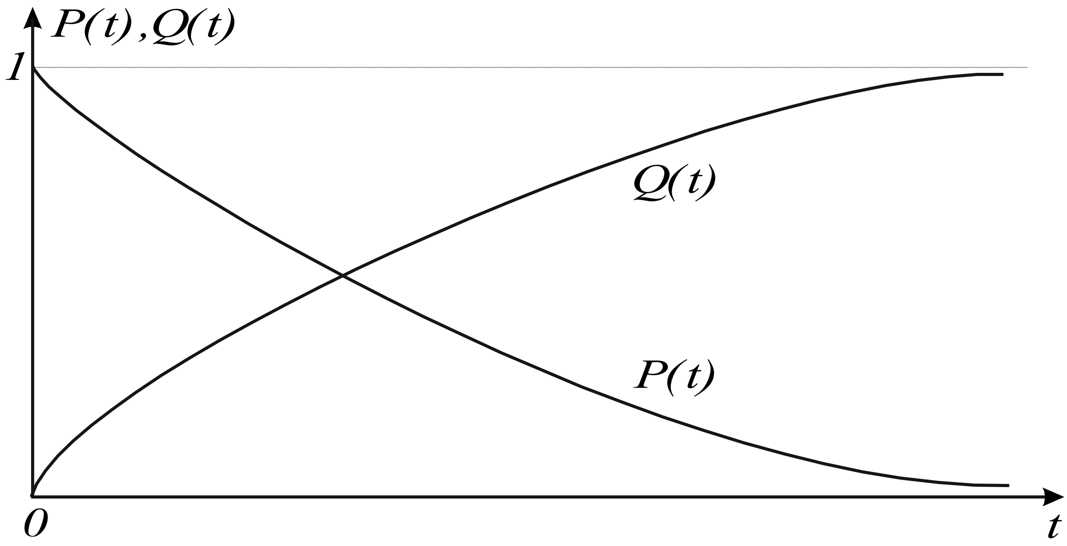

Fig. 2.3. Changing reliability indicators over time:
$P(t)$ – probability of failure-free operation; $Q(t)$– probability of failure

Judging from the *Fig. 2.3*, the nature of the change *Q (t)* is opposite to *P (t)*, so that at $t \to \infty$ $Q(t) = 1$ and for any instant *t* *P(t) + Q(t) = 1*.

Let us relate these concepts to the usual integral distribution function $F(t) = F(\tau < t)$ and the random density distribution function $f(t) = f(\tau < t)$ of failure-free operation time:

$$F(t)=1-P(t);\quad P(t)=1-F(t);\quad Q(t)=F(t);$$

$$f(t) = \frac{dF(t)}{dt} = -\frac{dP(t)}{dt}. \tag{2.3}$$

3. Failure intensity is the conditional density of the probability of a structure failure, which is determined if the failure did not occur before the time taken

$$\lambda(t) = \frac{f(t)}{1 - F(t)} = -\frac{1}{P(t)} \cdot \frac{dP(t)}{dt} = -\frac{p(t)}{P(t)}. \tag{2.4}$$

Failure intensity numerically shows the number of objects of this type that failed per unit of work time. For example, $\lambda = 10^{-3}$ 1/year means that when there are 1000 identical structures in operation, then one structure can be expected to fail in one year.

Interesting is the nature of the relationship $\lambda(t)$ with time t (*Fig. 2.4*), where you can distinguish three stages of the structure operation.

I. *Working out* («burning» of defective elements). This is due to the fact that in a large batch of new designs there are always instances of hidden defects that fail immediately after working start. To remedy this step, pre-loading and checking tests are used (for example, the acceptance test of a new tank by high-pressure).

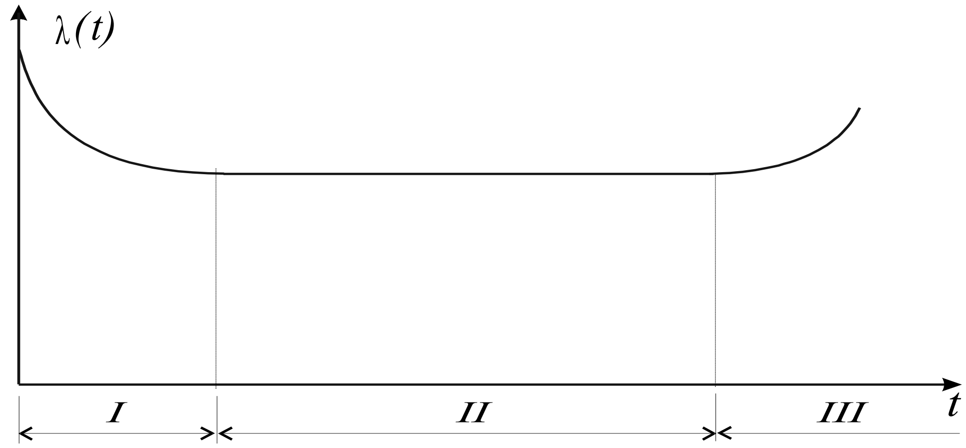

Fig. 2.4. Stages of the structure operation

II. *The period of normal operation* is characterized by a constant failure intensity $\lambda(t)$ = *const*. This is the main stage of operation of each structure, it has the longest duration, it is the «regular mode» of the structure. On the condition $\lambda(t) = \lambda$, solutions of a wide class of reliability problems of building structures are constructed.

III. *Aging period* – when the actuation and aging lead to a deterioration of the structure quality, the risk of its failure increases. The service life of many construction objects ends before the noticeable aging of these sites.

4. ***Operating time*** is the time of trouble-free operation of the structure from the beginning of operation until the first failure occurs.

The average operating time is the mathematical expectation of an object's uptime before the first failure

$$T_1 = \int_0^\infty t \cdot f(t) dt = \int_0^\infty [1 - F(t)] dt = \int_0^\infty P(t) dt \qquad (2.5)$$

The value of T_1 is equal to the area under the curve of the reliability function $P(t)$ (*Fig. 2.5*).

The statistical estimate of the mean average operating time is defined as

$$T_1^* = \frac{1}{N}\sum_{j=1}^{N}\tau_j\,, \qquad (2.6)$$

where N is the number of objects running at $t = 0$; τ_j – the time before the first failure of each object.

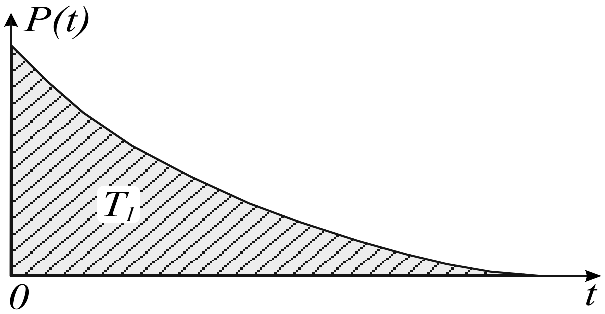

Fig. 2.5. Determining the average time to failure

5. ***Gamma-Percent operating time*** – a time during which an object's failure does not occur with a probability expressed as a percentage and which is defined as the root of the equation

$$P(t_\gamma) = \frac{\gamma}{100}. \qquad (2.7)$$

Thus, t_γ is the quantile of the corresponding distribution. For determining reliability indicators are set quite high γ = 90, 95, 99, 99,5%, etc., which corresponds to the failure probability in the interval [0; t] = 0,10; 0,05; 0,01; 0,005.

Control questions

1. Give a definition of the reliability of the building structure.
2. What are the components of a building object's reliability?
3. In what states can a building structure be located?
4. How are failures of building structures classified?
5. What are the reliability indicators of construction objects?
6. What are the stages of structure during operation?
7. How is the operating time determined?

LECTURE 3. ACCIDENTS FEATURES IN CONSTRUCTION

3.1. The problem of buildings and structures accidents
3.2. Accidents of buildings
3.3. Accidents with significant economic losses and human victims

3.1. The problem of buildings and structures accidents

This problem remains relevant in modern conditions. Cases of buildings collapses with significant economic losses and human victims make it more closely to work on this issue. That is why attention is paid to accounting for accidents in buildings and structures in recent years and to create an appropriate classification based on the collected data.

The phenomenon of risk is a subject of investigation for many both practitioners and theorists. However, only a few of them take these problems and try to formulate the problem within the framework of a procedure. In many publications, the authors deal with the problem of identification of hazards areas and their classification in different groups, among others, due to the source of origin, the impact size, etc. [1]. The number of papers proposing a methodology of quantifying of the risk and elaboration of procedures for the adoption of appropriate actions (so called "an appropriate strategy on risk response") is relatively lower [2].

The Code of Hammurabi is a well-preserved Babylonian code of law of ancient Mesopotamia, dated to about 1754 BC (Middle Chronology). It is one of the oldest deciphered writings of significant length in the world. The sixth Babylonian king, Hammurabi, enacted the code.

A partial copy exists on a 2.25-metre-tall (7.5 ft) stone stele. It consists of 282 laws, with scaled punishments, adjusting "an eye for an eye, a tooth for a tooth" as graded based on social stratification depending on social status and gender, of slave versus free, man versus woman.

229. If a builder build a house for a man and do not make its construction firm, and the house, which he has built collapse and cause the death of the owner of the house, that builder shall be put to death.

230. If it cause the death of a son of the owner of the house, they shall put to death a son of that builder.

231. If it cause the death of a slave of the owner of the house, he shall give the owner of the house a slave of equal value.

232. If it destroy property, he shall restore whatever it destroyed, and because he did not make the house which he built firm and it collapsed, he shall rebuild the house which collapsed from his own property (i.e., at his own expense) [3].

3.2. Accidents of buildings

On February 23, 2015 in Cherniakhovsk, Russia, a wall of the unfinished building which construction had been stopped for considerable term collapsed (*Fig. 3.1*). Because of an incident the 11-year-old teenager had died, during a collapse the plate fell onto the boy. The unfinished building was in a private property, after inspection of the scene the decision on initiation of legal proceedings was made [4].

Fig. 3.1. Collapce of the unfinished building, Cherniakhovsk, Russia, 2015

Fig. 3.2. Collapse of the house which was in process of construction, Mumbai, India, 2013

Accidents cause not only substantial economic losses but can also lead to loss of life. In India 71 people, 25 of them were children, in the result of a collapse of the house, which was in process of construction died. According to the Indian TV channel NDTV, the tragedy happened near the city of Mumbai, on April 6, 2013 (*Fig. 3.2*). Construction of the seven-storied residential building was conducted illegally, in the absence of the necessary documentation confirming works safety on an object. As law enforcement officers explain in spite of the fact that the building has been built illegally, and her construction is not finished, four floors were already populated with residents. Poor construction quality and construction materials became a probable accident cause. The part of the building collapse has entailed all design destruction. Witnesses tell that the seven-storied building fell down [5] in 3-4 seconds as a house of cards.

The specified tendency was confirmed in December, 2012 in the city of Vagkholy where in the result of a collapse of the unfinished house 13 people died, and earlier, in September, the building in the city of Pune, the State of Maharashtra collapsed therefore six people died [6]. On July 29, 2016 in the city of Pune, India, the part of the building that was at a construction stage collapsed. Because of an incident nine workers have died.

Accidents of this kind arise around the world. For example, on March 29, 2013 in the city of Dar es Salaam, Tanzania, the 12-storeyed unfinished building fell down therefore 36 people have died (*Fig. 3.3*). In relation to owners and construction contractors criminal proceedings are conducted during which nine people have already been arrested [5].

Fig. 3.3. Collapce of unfinished building, Dar es Salaam, Tanzania, 2013

On August 15, 2015 in the center of Moscow a new building collapsed. As a collapse result of overlapping between the first and second floors, two persons were injured (*Fig. 3.4*) [7].

In Surgut the new building collapsed on March 6, 2014 *(Fig. 3,5*). Overlapping between the fourth and fifth floors has collapsed. Under blockages rescuers have found three people, two of them were dead. Despite it, media have not given any information on discovery of criminal consequence or about the inquiry commission work at accident scene [8].

Fig. 3.4. Building project of administrative and domestic office of Gazprom Recycling LLC, in which the flooring collapsed

Fig. 3.5. Building collapse,
Surgut, Russia, 2014

On September 1, 2010, in St. Petersburg, on Ligovsky Prospekt, 145, the ceilings of the eight-story building was collapsed. The crash began from the roof, and ended in the very bottom [9].

Next accident took place on January 16, 2013 in Alexandria, Egypt, where an eight-story dwelling house collapsed. The saviors freed 25 bodies from the rubble, 15 wounded were found. As the Alexandria governor said, the construction was carried out without the necessary documents, the municipal authorities did not issue a building company a building license.

The acuteness of the illuminated problem can be clearly imagined if you explore the global information network. Only in one day around the world there were buildings collapses during their construction, as a result, many people were killed and injured.

For example, at 13:00 on September 5, 2016, the Israel police press service announced a building collapse in Tel Aviv (*Fig. 3.6*) that was in the construction phase, leaving two people dead and five more missing. The mobile crane, which drove on the multi-storey car park roof on Ha-Barzel Street in the Tel-Aviv district of Ramatha-Khayal, dropped off the building part that could not bear the weight of a huge machine [10].

Fig. 3.6. Building collapse,
Tel-Aviv, Israel, 2016

On the same day, at 17 o'clock, the press service of RIA Vista News reported on the collapse of an unfinished residential building in the Ural, Russia (*Fig. 3.7*), resulting in serious injury to one of the workers.

Fig. 3.7. The destruction building during construction, Ural, Russia, 2016

The incident took place in the Sverdlovsk region. According to preliminary information, the workers carried out the building structures dismantling of an unfinished dwelling house. During these works, the one floor overlap could not withstand the load and collapsed on the worker. Now, the commission operates on the scene of the accident, which, as a matter of urgency, must provide a legal assessment of the incident [11].

Fig. 3.8. Building collapse, Lutsk, Ukraine, 2012

On June 10, 2012 in Lutsk, Ukraine, a five-story residential building collapsed (*Fig. 3.8*) – the bearing walls from the first to the fifth floor between the first and second entrances were destroyed. Rescuers have rescued from the building 18 people. Because of the tragedy, two people were killed and one was injured.

In areas with difficult climatic conditions, as a rule, there is a high probability of a building accident, therefore, the requirements for the facilities construction in these areas are set more stringent. However, it is difficult to prevent the building of possible floods or other cataclysms.

Fig. 3.9. The fall of a sports arena, Hartford, USA, 1978

Such accidents also include accidents that occurred due to design failures, such as in January 1978 in the city of Harford, Connecticut, USA, due to overloading with snow in the city center, where a hockey match was conducted during the day, overnight collapsed on the night from a height of 30 m a sports arena measuring 92 by 110 m (*Fig. 3.9*). The investigation revealed errors in the calculations of designers [12].

Nowadays we have still enough examples of building collapses, including Ukraine territory. For example, the tragedy in Drobich. It took place on the night of August 27, 28, 2019 and claimed the lives of eight residents of the city (*Fig. 3.10*). SES personnel rescued 7 people, including 5 children. The destruction was because the middle wall of the bearing, on which the other two rests, was unusable. To this is added the fact that the brick, which was built the load-bearing wall - was not compliant with standards [13].

Fig. 3.10. Building collapse,
Drobich, Ukraine, 2019

3.3. Accidents with significant economic losses and human victims

Fig. 3.11. Collapse of the Sampoong shopping center,
Seoul, South Korea, 1995

It is impossible to ignore the most serious accidents over the past two decades, which resulted in dozens, but not hundreds of casualties and thousands of wounded. These include the Sampoong shopping center collapse in Seoul (South Korea). On June 9, 1995, one of South Korea's largest buildings - the largest supermarket in Seoul, Sampoong, collapsed. Under the building ruins, 502 people died, 937 were injured and serious injuries. According to the investigation, it was discovered that a building whose collapse lasted only 20 seconds collapsed due to a number of reasons, the main of which were violations of building codes (*Fig. 3.11*).

One of the reasons for the collapse building was the center's leadership decision to put on the roof three huge industrial air conditioners. In 1993, they were placed on a roof on special pallets, thus adding a load on a so weakened central part of the building (*Fig. 3.12*) [14].

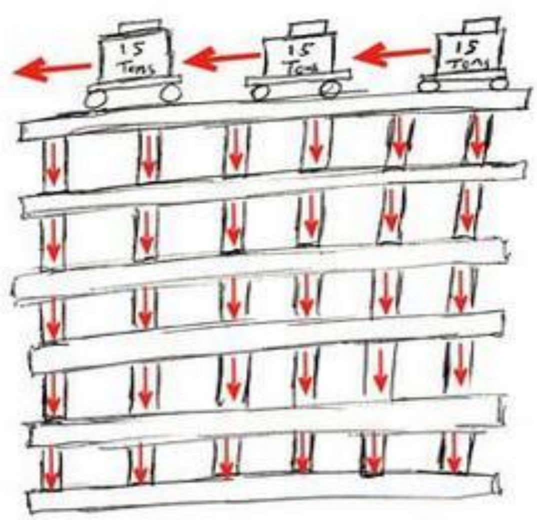

Fig. 3.12. The scheme of placement on the roof of three huge industrial air conditioners

Large-scale accidents in the construction industry cannot be attributed to the destruction of the shopping center "Maxima" in Riga (*Fig. 3.13*), which happened on the evening of November 21, 2013 in the district of Zolitude. Approximately, at 5:45 pm, the roof and the supermarket walls deformed, numerous customers and workers were locked inside. At 18:00, one of the center walls fell and the roof over the ticket offices fell. At noon on November 23, the number of deaths reached 52 people: 51 Latvians and one Armenian citizen. The Latvian police put forward three versions of the disaster: 1) violation of the design; 2) violation of the rules of construction; 3) storage on the roof of building materials [15].

Fig. 3.13. Collapse of the Maxima Shopping Center
Riga, Latvia, 2013

The record number of dead and wounded in the last decade has been recorded in 2013, when in Sawar (Bangladesh) on April 24, a complex containing a bank branch, a shopping center with lots of stores and five sewing factories was destroyed (Fig. 14). On May 9, the death raised to 953 people, more than a thousand people were injured.

Fig. 3.14. Collapse of the complex
Savar, Bangladesh, 2013

On May 3, Interior Ministry experts have established the building collapse reasons: a strong vibration from powerful electric generators. Four giant generators were installed in the building in violation of all the rules, and when they re-started after the electricity was switched off for some time, their vibration, together with the vibration of thousands of machines, led to the collapse of the building [16].

Control questions

1. What is the name of the most ancient laws code concerning construction? What is their essence?
2. Give an example of the most widespread building accidents and name their causes.
3. Compare the 1995 collapse of the Sampoong shopping center to the 2013 collapse of the Savar complex (Bangladesh). What are their similarities? How could the catastrophe have been avoided?

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LECTURE 4. CLASSIFICATION OF BUILDING ACCIDENTS

4.1. General considerations of building accidents
4.2. Building accidents at a stage of construction and acceptance in operation
4.3. Accidents during the buildings reconstruction
4.4. Accidents due to the large age of buildings

4.1. General considerations of building accidents

The problem of buildings and structures accidents remains relevant in modern conditions. Cases of buildings collapses with significant economic losses and human victims make it more closely to work on this issue. That is why, in this lecture, attention is paid to accounting for accidents in buildings and structures in recent years and to create an appropriate classification based on the collected data.

Several publications are devoted to construction accidents, including the monograph B.I. Belyaeva [1], M.M. Laschenko [2], M.M. Sakhnovskii [3], O.M. Shkineva [4] and many others. Quite detailed material of accident statistics was presented by A.V. Perelmuter in the table form of steel structures accidents causes [5]. Also noteworthy are publications by K.I. Yeremin with references to this subject [6, 7].

Speaking about building accidents, firstly is necessary to consider the reasons of structures failure, among which, except cases of excessive casual load, the accidental magnitude of the load capacity (inadequacy of safety ultimate factor), there are many others (unexplored constructions, errors in design, manufacturing and installation, violation operating rules, etc.) [5]. Also during construction often enough do not adhere to those or other norms and requirements for construction work, which in turn can lead to fatal mistakes, at the cost of which can become human life.

The analysis of publications on the estimation of the accident rate of construction objects shows that the statistics of accidents are not perfect. This is not only about the lack of well-documented failures and accidents, but also about the imperfection of the methodology for processing data on them [5]. In our time, despite the great opportunities in the issues of publicity and the press, it is difficult to obtain objective information of accidents, as the construction market is a commercial struggle between construction companies. As a result, many accidents are deliberately silent, and in the future, such incidents are not publicized. In addition, at the current stage of construction development in Ukraine, the question arose about the justification in the state building codes of the people number issue who are constantly on the site and are at risk of accidents.

Speaking about accident statistics, general information is provided annually by the city expert center, which is recognized consultant number 1 in the field of manufacturing in Russia (according to the RA Expert's ratings in 2012). The company «MCE-North», part of the international holding, is provides technical expertise (technical diagnosis) of buildings, structures and equipment [8].

Without claiming the full problem coverage the as a whole, possible to distinguish the ***most widespread cases of buildings and structures accidents***, namely: **errors of engineers in the calculations; negligence of builders during construction an object, improper operation or incorrect reconstruction, cases of which have increased significantly over the past few years.**

Accidents should also be classified according to the class of consequences, in accordance with the National Standard of Ukraine [9]. Taking into account the research carried out, the most widespread buildings accident can be considered objects with a consequences class of CC2, in particular residential buildings with the people number who are constantly in the building, up to 400 people (*Fig. 4.1*).

Consequences Class	**Possible failure consequences characteristics of houses, buildings, constructions, linear objects, engineering and transport infrastructure**					
	Possible danger, number of people			**The possible economic damage amount, m.w.**	**Cultural heritage sites loss, categories of objects**	**Termination of engineering and transport infrastructure facilities, level**
	For the people health and lives who are constantly at the facility site	**For the people health and lives of who are periodically on the facility site**	**For people health and lives who are periodically outside the facility site**			
CC3 Significant consequences	More than 400	More than 1000	More than 50000	More than 150000	national importance	regional, local
CC2 Average consequences	From 50 to 400	From 100 to 1000	From 100 to 50000	From 2000 to 150000	local significance	regional, local
CC1 Minor consequences	up to 50	up to 100	up to 100	up to 200	-	-

Fig. 4.1. Classification according to the class of consequences. The National Standard of Ukraine

However, it should be noted that the most significant accidents, with hundreds of victims and colossal consequences, occurred in the buildings of the consequences of CC3. These include shopping centers, sports arenas, industrial enterprises and entertainment complexes.

Materials on accidents were sought out through the information network, world news and modern scientific publications, which considered these issues in particular. Based on the received material, classification tables were created for the types of accidents that have occurred in recent years. It should be noted that the information is constantly updated depending on the incidents occurring at the given time.

As a result of the study, a table was created showing examples of accidents and structures in recent years with available information on their destruction, the location and incident causes, as well as the number of victims.

4.2. Building accidents at a stage of construction and acceptance in operation

Let us consider such important indicator of building reliability as failure rate λ, which shows quantity of the objects of this type, which have failed at work per unit of, work time. The indicative nature $\lambda(t)$ of relation with *t* on which three stages of operation of buildings and constructions are allocated. That is extra earnings (I), period of normal work (II) and period of aging (III) (*Fig. 4.2*) [10].

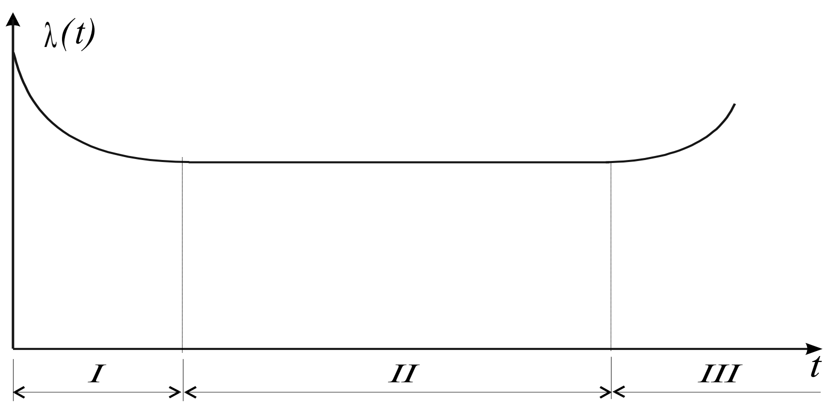

Fig. 4.2. Buildings operation stages

Having analyzed such schedule, it is possible to draw a conclusion that the emergence probability of an accident during facility construction and at its delivery in operation is big enough.

For more detailed research of the matter collection and analysis of information about accidents in construction of constructed objects have been carried out. Materials have been received by means of various information sources, Internet resources and mass media. In the course of work, acquaintance with scientific works according to accidents and their typification has also been carried out. Being guided by the obtained information, accidents of new buildings have been carefully analyzed and systematized in the form of the table. The list of accidents covers time interval 2003 – 2020 and world territorial arena (*Table 4.1*).

Rather often objects of accidents are those buildings, which are being reconstructed, or are in a condition of incomplete construction. For example, on March 5, 2003 in Moscow, Russia, designs of multipurpose shopping center at dismantle of brick diaphragms (poles) which were around staircases have collapsed. Technology violations of works at design dismantle became the main reason for a building collapse. The accompanying reasons were a deviation from design decisions at construction of the dismantled part of the building (insufficient jamming of a horizontal two-leg beam, fastening anchors diameter of a beam to an embedded part of a basic pillow made 12 mm instead of 25 mm, a tail part of this beam hadn't been reliably connected by welding to the main part of the beam, at the same time the imitating (false) seam had been executed [11].

Table 4.1

Accidents of buildings and constructions at the stage of construction

N	Description of accident	City, country	Date	Number of the victims
1	Collapse of the shopping center structures	Moscow, Russia	5.03.2003	-
2	The destruction of an unfinished 13-storeyd building	Shanghai, China	27.06. 2009	1 person
3	The collapse of the unfinished building	Africa	10.07. 2009	14 died, 40 were injured
4	The collapse of the unfinished construction, which was almost ready for delivery	Dubai United Arab Emirates	16.08. 2009	-
5	The destruction of the 4-storeyd building shopping center. The exact cause of failure is unknown	Istanbul, Turkey	27.04. 2009	-
6	Collapse of the hotel that was under construction process	Baku, Azerbaijan	28.04. 2009	3 people
7	Collapse of the4-storeyed building that was under construction. Caused by poor quality of the construction materials.	Xi'an, China	02.10. 2010	10 people were injured
8	Building collapsedduring the construction	Puna, India	September 2012	6 people died

9	Building collapse	Alexandria, Egypt	November 2012	10 people died
10	Unfinished house collapse	Vahholy, India	December 2012	13 people were died
11	The accident when constructing of a residential house. Reasons were the illegal construction, negligence, failure to comply with standards	Taganrog, Russia	13.12. 2012	5 people died, 14 were injured
12	Destruction of 8-storeyed building. The reasons were failure to comply with standards, the illegal construction	Alexandria, Egypt	16.01. 2013	25 people died, 15 were injured
13	The destruction of the 12-storeyed unfinished building	Dar Es Salaam, Tanzania	29.03. 2013	36 people died
14	7-storeyed residential building collapse. Causes are negligence, the illegal construction	Mumbai, India	6.04.2013	71 people died
15	The destruction of the unfinished facility walls, whose construction was suspended. The reason was the frozen construction	Chrnia-khovsk, Russia	23.02. 2015	11-year-old boy died
16	Newly-built floors collapse of a building in the city center	Moscow, Russia	15.08. 2015	2 people were injured
17	Destroyed building during construction	Tel Aviv, Israel	5.09. 2016	2 people were injured
18	The collapse of the ceiling of an unfinished residential building	Ural, Russia	5.09. 2016	1 person was injured
19	The collapse of the unfinished construction	Saransk, Russia	13.11. 2017	2 people died, 3 people were injured
20	The collapse of the unfinished construction of the mall	Sumy, Ukraine	13.02. 2013	-
21	Building collapse during the construction. The collapse of the newly-built floor construction	Kiev, Ukraine	19.11. 2017	-

In Sumy, the two-storey shopping center under construction was collapsed on the territory of the local market.

Information about the collapse of the building construction was received at 0:38, UNIAN reports.

It is noted that the new building was literally in half, and its structures came to complete disrepair. According to the police, there were no signs of extraneous intervention on the site of the building collapse.

During a state of construction, there were about 10 people at the construction site, and the accident occurred when concrete slabs of the second floor were poured. After the incident, the supervisor consulted the list of workers and made sure that there were no casualties: by lucky chance at the time of the collapse workers were drinking tea in the cabins.

It is worth noting that the shopping center began to build in March 2012, and planned to be completed in April 2014. The customer is the enterprise of the Sumy regional consumer union "Central Sum market", general contractors - DP "BS-Visotnik" and LLC "Ukrgazmontazhproekt" [12].

4.3. Accidents during the buildings reconstruction

Speaking about accidents during the buildings and structures reconstruction (*Table 4.2*), it should be noted that the incidents of such accidents have increased significantly over the past few years. The works in many cases are carried out incorrectly, poor quality materials are used, and negligence during reconstruction is also excluded.

On September 1, 2010, in St. Petersburg, on Ligovsky Prospekt, 145, the ceilings of the eight-story building was collapsed. The crash began from the roof, and ended in the very bottom [13].

Quite often, the accident objects are those buildings that are under reconstruction. For example, on March 5, 2003 in Moscow, Russia, the construction of a multifunctional shopping center collapsed when dismantling brick diaphragms (pylons) that were located around staircase cells [14].

4.4. Accidents due to the large age of buildings

During researching and analyzing the buildings and structures accidents, it is not impossible to avoid accidents that occurred due to the facility large age, or as a result of failure to perform timely repairs in buildings that need it.

A good example of inappropriate care for buildings can be the historical significance construction – the Cadet Corps, Poltava, Ukraine (*Fig. 4.3*).

This building was built in 1840, is currently inactive and is in a dilapidated state. The building reconstruction is not carried out; therefore the building is in a miserable condition, which in the future may lead to another accident in the construction industry. Moreover, such cases are not isolated, and unfortunately, are quite common in the Ukraine territory.

Table 4.2

Accidents during the buildings reconstruction

Description of accident	**Reasons of accident**	**City, country / date**	**Number of the victims**
A four-storey office building that was under reconstruction. Slab floor was collapsed.	Unauthorized planning of premises on the first and second floors.	Krasnoyarsk, Russia, June 15, 2009	3
Five-storey house of dormitory. Collapse of two entrances.	Deal of deterioration	Astrakhan, Russia July 22, 2009	2
Four-story building, during the reconstruction period. The three floors are destroyed.	Deal of overlappings deterioration	Prague, Czech Republic 10.02.2009	-
A dwelling house built more than half a century ago.	Repairs	Hong Kong, China January 29, 2010	5
The building adjoining the hotel "Kharkiv".	During the reconstruction period	Kharkov, Ukraine March 16, 2010	-
Eight-story house. Overlapping was collapsed.	During the reconstruction period	St. Petersburg, Russia, September 01, 2010	A few
Three-story house.	Repair work, which resulted in violations of bearing structures.	Dumyat, Egypt 01.02.2012	35
Five-story house.	Illegal construction in violation of safety rules.	Sian, China June 26, 2011	7

Fig. 4.3. The appearance of the Cadet Corps in Poltava at present

We give additional examples of this type accident. Namely, in January 2010 in Tbilisi, Georgia, there were just two accidents. At first, the carrier wall of a residential three-story building collapsed, a day later – carrying two-story structures. In both cases, the buildings were in a emergency state. It should be noted that emergency measures were not carried out before the collapse. Fortunately, there are no victims [11].

On October 26, 2010, a residential building was partially destroyed in the Kirov region, Sovetsky, Russia. The load-bearing wall collapsed, followed by stairs marches and inter-floor overlays. The pre-war building needed major repairs; the means for repairs were allocated slowly. People were not affected by the accident.

In addition, on the basis of the processed material a table was created describing the accidents and structures requiring repair work (*Table 4.3*).

The problem of studying accidents in buildings and structures is incomplete information about certain accidents. In the finding process in the various sources of necessary information, it has to be repeatedly encountered with the illuminated problem incompleteness.

Table 4.3

Accidents of buildings and structures requiring repair work

Description of accident	City, country	The date	Number of victims
Collapsed bearing wall of a residential three-story building. The building was in an emergency. No collision preventive measures were taken.	Tbilisi, Georgia	January, 2010	-
The wreck of the emergency wings is destroyed. The building was declared emergency. The inhabitants were evicted.	Odessa, Ukraine	March 21, 2010	
Partially demolished dwelling house. The load-bearing brick wall collapsed, followed by stairs and blanking. The building needed major repairs.	Sovetsk, Russia	November 26, 2010	-
Collapse of a three-story building that was in an emergency. The destruction occurred due to repairs that were carried out in the neighborhood.	Barletta, Italy	October 3, 2011	4
The seven-story building, which was in an emergency, was destroyed.	Luxor, Egypt	February 11, 2011	15 died, 20 were injured
Collapsed unoccupied emergency facility located near low-rise buildings.	Alexandria, Egypt	14 July, 2012	15
A five-story building collapsed. The cause of the accident was the cracking of the old building, formed as a result of heavy rains.	Beirut, Lebanon	15 January, 2012	27 died, 12 were injured

Studying the accidents statistics and the characteristics in construction, a number of eminent researchers have been trying for decades to create a unified, well-founded classification of this type. But the goal set before the scientists is so unrestricted in the study, as in the implementation methods.

For the most part, accident statistics are currently being conducted in the most obvious way, namely, the accident information collection in tabular form, with the indicated reasons, the injuries number and the incident date. If the accidents collection covers the international territory, the general table is supplemented with information about the country where the incident occurred. Such a collecting information method can be defined as a general one. It allows you to summarize all the processed data from various resources and sources, on the basis of which the accident rate charts can be constructed depending on the selected indicators: crashes by type of building, destroyed structures, places (countries or cities) or number of victims.

The next part of the information statistical processing is a more detailed resulting general table breakdown by the objects type that have been destroyed. For example, the accidents types can be divided into three components: the buildings and structures destruction at the construction stage, in the objects reconstruction and the accident due to the large building age. Classification is precisely on these grounds due to the high repeatability level during the study of this issue, which implies that the probability of such an accident occurrence is highest.

On the research-conducted basis, graphs and charts are created, which reflect the results obtained, which are already making final conclusions.

An example of generalized data processing is the annual accidents statistics, created by the Russian company «City Center of Expertise». The peculiarity of this company’s work is its transparency and results publicity. The statistics provided over the past few years are freely available on the Internet, with the components of which can be read by anyone. At the same time, official statistics, which is conducted by state authorities, do not have access to ordinary citizens. On this basis, there is a need to address the work transparency issue of the Commissions investigating accidents in buildings and structures.

The possibility of providing public information can be a significant step in addressing accidents that occurred during the construction phase, as the publicity of incidents and work results carried out by the special commission will be a major impetus for the elimination of accidents certain types.

In addition, the data statistical processing on accidents building objects makes it pay attention to the high-rise buildings problem, which are decommissioned, but not later dismantled. The authorities often do not pay attention to their accident rate and the destruction highest probability. The result of long-term dismantling, and in most cases, its complete absence, can become human life.

If for some time, the accident was considered as a probabilistic event, which has no regularities and whose results cannot be predicted, then at present scientists have made a tangible breakthrough in this field of knowledge. With the introduction of such concepts as economic and non-economic consequences, the development and implementation of possible losses calculations, depending on the design failure.

The approach to the accident description can be considered with its probability. That is, an accident may be probable, impossible or accidental (*Fig. 4.4*).

These are three fundamental features that make it possible to differentiate the event and its progressiveness. That is, there is a certain antinomy of concepts: chaos, irregular series of events - in this case, the objects construction and their exploitation - acquires a regular order only when we narrow the range of statistical selection. Thus, moving from macro to micro-research, we create more complex statistics, which includes a clear understanding of the probability, impossibility or chance of an event.

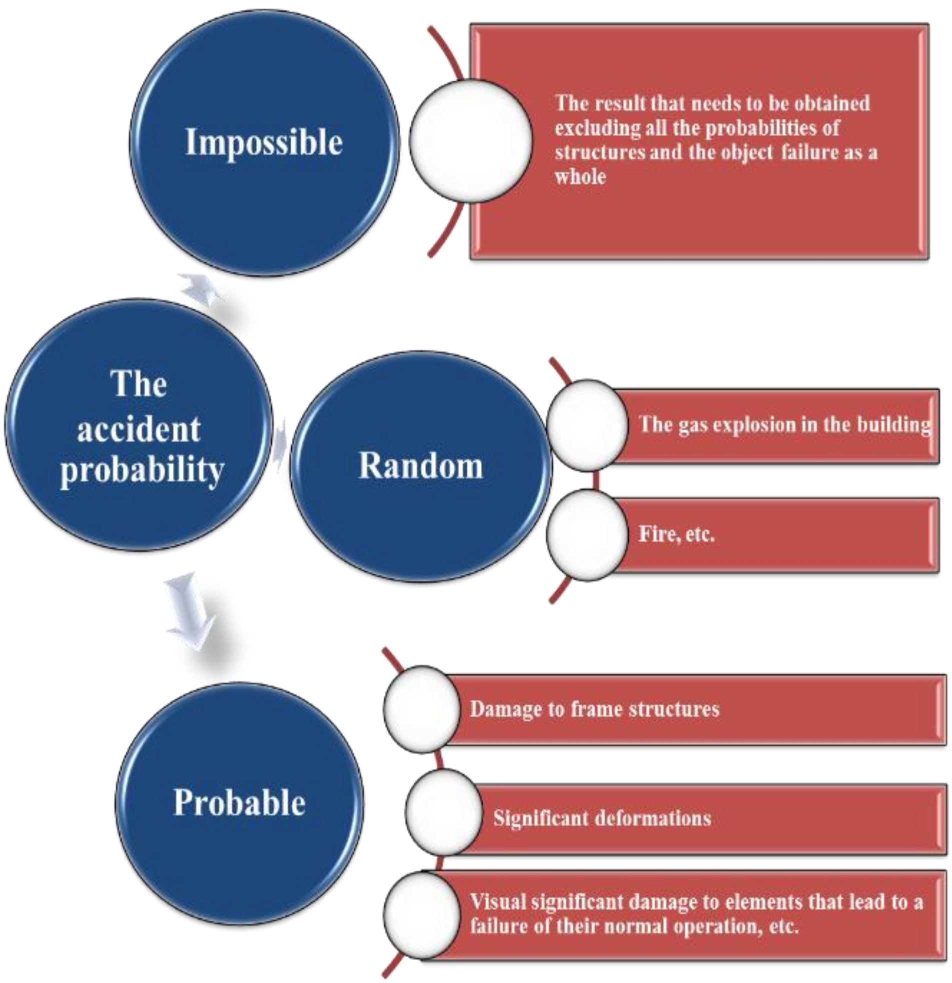

Fig. 4.4. Classification of accidents on the probability of their occurrence

Here are examples of situations in which the accident was probable. In this case, this is an accident near the city of Mumbai, April 6, 2016 [15]. The collapse of a seven-storey residential building provoked a number of reasons, such as a violation of building codes, negligence in the construction, illegal construction works. The probability of emergence of an emergency situation was the maximum in this case.

Accidental accidents include the explosion of gas in a residential building in Brussels, which took place on March 18, 2017, resulting in the loss of one person [16]. One building collapsed completely, from the other only the facade remained. Or the fire that occurred on February 21, 2015, in the OAU, where the tallest Fakel building fired [17]. No one was hurt.

The result of the accidents analysis that occurred in construction should be the impossibility of an accident. A striking example of working out the past years' experience, the implementation of necessary improvements and the various accidents types prevention is the modern complex «Federation», which consists of two skyscrapers of 324 meters high (*Fig. 4.5*) [18].

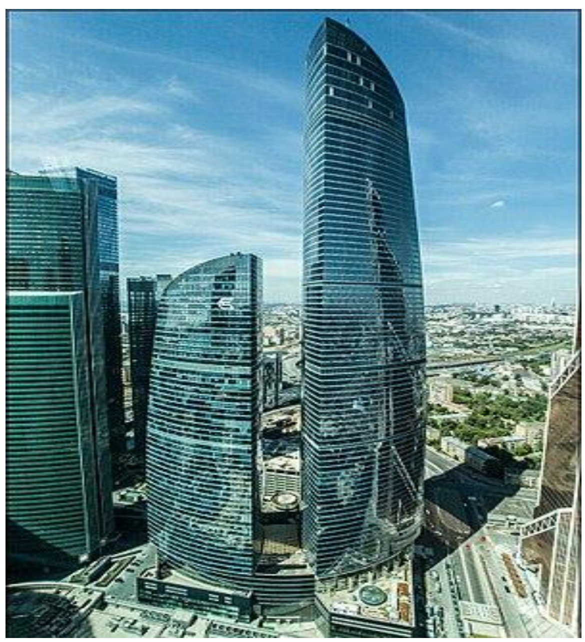

Fig. 4.5. Modern complex «Federation», Moscow, Russia

The building is equipped with cutting-edge technology, and is the highest in Europe and the strongest in the world. The hard frame "Federation" is designed in such a way that the output from the work of one element does not affect the normal work of the entire design. The experience of past years with the problems of fire safety and explosive environment introduced the latest high-tech designs. This facility serves as a vivid example of effective work on building mistakes.

Control questions

1. Name the most common accidents causes in buildings and structures. Give examples of each collapse type.
2. What does the buildings operation stages diagram look like? Describe each of the steps.
3. What are the most common building accidents causes at the construction stage? Give examples.
4. What are the most common causes of building failures during the reconstruction stage? Give examples.
5. What is accidents classification on the occurrence probability?

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LECTURE 5. PROBABILISTIC DESCRIPTION OF RANDOM VARIABLES. NORMAL DISTRIBUTION

5.1. Probabilistic description of random variables
5.2. Normal distribution law
5.3. Strength rating of rolled steel
5.4. Numerical example: determination of the characteristic and design steel resistances

5.1. Probabilistic description of random variables

5.1.1. Key definitions. ***A random value (RV)*** is a variable, because 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;
- structural loads.

Designations: $\tilde{x}$ – a random value; x – 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

$$P^*(A) = \frac{m}{n}. \tag{5.1}$$

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

$$P^*(A) \to P(A),\ n \to \infty. \tag{5.2}$$

For example: tossing a coin when *P(A) = P(B) = 0,5*, if $n \to \infty$, where *A* is the loss of the “eagle”, *B* is the loss of the “tails”.

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

$$F(x) = P(\tilde{x} < x). \tag{5.3}$$

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. 5.1*.

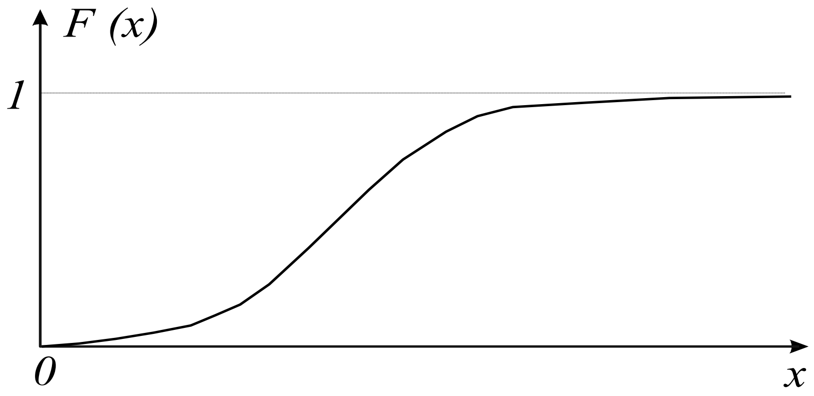


Fig. 5.1. Random distribution function

Derivative function *F(x)*

$$f(x) = \frac{dF(x)}{dx} \tag{5.4}$$

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. 5.2)*.

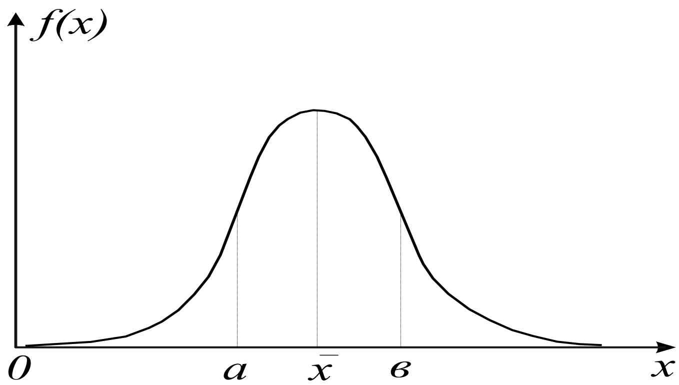


Fig. 5.2. Random distribution curve

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* $F(x)$:

$$F(x=a)=F(\tilde{x}<[x=a])=\int_{-\infty}^{a} f(x)dx; \qquad (5.5)$$

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

$$F(a<\tilde{x}<b)=\int_{a}^{b} f(x)dx; \qquad (5.6)$$

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

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

5.1.3. Numerical distribution characteristics of random variables.

Mathematical expectation

$$\bar{x}=\int_{-\infty}^{\infty} xf(x)dx. \qquad (5.7)$$

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. 5.3*).

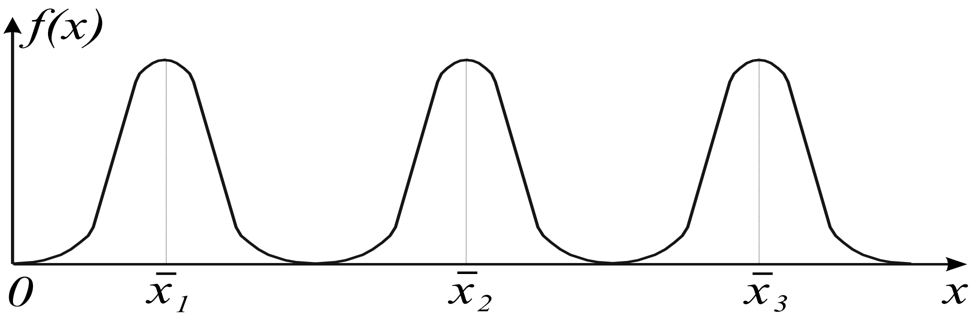


Fig. 5.3. Distribution curves with different mathematical expectation: $(\bar{x}_3>\bar{x}_2>\bar{x}_1)$

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

$$\hat{x}=\int_{-\infty}^{\infty}(x-\bar{x})^2 f(x)dx\cdot \qquad (5.8)$$

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

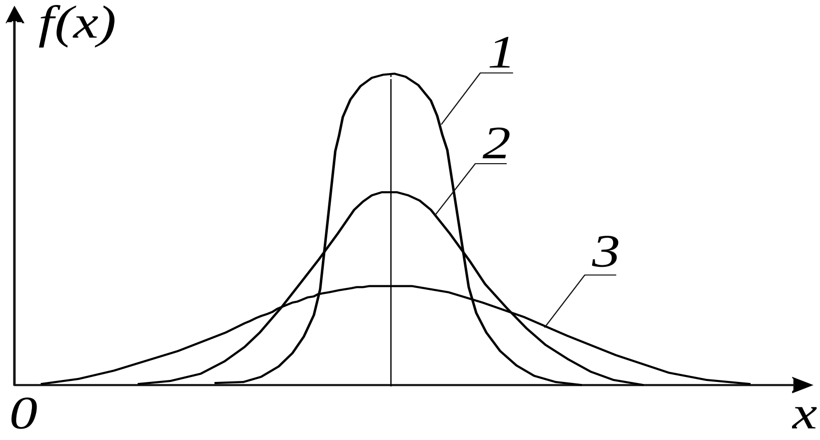


Fig. 5.4. Distribution curves with different standards: $\left(\hat{x}_1 < \hat{x}_2 < \hat{x}_3\right)$.

The standard deviation (standard) $\hat{x}$ and the coefficient of variation V characterize *the scatter of the values* of a random variable (*Fig. 5.4*):

$$\hat{x} = \sqrt{\widehat{x}}; \quad V = \frac{\hat{x}}{\overline{x}}. \tag{5.9}$$

The asymmetry coefficient A_x determines *the slanting* distribution of a random variable (*Fig. 5.5, a*):

$$A_x = \frac{\mu_3}{\hat{x}^3}, \tag{5.10}$$

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

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

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

$$E_x = \frac{\mu_4}{\hat{x}^4} - 3. \tag{5.11}$$

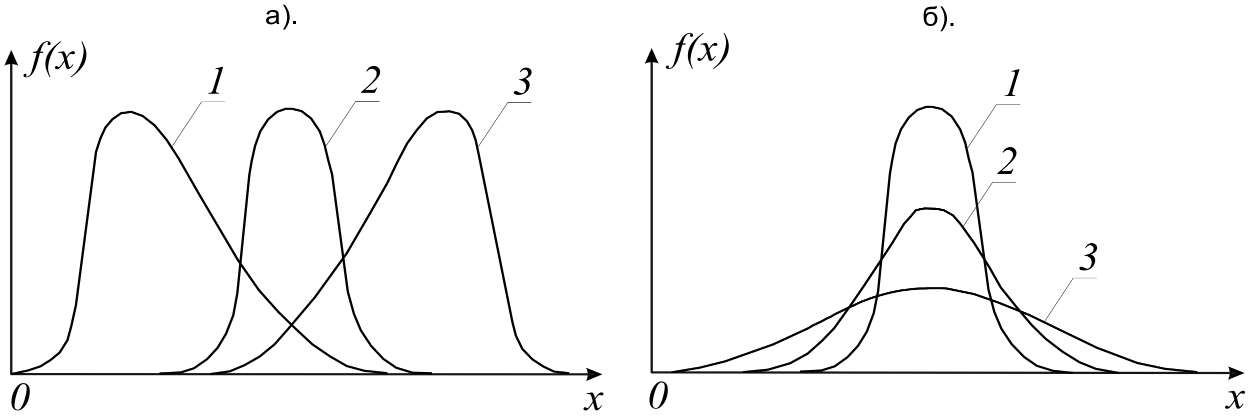


Рис. 5.5. Distribution curve options:

а) with different asymmetries: $A_1 > 0$, $A_2 \approx 0$, $A_3 < 0$;

б) with different kurtosis: $E_1 > 0$, $E_2 \approx 0$, $E_3 < 0$

5.2. Normal distribution law

An important aspect of probabilistic calculation methods is the reasonable choice of the distribution laws of random variables and the ordinates of random processes. The most common in the theory and probability calculations practice is ***the normal law***, which is also called the Gauss law. This symmetric distribution with infinite limits *(Fig. 5.6)* has a density

$$f(x) = \frac{1}{\hat{X}\sqrt{2\pi}} \exp\left[-\frac{(X-\bar{X})^2}{2\hat{X}^2}\right], \tag{5.12}$$

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

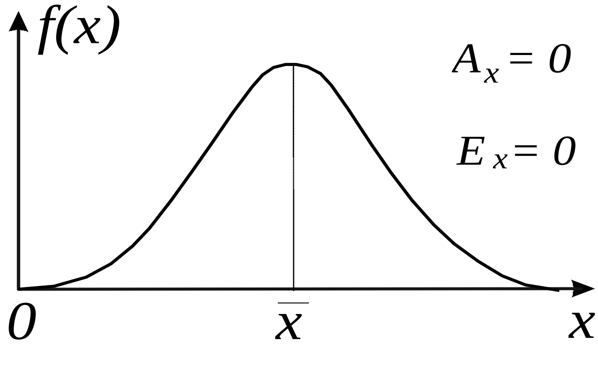


Fig. 5.6. Normal distribution of a random variable

The asymmetry and kurtosis of this distribution are zero.

If instead of argument X the normalized deviation from the center is taken into account $\gamma = (X - \overline{X}) / \hat{X}$, then the normalized density of the normal law of the following form is determined:

$$f(\gamma) = (\sqrt{2\pi})^{-1} \exp(-0.5\gamma^2). \qquad (5.13)$$

With this in mind, the integral function of the normal distribution will je

$$F(X) = \left(\sqrt{2\pi}\right)^{-1} \int_{-\infty}^{X} \exp\left(-0{,}5\gamma^2\right) d\gamma = \Phi(\gamma), \qquad (5.14)$$

where $\Phi(\gamma)$ – the Laplace function is tabulated along with formula (5.12) in many statistical tables *(Tables D.1, D.2)*.

The prevalence of the normal law in reliability problems is related to its relative analytical simplicity, since it depends on two parameters, the presence of ready-made tables, close correspondence to the strength distributions of materials and some loads, the asymptotic desire for a normal distribution of the sum of several random variables with different distribution laws.

5.3. Strength rating of rolled steel

The main strength characteristic of a structural material, including rolled steel, is the ***characteristic*** (or ***normative***) ***resistance***. Probability (probability of deviations to the smaller side) of the standard resistance of steel should be $P = 0{,}95$. This value of security is estimated based on the Gaussian distribution as

$$P = 0{,}5 + \Phi(\beta), \qquad (5.15)$$

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

The corresponding value of the characteristic steel resistance

$$R_{yn} = \overline{\sigma}_y \left(1 - 1{,}64 V_m\right) = \overline{\sigma}_y - 1{,}64 \hat{\sigma}_y. \qquad (5.16)$$

The design steel resistance is the minimum possible steel resistance, which can be determined based on the statistical distribution of the yield strength, for example, on the basis of the "three sigma" rule. At the same time, the security of the design resistance should be at least 0,998.

$$R_{yn} = \overline{\sigma}_y - 3\hat{\sigma}_y. \qquad (5.17)$$

5.4. Numerical example: determination of the characteristic and design steel resistances

Direction. *Determine the characteristic and design tensile steel resistance by experimental statistical data.*

5.4.1. Initial data: selection of experimental data for tensile testing of steel samples in a volume of $n=50$ (*Table 5.1*).

Table 5.1

The experimental values of the yield steel strength, MPa

253,2	*270,8*	*360,0*	*371,0*	*406,6*
277,7	*280,9*	*306,9*	*249,1*	*345,7*
311,1	*322,4*	*324,7*	*376,3*	*304,4*
328,8	*275,7*	*301,1*	*298,4*	*323,9*
332,3	*287,3*	*361,1*	*346,6*	*339,5*
324,1	*340,4*	*339,3*	*250,1*	*406,7*
346,2	*266,8*	*330,2*	*373,5*	*258,5*
363,9	*288,8*	*391,3*	*337,2*	*302,5*
272,6	*257,8*	*301,1*	*202,7*	*314,0*
325,3	*413,6*	*307,4*	*326,2*	*328,5*

Table 5.2

Calculation of sample numerical characteristics

Interval bounda-ries		x_i	n_i	f_i^*	u_i	$n_i u_i$	$n_i u_i^2$	$n_i u_i^3$	$n_i u_i^4$	$n_i(u_i+1)^4$
		1	***2***	***3***	***4***	***5***	***6***	***7***	***8***	***9***
202	*222*	*212*	*1*	*0,02*	*-6*	*-6*	*36*	*-216*	*1296*	*625*
222	*242*	*232*	*0*	*0*	*-5*	*0*	*0*	*0*	*0*	*0*
242	*262*	*252*	*5*	*0,1*	*-4*	*-20*	*80*	*-320*	*1280*	*405*
262	*282*	*272*	*6*	*0,12*	*-3*	*-18*	*54*	*-162*	*486*	*96*
282	*302*	*292*	*5*	*0,1*	*-2*	*-10*	*20*	*-40*	*80*	*5*
302	*322*	*312*	*6*	*0,12*	*-1*	*-6*	*6*	*-6*	*6*	*0*
322	*342*	*332*	*14*	*0,28*	*0*	*0*	*0*	*0*	*0*	*14*
342	*362*	*352*	*5*	*0,1*	*1*	*5*	*5*	*5*	*5*	*80*
362	*382*	*372*	*4*	*0,08*	*2*	*8*	*16*	*32*	*64*	*324*
382	*402*	*392*	*1*	*0,02*	*3*	*3*	*9*	*27*	*81*	*256*
402	*422*	*412*	*3*	*0,06*	*4*	*12*	*48*	*192*	*768*	*1875*
Сума			*50*	*1*		*-32*	*274*	*-488*	*4066*	*3680*

Table 5.2 explanation:

- to simplify the calculations, “conditional zero” C = 332 was selected, which corresponds to the value of x_i with a maximum frequency, and conditional options are calculated u_i;
- computation control

$$\sum n_i u_i^4 + 4\sum n_i u_i^3 + 6\sum n_i u_i^2 + 4\sum n_i u_i + n = 3680,$$

which matches the total value in the column 9 *Table 5.2*.

5.4.2. Construction experimental distribution polygon.

The polygon is constructed in this sequence *(Table 5.2):*

- the range of possible values of a random value (*RV*) of the yield strength of steel (the difference between the highest and lowest values in the sample) is divided into 8 - 12 equal intervals with average values of x_i (column 1);
- the number of *RV* n_i hits in each interval is calculated, moreover $n = \sum n_i$ (column 2);
- the experimental frequencies of the hit of RV in each interval are calculated (including values equal to the lower boundary of the interval)

$$f_i^*(x) = \frac{n_i}{n}, \tag{1.17}$$

Moreover $\sum f_i^* = 1$. Frequency calculations can be performed as a percentage, then $\sum f_i^* = 100\%$. The numerical values of the frequencies of the above sample are presented in the column 3 *Table 5.2*;

- Experimental distribution polygon is constructed (*Fig. 5.7*).
-

5.4.3. Determination of selective numerical characteristics of the yield steel strength. Calculations are performed using the product method in the *Table 5.2.*

Conditional moments 1 - 4 orders are determined:

$$M_1^* = \frac{\sum n_i u_i}{n} = -0{,}64; \qquad M_2^* = \frac{\sum n_i u_i^2}{n} = 5{,}48;$$

$$M_3^* = \frac{\sum n_i u_i^3}{n} = -9{,}76; \qquad M_4^* = \frac{\sum n_i u_i^4}{n} = 81{,}32.$$

Selected numerical characteristics are equal to:

$$\bar{x}^* = M_1^* \cdot h + C = -0{,}64 \cdot 20 + 332 = 319{,}2 \text{ MPa;}$$

$$\hat{x}^* = \left[M_2^* - \left(M_1^*\right)^2 \right] \cdot h^2 = \left(5{,}48 - (-0{,}64)^2\right) \cdot 20^2 = 2028{,}16\,;$$

$$\mu_3 = \left[M_3^* - 3 \cdot M_1^* \cdot M_2^* + 2 \cdot \left(M_1^*\right)^3 \right] \cdot h^3 = 1898{,}5;$$

$$\mu_4 = \left[M_4^* - 4 \cdot M_1^* \cdot M_3^* + 6 \cdot \left(M_1^*\right)^2 \cdot M_2^* - 3 \cdot \left(M_1^*\right)^4 \right] \cdot h^4 = 11087797;$$

$$\hat{x} = \sqrt{2028{,}16} = 45{,}04 \text{ МПа;} \qquad V = \frac{45{,}04}{319{,}2} = 0{,}141;$$

$$A = \frac{1898{,}5{,}124}{45{,}04^3} = 0{,}02; \qquad E_x = \frac{11087797{,}58}{45{,}04^4} - 3 = -0{,}30.$$

5.4.4. Normal distribution selection. The density of the normal distribution (Gaussian) is described by the expression (5.12). This is a symmetric distribution, which is determined by two parameters: $\bar{x}$ and $\hat{x}$. The ordinates of the normalized normal curve for $\bar{x} = 0$ and $\hat{x} = 1$

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

are given in *Table D1* of this tutorial.

The transition to the parameters of the experimental sample is performed as follows:

$$p(x) = \frac{n \cdot h}{\hat{x}} \cdot \varphi(x), \tag{5.18}$$

where n – sample size; using relative frequencies n = 1,0, as a percentage – 100%; h – a step (an interval) equal to 20 in this example; $x = (x - \bar{x})/\hat{x}$ – normalized argument of the normal distribution.

The selection of the ordinates of the normal distribution is presented in *Table 5.3*, the selected normal distribution together with the experimental test site is shown in *Fig. 5.7*.

Table 5.3

The ordinates selection of the normal distribution

x_i	$x_i - \bar{x}$	$(x-\bar{x})/\hat{x}$	$\varphi(x)$,%	$p(x)$, %
212	*-107,20*	*-2,38*	*2,35*	*1,04*
232	*-87,20*	*-1,93*	*6,08*	*2,70*
252	*-67,20*	*-1,49*	*13,15*	*5,84*
272	*-47,20*	*-1,04*	*22,99*	*10,21*
292	*-27,20*	*-0,60*	*33,32*	*14,79*
312	*-7,20*	*-0,15*	*39,39*	*17,49*
332	*12,80*	*0,28*	*38,36*	*17,03*
352	*32,80*	*0,73*	*30,56*	*13,56*
372	*52,80*	*1,17*	*20,21*	*8,97*
392	*72,80*	*1,61*	*10,92*	*4,85*
412	*92,80*	*2,06*	*4,78*	*2,12*
$\bar{x}$ ***=319,2***	***0***	***0***	***39,89***	***17,71***

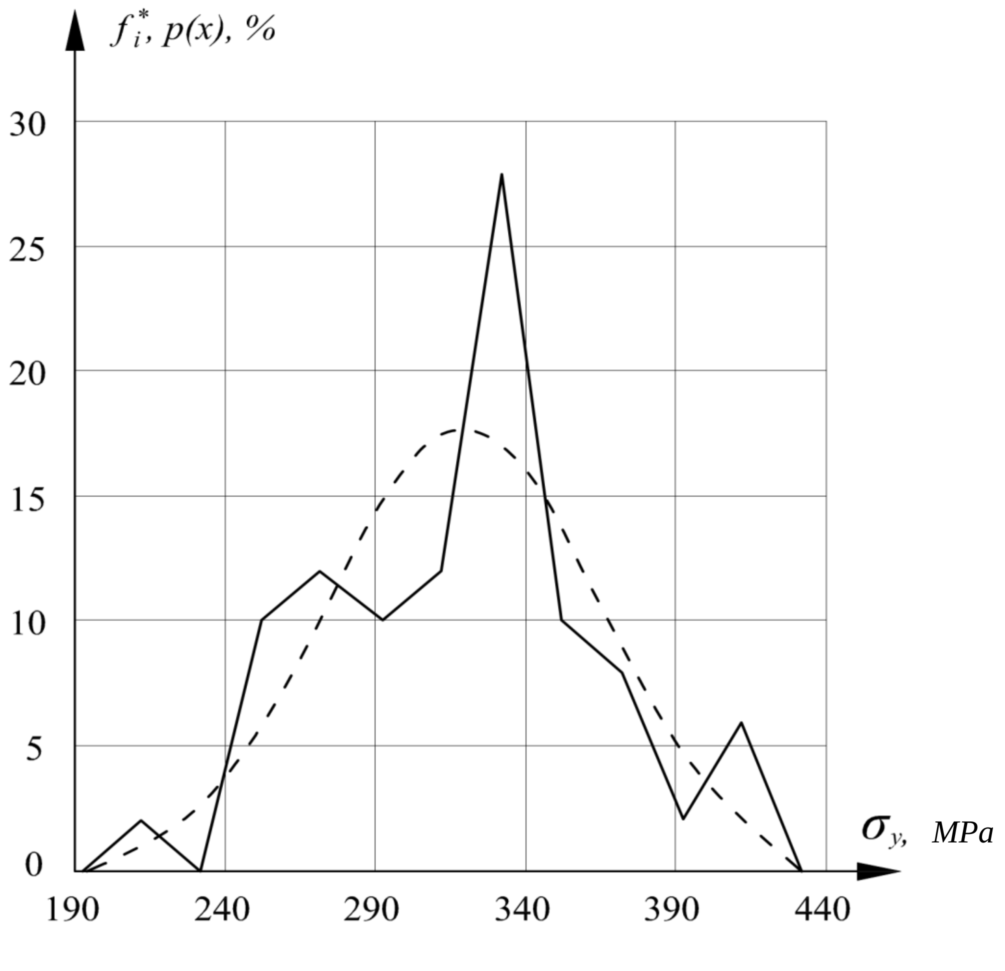


Fig. 5.7. Experimental polygon (solid line) and normal distribution (dotted line)

As can be seen from *Fig. 5.7*, the normal distribution describes quite well the nature of the experimental test site, which oscillates up and down relative to the normal curve; the mode (maximum) of the polygon is close to the center, corresponds to slight asymmetry (*A = 0,02*), some flattening is indicated by a negative kurtosis (*E = -0,3*).

5.4.5. Compliance verification of the experimental distribution to normal. For verification we will use the Pearson criterion in the following order:

- compile the calculated *Table. 1.4*, according to which we find the value of the Pearson criterion from the above observations

$$\chi^2_{cnoc} = \sum \frac{[n_i - p'(x)]^2}{p'(x)}; \tag{5.19}$$

Table 5.4

The calculated values of the Pearson criterion

x_i	n_i	$p'(x) = p(x)n_i$	$n_i - p'(x)$	$[n_i - p'(x)]^2$	$\chi^2_{cnoc} = \frac{[n_i - p'(x)]^2}{p'(x)}$
212	*1*	*0,52*	*0,46*	*0,212*	*0,407*
232	*0*	*0*	*0*	*0*	*0*
252	*5*	*2,92*	*2,08*	*4,326*	*1,482*
272	*6*	*5,10*	*0,90*	*0,810*	*0,159*
292	*5*	*7,40*	*-2,40*	*5,760*	*0,778*
312	*6*	*8,75*	*-2,75*	*7,563*	*0,864*
332	*14*	*8,50*	*-5,50*	*30,250*	*3,559*
352	*5*	*6,80*	*-1,80*	*3,240*	*0,476*
372	*4*	*4,50*	*-0,50*	*0,250*	*0,056*
392	*1*	*2,45*	*-1,45*	*2,103*	*0,858*
412	*3*	*1,06*	*1,94*	*3,764*	*3,551*
				Total χ^2_{cnoc}	***12,190***

- from the table of critical distribution points χ^2 (*Table D.4*) for a given significance level $\alpha = 0{,}05$ and the number of degrees of freedom $k = i - 3 = 8$ (*i* is the number of sample intervals equal to 11 in our example), we find the critical point $\chi^2 = 15{,}5$;

- since it is determined from the results of observations, the criterion value does not exceed the critical point:

$$\chi^2_{cnoc} = 12{,}190 < \chi^2 = 15{,}50,$$

we conclude that the hypothesis of normality is not rejected.

5.4.6. Normal distribution operations. The normal distribution function is determined by density integration (5.12) and can be easily calculated using tabulated Laplace functions $\Phi(\gamma)$ *(Table D.2):*

$$\begin{aligned} F(x) &= \int_{-\infty}^{x} f(x)dx = \frac{1}{\hat{x}\sqrt{2\pi}} \cdot \int_{-\infty}^{x} e^{-\frac{(x-\bar{x})^2}{2\hat{x}^2}} dx = \\ &= 0{,}5 \pm \Phi\left(\frac{x-\bar{x}}{\hat{x}}\right) = 0{,}5 \pm \Phi(\gamma) \end{aligned} . \quad (5.20)$$

A plus sign corresponds to a positive value of the normalized deviation, a minus sign corresponds to a negative value.

1. ***Determination of the falling probability into the interval.***

The calculation is carried out using the Laplace function

$$F(a < x < b) = \Phi\left(\frac{b-\bar{x}}{\hat{x}}\right) - \Phi\left(\frac{a-\bar{x}}{\hat{x}}\right). \quad (5.21)$$

Determine the probability of the random variable considered in the example falling into the interval 290,0 MPa < x <340,0 MPa.

We calculate the normalized arguments:

$$\gamma_1 = \frac{a-\bar{x}}{\hat{x}} = \frac{290{,}0-319{,}2}{45{,}04} = -0{,}648; \quad \gamma_2 = \frac{340{,}0-319{,}2}{45{,}04} = 0{,}462.$$

Use the Laplace functions (*Table D.2*)

$$\Phi_2(0{,}462) = 0{,}5 + 0{,}178 = 0{,}678; \qquad \Phi_1(-0{,}648) = 0{,}5 - 0{,}2415 = 0{,}2585.$$

The probability of falling into a certain interval

$$F(290{,}0 < x < 340{,}0) = 0{,}678 - 0{,}2585 = 0{,}4125 \cong 0{,}413.$$

2. The probability of a random value falling into intervals that are multiples of standards:

$$F\left(\bar{x} \pm \hat{x}\right) = 2 \cdot \Phi(1) = 0{,}3413 = 0{,}6826\,;$$
$$F\left(\bar{x} \pm 2\hat{x}\right) = 2 \cdot \Phi(2) = 0{,}4772 = 0{,}9544\,;$$
$$F\left(\bar{x} \pm 3\hat{x}\right) = 2 \cdot \Phi(3) = 0{,}49865 = 0{,}9973\,.$$

The last line shows that a random variable going beyond the limits $\bar{x} \pm 3\hat{x}$ has a probability of *0,27%*, i.e. it is practically impossible (the “three sigma” rule).

3. Determination of the characteristic steel resistance. As shown in Chapter 5.3 above, the characteristic rolled steel resistance is equal to the yield strength of steel with a security of 0,95 and is determined by the formula (5.16); for a given sample of steel specimen testing, we have for the standard tensile strength of steel:

$$R_{yn} = \bar{x} - 1{,}64\hat{x} = \bar{\sigma}_y - 1{,}64\hat{\sigma}_y = 319{,}2 - 1{,}64 \cdot 45{,}04 = 245{,}33 \text{ MPa.}$$

4. The design steel resistance. According to the recommendations of chapter 5.3, the design steel resistance is determined at a distance of three standards from the mathematical expectation of yield strength, that is, in accordance with the “three sigma” rule, formula (5.17):

$$R_y = \bar{x} - 3\hat{x} = \bar{\sigma}_y - 3\hat{\sigma}_y = 319{,}2 - 3 \cdot 45{,}04 = 184{,}08 \text{ MPa.}$$

Control questions

1. Expand the essence of the distribution curves RV. Draw a graph of this function.
2. What is the numerical distribution characteristics of random variables and what is included in it?
3. What is the strength rating of rolled steel? The Gaussian distribution.
4. How to calculate the characteristic and design steel resistance?

LECTURE 6. EXPONENTIAL LAW OF RELIABILITY

6.1. General formulas
6.2. Numerical example: applying an exponential distribution

6.1. General formulas

6.1.1. Exponential law equation. An important indicator of reliability is ***the failure rate*** - the conditional density of the structural failure probability, is determined provided that the failure did not occur before the accepted point in time:

$$\lambda(t) = \frac{f(t)}{1 - F(t)} = -\frac{1}{P(t)} \cdot \frac{dP(t)}{dt} = -\frac{p(t)}{P(t)}. \qquad (6.1)$$

The failure rate λ numerically shows the number of objects of this type that fail per unit of time. For example, $\lambda = 10^{-3} 1$ / year may mean that when there are 1000 identical structures in operation, then in one year one structure can fail.

Integrate expression (6.1) to determine the failure rate

$$\int_0^t \lambda(t)dt = -\ln P(t).$$

Next, performing the potentiation operation, as a result of which we obtain the formula for the reliability function

$$P(t) = e^{-\int_0^t \lambda(t)dt}. \qquad (6.2)$$

Within the period of normal operation of a system with a constant failure rate $\lambda(t) = \lambda = const$ is obtained

$$P(t) = e^{-\lambda t}. \qquad (6.3)$$

This is ***an exponential law*** of change in reliability over time *(Fig. 6.1, a)*, widespread in practical calculations of reliability in engineering, in particular construction objects and structures. The exponential law is natural from a physical point of view, it is simple and convenient to use, it has only one parameter λ. Its application significantly simplifies the formulas of the theory of structures reliability.

The integral function of the exponential law

$$F(t)=1-P(t)=1-e^{-\lambda t}. \qquad (6.4)$$

Differential function of exponential law

$$f(t)=\lambda e^{-\lambda t}. \qquad (6.5)$$

The numerical characteristics of the exponential law - mathematical expectation and standard - are equal to each other:

$$\bar{t}=\hat{t}=\frac{1}{\lambda}. \qquad (6.6)$$

The coefficient of variation is obviously $V_t=1$.

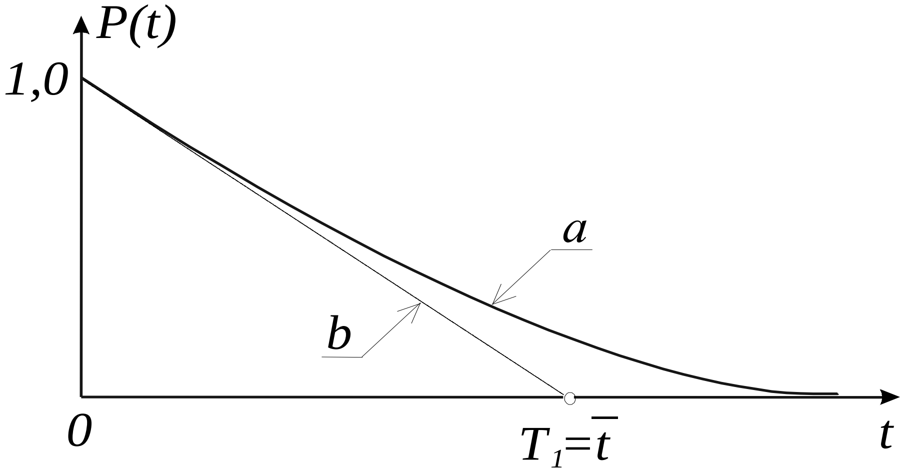


Fig. 6.1. The exponential law of change in reliability:
a) function (6.3); b) linearized expression (6.7)

6.1.2. Linearization of exponential law. In the case when $P(t)>0{,}9$, that is, for highly reliable objects, the linearization operation is performed with the nonlinear function being replaced by a tangent at the selected point. To do this, the function is expanded into a Taylor series and nonlinear terms are rejected. In our case, the Maclaurin series is used, which is a special case of the Taylor series when the abscissa of the schedule point is $t=0$:

$$P(t)=P(0)+\frac{P'(o)}{1!}t+\frac{P''(o)}{2!}t^2+\ldots+\frac{P^{(n)}(o)}{n!}t^n;$$

$$P(t) = 1 - \lambda t + \frac{\lambda^2}{2!} t^2 - \frac{\lambda^3}{3!} t^3 \ldots$$

Rejecting nonlinear terms, we obtain a simpler expression for the reliability function *(Fig. 6.1, b)*

$$P(t) = e^{-\lambda t} \cong 1 - \lambda t . \qquad (6.7)$$

The integral distribution function accordingly has the expression

$$F(t) = 1 - e^{-\lambda t} \cong \lambda t .$$

Formula (6.7) gives sufficient accuracy of practical calculations, in particular building structures.

6.1.3 Mean time to failure for an exponential law is determined taking into account (6.3)

$$T_1 = \int_0^\infty e^{-\lambda t} dt = -\frac{1}{\lambda} \cdot e^{-\lambda t} \Big|_0^\infty = \frac{1}{\lambda} .$$

So, we have such simple relations:

$$\lambda = \frac{1}{T_1}; \;\; T_1 = \frac{1}{\lambda}. \qquad (6.8)$$

Given this function of the exponential law (6.4), (6.5) we obtain the following form:

$$F(t) = 1 - e^{-\frac{t}{T_1}}; \quad f(t) = \frac{1}{T_1} e^{-\frac{t}{T_1}} . \qquad (6.9)$$

6.2. Numerical example: applying an exponential distribution

Direction. Describe the experimental data by the exponential distribution, determine the structure gamma-percentile operating time to failure t_γ at the following levels: γ = 50, 80, 90, 95 та 99%.

6.2.1. Initial data. An experimental values selection of the operating time of auxiliary structures (in years) (volume n = 20): 39,5; 63,9; 10,9; 83,9; 52,7; 27,1; 70,8; 0,3; 244,1; 5,4; 180,5; 58,4; 7,4; 228,2; 9,2; 7,8; 140,2; 193,9; 297,7; 3,3.

6.2.2. Construction a research distribution histogram. We distribute the range of changes in the research operating time into intervals, count the number of hits n_i in each interval, the relative frequencies $f_i^*(t) = n_i / n$ and ordinates of the experimental histogram $p_i^* = f_i^*(t)/h$, where $h = 50$ is the sampling step (interval). The obtained numerical values are given in columns 2, 3 and 4 of the *Table 6.1*, research histogram – in *Fig. 6.2, a*.

6.2.3. Calculation of sample numerical characteristics.

According to the procedure given in paragraph 5.4.3 above, we calculate the estimates of the numerical characteristics of a given sample:

- Mathematical expectation (mean time to failure) $\bar{t} = T_1 = 86{,}26$ years;
- standard $\hat{t} = 93{,}55$ years, coefficient of variation $V_t = 1{,}08$;
- asymmetry $A_t = 0{,}89$, kurtosis $E_t = -0{,}68$.

Table 6.1

Comparison of research and theoretical distributions

t_i	*Research distribution*			*Theoretical distribution*			*Pearson criterion*		
	n_i	f_i^*	p_i^*	$f(t)$	$F(t)$	$P(t)$	F_i^*	F_i	χ^2
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*
50	*9*	*0,45*	*0,009*	*0-0,0116* / *0,0065*	*0,440*	*0,560*	*0,45*	*0,440*	*0,004*
100	*6*	*0,30*	*0,006*	*0,0036*	*0,686*	*0,314*	*0,30*	*0,246*	*0,234*
150	*0*	*0*	*0*	*0,0020*	*0,824*	*0,176*	*0,15*	*0,232*	*0,596*
200	*3*	*0,15*	*0,003*	*0,0011*	*0,918*	*0,098*			
250	*1*	*0,05*	*0,001*	*0,00064*	*0,945*	*0,055*	*0,10*	*0,051*	*0,940*
300	*1*	*0,05*	*0,001*	*0,00035*	*0,969*	*0,031*			
350				*0,0002*	*0,982*	*0,017*			
$\sum$	*n=20*								$\chi^2_{cnoc.} = 1{,}776$

6.2.4. Parameter calculations λ, construction of theoretical curves. We determine the exponential distribution parameter**.**

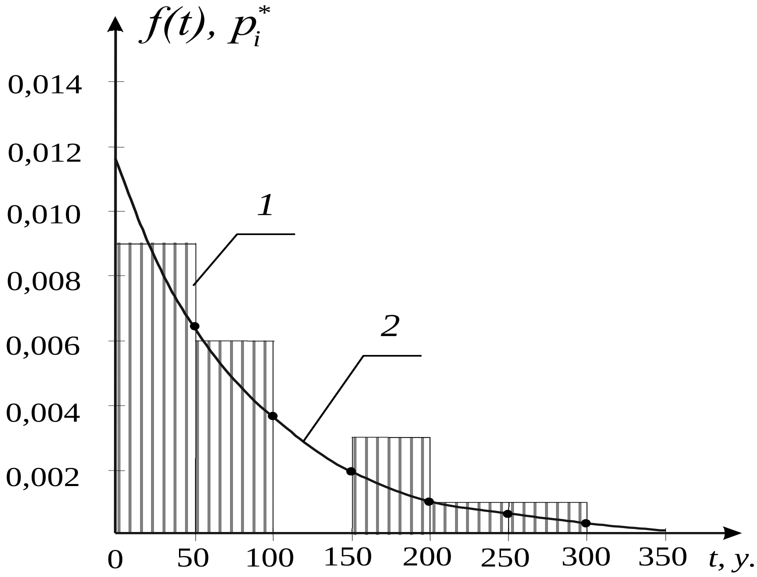
$$\lambda = \frac{1}{T_1} = \frac{1}{86{,}26} = 0{,}0116 \text{ 1/year.}$$

Constructing the curves of the exponential distribution approximating the experimental histogram:

- distribution curve $f(t)$, according to the formula (6.5) (*Fig. 6.2, a, position 2)*;
- reliability function $P(t)$, according to the formula (6.3) (*Fig.6.2, b*);
- integral exponential distribution function $F(t)$, according to the formula (6.4) (*Fig. 6.2, b*);
- linearized function (6.7) (*Fig. 6.2, b, position 3*).

The numerical values of the listed functions are given in columns 5, 6 and 7 of the *Table 6.1*.

a)



b)

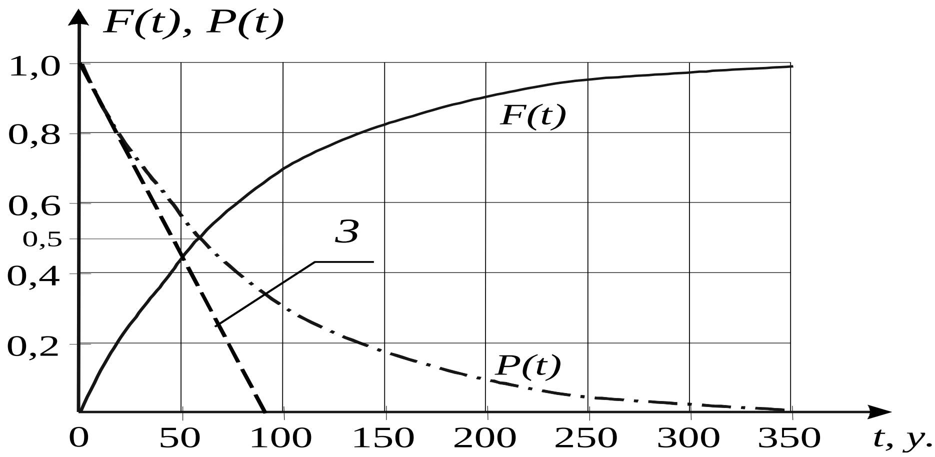


Рис. 6.2. The exponential distribution:

a) research histogram (1), the exponential law (2); b) reliability function, integral exponential distribution function and linearized function (3)

6.2.5. The correspondence of the experimental distribution to the theoretical one is estimated using the Pearson criterion, the corresponding numerical values are given in columns 8, 9 and 10 of *the Table. 6.1*. In calculating the values of the criterion, we will use the values of the probabilities, moreover, enlargement of the intervals is allowed:

$$\chi^2 = \sum \frac{\left(F_I^* - F_i\right)^2}{F_i}.$$

Since the exponential distribution is one-parameter, the number of degrees of freedom is defined as $k = i - 2$; therefore, in this case $k = 2$ (i is the number of sampling intervals). From *Table D.4* in terms of significance $\alpha = 0{,}05$ and $k = 2$ figure out $\chi^2 = 6{,}0 > \chi^2_{спос.} = 1{,}776$, therefore there is no reason to reject the hypothesis of an exponential distribution of the structures operating time.

6.2.6. Gamma-percentile operating time to failure.

Operating time to failure is the time of failure-free operation of the structure from the start of operation until the first failure.

Average mean time to first failure is the mathematical expectation of the mean time to first failure.

$$T_1 = \int_0^\infty t \cdot f(t)dt = \int_0^\infty [1 - F(t)]dt = \int_0^\infty P(t)dt. \qquad (6.10)$$

The value of T_1 is equal to the area under the curve of the reliability function $P(t)$.

Gamma-percentile operating time to failure t_γ – operating time during which structural failure does not occur with a probability γ expressed as a percentage, and which is defined as the equation root

$$P(t_\gamma) = \frac{\gamma}{100}. \qquad (6.11)$$

Thus, t_γ is the quantile of the corresponding distribution. To determine the reliability indicators of structures, rather high levels are equal γ = *90, 95, 99, 99,5%*, etc., which corresponds to the failure probabilities in the interval [*0; t*] *Q = 0,10; 0,05; 0,01; 0,005*.

Equate expression (6.11) in the exponential formula (6.3)

$$P(t_\gamma) = \frac{\gamma}{100} = e^{-\lambda t_\gamma}.$$

After the transformations, we obtain an expression that determines γ the percentage time between failures if the time is described by an exponential distribution

$$t_\gamma = -\frac{ln\left(\frac{\gamma}{100}\right)}{\lambda}. \qquad (6.12)$$

The numerical values of operating hours are illustrated in *Fig. 6.3*, they are equal to: $t_{50} = 59{,}84$ y.; $t_{80} = 19{,}24$ y.; $t_{90} = 9{,}14$ y.; $t_{95} = 4{,}44$ y.; $t_{99} = 0{,}99$ y.

Thus, with increasing γ in the percentage level, the operating time t_γ decreases rather quickly:

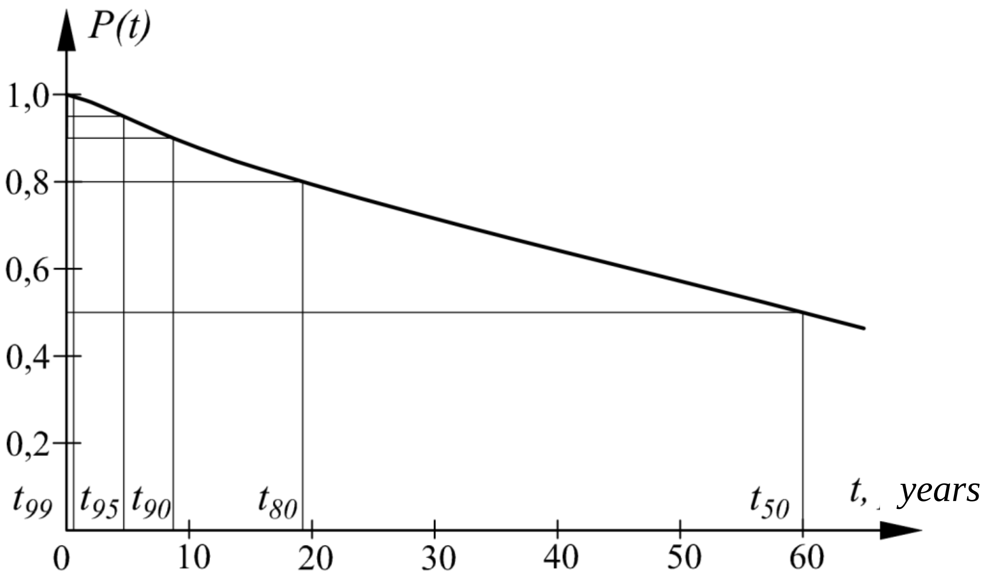


Fig. 6.3. Gamma-percentile operating time to failure

Control questions

1. What is the failure rate? How is the failure rate related to an exponential law?

2. Graphically depict the exponential law of change in reliability.

3. Expand the essence of mean time to failure.

4. How does the correspondence of the experimental distribution to the theoretical take place?

5. Reveal the formula for calculating gamma-percentile operating time to failure.

LECTURE 7. RELIABILITY ASSESSMENT OF BUILDING STRUCTURES

7.1. Random value technique solution
7.2. Random process solution
7.3. Using an independent test scheme
7.4. Numerical example: structural reliability calculation in the technique of random variables
7.5. Numerical example: structural reliability calculation in the technique of random processes
7.6. Numerical example: structural reliability calculation according to the scheme of independent tests

7.1. Random value technique solution

The generalized condition for failure-free operation (indestructibility) of the structure has the following form:

$$\tilde{Y}(t) = \tilde{R}(t) - \tilde{S}(t), \tag{7.1}$$

Where $\tilde{R}(t)$ is the generalized bearing capacity of the structure; $\tilde{S}(t)$ – generalized load on the structure; $\tilde{Y}(t)$ – characteristic introduced by A.R. Rzhanitsyn and called the safety margin. Let us call this characteristic ***a reserve of bearing capacity***, taking into account the fact that within the framework of this concept the reliability problems of compressed and compressed-curved elements will be further solved.

In this chapter, we apply the probabilistic technique of random variables without taking into account the time factor t, which is justified under the influence of loads, changes little in time (constant and some technological) or of a one-time nature. In this case, the function of the reserve of bearing capacity is written as

$$\tilde{Y} = \tilde{R} - \tilde{S} \geq 0. \tag{7.2}$$

Let's consider a simple case when a structure is loaded with random mechanical loads $\tilde{q}$, which entails random mechanical stresses $\tilde{\sigma}$ for the structure. Structural strength is also a random variable; it is determined by random fracture stresses $\tilde{\sigma}_y$ (for example, yield strength for mild steel). The bearing capacity of the structure is equal to $\tilde{R} = \tilde{\sigma}_y A$, where A is the geometric characteristic of the section of the structure; $\tilde{S} = \alpha\tilde{q}$ - efforts in the design from external load. The reserve of bearing capacity can also be determined in the stress space, then $\tilde{R} = \tilde{\sigma}_y$, $\tilde{S} = \tilde{\sigma}$.

The zone of permissible states is a set, for each element of which the inequality

$$\Omega = \{Y; y = (r - s) \geq 0\}.$$

In coordinates $R - S$, the zone Ω has a triangular shape and is located above the line $R = S$ (*Fig. 7.1*).

The probability failure-free operation is

$$P = \int_{\Omega} f(Y)dY. \tag{7.3}$$

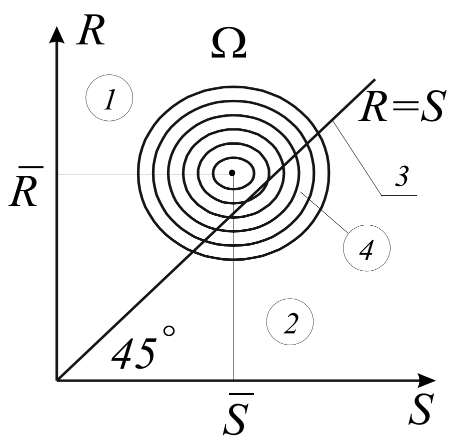


Fig. 7.1. On the reliability assessment in the technique of random variables:
1 - zone of permissible states; 2 - failure zone; 3 - limit of permissible zone;
4 - distribution projection $f(Y)$

The mathematical expectation and the standard of the bearing capacity reserve are determined, as for a linear function:

$$\bar{Y} = \bar{R} - \bar{S}; \quad \hat{Y} = \sqrt{\hat{R}^2 + \hat{S}^2}. \tag{7.4}$$

Characteristic that is defined as

$$\beta = \frac{\bar{Y}}{\hat{Y}} = \frac{1}{V_Y} = \frac{\bar{R} - \bar{S}}{\sqrt{\hat{R}^2 + \hat{S}^2}}, \tag{7.5}$$

Is called ***a safety characteristic*** (A.R. Rzhanitsyn) or ***a safety index*** (S.A. Cornell), it determines the probability of failure *(Fig. 7.2)*

$$Q(Y \le 0) = F_Y(0) = F_Y\left(\overline{Y} - \beta \hat{Y}\right). \qquad (7.6)$$

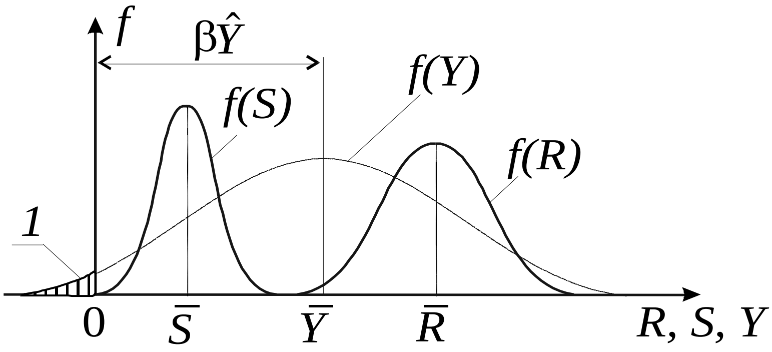


Fig. 7.2. Determining the probability of failure:
1 - failure section

Using the coefficients of variation $V_S = \hat{S}/\overline{S}$ and $V_R = \hat{R}/\overline{R}$ the corresponding expressions $\overline{R} = \hat{R}/V_R$ and $\overline{S} = \hat{S}/V_S$, taking into account the ratio $p = \hat{R}/\hat{S}$, we obtain a convenient dimensionless form for β

$$\beta = \frac{pV_S - V_R}{V_R V_S \sqrt{1 + p^2}}. \qquad (7.7)$$

In the case of a normal distribution $f(Y)$, the safety characteristic is very convenient for determining the probabilities of failure (Q) and failure free operation (P):

$$Q(Y < 0) = 0{,}5 - \Phi(\beta), \quad P(Y \ge 0) = 0{,}5 + \Phi(\beta), \qquad (7.8)$$

where $\Phi(\beta)$ – is Laplace function *Table D.2.*

7.2. Random process solution

When applying the probabilistic model of random processes in expression (7.1), $\tilde{S}(t)$ is the force (or stress) in the structure in the form of a random process; $\tilde{R}(t)$ - a random process or a random value $\tilde{R}$ of the bearing capacity; $\tilde{Y}(t)$ - random reserve process of the bearing capacity of the structure.

Under such conditions, a structural failure is interpreted as an ***outlier*** of random force $\tilde{S}(t)$ at a random level of bearing capacity $\tilde{R}(t)$ *(Fig. 7.3)* or as an outlier of RV $\tilde{Y}(t)$ in the negative section.

If we assume that the load and bearing capacity are described by stationary or quasi-stationary random processes, then the estimate of the probability of structural failure can be determined by the number of outlier $N_+(t)$ as

$$Q(t) \cong N_+(t) = \frac{\omega_q f_Y(\beta) t}{\beta_\omega \sqrt{2\pi}}. \qquad (7.9)$$

This formula was obtained in [2], it uses the following notation: ω_q – effective frequency of the random process of the reserve of bearing capacity; $f_Y(\beta)$ – ordinate of the density distribution of the reserve function of the reserve of bearing capacity $\tilde{Y}$, which corresponds to the value of the safety characteristic β *(Fig. 7.2);* t – operating time design; β_ω – coefficient of broadband random process $\tilde{Y}(t)$.

If $\tilde{R}(t)$ and $\tilde{S}(t)$ are distributed normally, then $\tilde{Y}(t)$ also, a normal distribution, and the formula for $Q(t)$ gets the following form:

$$Q(t) = \omega_q \cdot \frac{e^{-0,5\beta^2} t}{2\pi\beta_\omega}. \qquad (7.10)$$

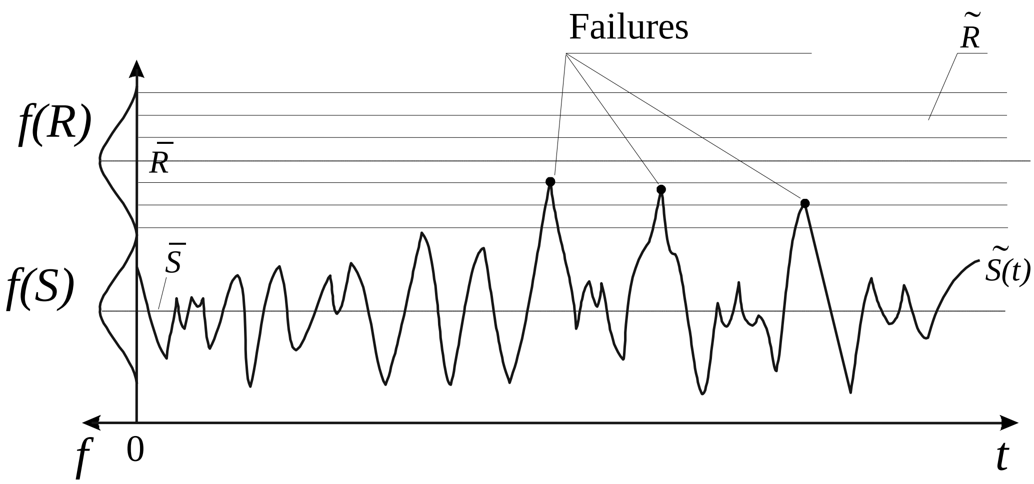


Fig. 7.3. Element failures under load in the form of a random process

Based on the formula (7.10) and given the standard values of the probability of failure $[Q]$, it is easy to determine the corresponding safety characteristic

$$\beta=\sqrt{2\ln\frac{\omega_q t}{2\pi[Q]\beta_\omega}}\,. \tag{7.11}$$

7.3. Using an independent test scheme

An independent test scheme is a random sequence of independent random loads, according to which the probability of a load not exceeding a normalized level γ over time t is determined as the result $n_н$ of independent tests (independent loads)

$$P(\gamma,t)=[P(\gamma)]^{n_н}, \tag{7.12}$$

where $P(\gamma)=P_1(\gamma)$ − the probability of not exceeding the level γ in a separate test.

Accordingly, the probability of failure is defined as

$$Q(\gamma,t)=1-P(\gamma,t)=1-[P_1(\gamma)]^{n_н}\,. \tag{7.13}$$

For highly reliable structures, formula (3.13) is simplified with sufficient accuracy:

$$Q_n(\gamma,t)=1-[P_1(\gamma)]^{n_н}\cong n_н Q_1(\gamma). \tag{7.14}$$

In practical calculations, it is useful to use the ratio between the differential and integral functions of any distribution

$$\mu(\gamma)=\frac{f(\gamma)}{1-F(\gamma)}=\frac{f(\gamma)}{Q(\gamma)}, \tag{7.15}$$

where $\mu(\gamma)$ – distribution intensity function.

For some distributions $\mu(\gamma)$ is determined in a general analytical form (Weibul distribution), for most distributions (for example, normal) $\mu(\gamma)$ numerically, tables can also be used.

Based on the simplified formula (3.14), using $\mu(\gamma)$ the standard value of the probability of failure $[Q]$, we obtain the ratio for the normal distribution $Q_1(\beta)$:

$$n_н Q_1(\beta) = n_н \frac{f(\beta)}{\mu(\beta)} = n_н \frac{\exp\left(-0{,}5\beta^2\right)}{\mu(\beta)\sqrt{2\pi}} = [Q].$$

The corresponding safety characteristic is easily determined from here

$$\beta = \sqrt{2\ln \frac{n_н}{\mu(\beta)\sqrt{2\pi}[Q]}}\ . \qquad (7.16)$$

The solution to such a nonlinear equation is simplified when you consider that for a normal law for large β, the relation $\mu(\beta) \approx \beta)$ is valid.

7.4. Numerical example: structural reliability calculation in the technique of random variables

Direction. To evaluate the reliability (in the technique of random values) of an overhead line wire by the criterion of mechanical strength under icing conditions. In case of insufficient reliability, select the required wire diameter *(Fig. 7.4, a)*.

7.4.1. Initial data:

- individual: $d = 10\,mm$ – the wire diameter; $l = 100\,m$ – wire span; $b = 30\,mm$ – ice thickness; $\bar{\rho} = 0{,}50\,g/cm^3$ – average ice density; $V_\rho = 0{,}3$ – variation coefficient of density;
- general: $\bar{\sigma}_y = 150\,MPa$ – average ultimate stress in a wire; $V_\sigma = 0{,}2$ – variation coefficient of the ultimate stress; $[P] = 0{,}999$ – normative indicator of reliability (probability of failure free operation).

7.4.2. Wire bearing strength:

- wire cross-sectional area:

$$A = \frac{\pi d^2}{4} = \frac{\pi \cdot 1^2}{4} = 0{,}785\ cm^2;$$

- mathematical expectation of wire carrying capacity

$$\bar{R} = \bar{\sigma}_y A = 15 \cdot 0{,}785 = 11{,}8\,кN;$$

- bearing capacity standard

$$\hat{R} = \bar{R} V_\sigma = 11{,}8 \cdot 0{,}2 = 2{,}36\,кN.$$

7.4.3. Effort in a wire from an external load.

- Neglecting the own weight of the wire, taking into account only ice with a cross-sectional area

$$S_{кр} = \pi\left(b^2 + db\right).$$

- The mathematical expectation of the linear load of the wire from ice

$$\bar{q} = \pi\left(b^2 + db\right)\bar{\rho} g \cdot 10^{-3} = \pi\left(30^2 + 10 \cdot 30\right) \cdot 0{,}5 \cdot 9{,}8 \cdot 10^{-3} = 18{,}47 \ N/m,$$

where b and d – ice thickness and diameter, *mm*;
$\bar{\rho}$ – average ice density, *g/cm³*;
g – acceleration of gravity, *m/c²*.

- The average strut (effort) in the wire as in an inappropriate flexible thread (at $f \geq \frac{l}{20} = 5\,м$):

$$\bar{H} = \bar{S} = \frac{\bar{q} l^2}{8 f} = \frac{18{,}47 \cdot 100^2}{8 \cdot 5} = 4{,}62\,кN.$$

$$\hat{S} = \hat{H} = \bar{H} V_\rho = 4{,}62 \cdot 0{,}3 = 1{,}39\,кN.$$

7.4.4 Wire reliability assessment.

- The characteristic of the reserve of bearing capacity according to formulas (7.4):

$$\bar{Y} = \bar{R} - \bar{S} = 11{,}8 - 4{,}62 = 7{,}18 kN.$$

$$\hat{Y} = \sqrt{\hat{R}^2 + \hat{S}^2} = \sqrt{2{,}36^2 + 1{,}39^2} = 2{,}74 \ кN.$$

- Safety characteristic according to the formula (7.5)

$$\beta = \frac{\bar{Y}}{\hat{Y}} = \frac{7{,}18}{2{,}74} = 2{,}62.$$

- The probability of failure free operation according to the formula (7.8)

$$P(\beta)=0{,}5+\Phi(\beta)=0{,}5+0{,}4956=0{,}9956<[P]=0{,}999.$$

The reliability of the wire is insufficient.

7.4.5. Selection of a new wire diameter. Accepting $d=12mm$ and repeat the reliability check.

$$A=\frac{\pi\cdot 1{,}2^2}{4}=1{,}13\ sm^2;\quad \overline{R}=15\cdot 1{,}13=16{,}96\ kN;$$

$$\hat{R}=16{,}96\cdot 0{,}2=3{,}39\ \kappa N;$$

$$\overline{q}=\pi\left(30^2+12\cdot 30\right)\cdot 0{,}5\cdot 9{,}8\cdot 10^{-3}=19{,}4\ N/m;$$

$$\overline{S}=\frac{19{,}4\cdot 100^2}{8\cdot 5}=4{,}85kN;\ \hat{S}=4{,}85\cdot 0{,}3=1{,}46\ \kappa N;$$

$$\overline{Y}=16{,}96-4{,}85=12{,}1kN;\quad \hat{Y}=\sqrt{3{,}92^2+1{,}46^2}=3{,}69\ \kappa N;$$

$$\beta=\frac{12{,}1}{3{,}69}=3{,}28;\quad P(\beta)=0{,}9995>[P]=0{,}999.$$

The reliability of the wire is sufficient.

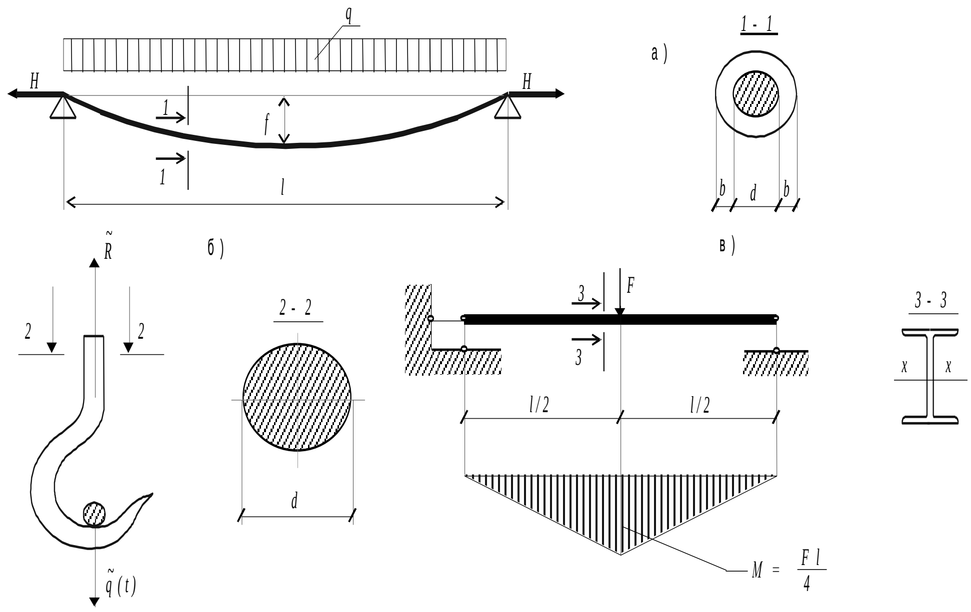


Fig. 7.4. To assess the reliability of structures:
a - wire overhead line;
b - cargo hook of an overhead crane;
c - beam under repeated loading

7.5. Numerical example: structural reliability calculation in the technique of random processes

Direction. *Estimate the probability of the cargo hook failure of an overhead crane (using the random process technique). Choose a new diameter of the cylindrical part, based on the condition* $Q(t) \le [Q] = 10^{-3}$ *(Fig. 7.4, b).*

7.5.1. Initial data:

- individual: $d = 50\,mm$ – cylinder diameter; $q = 30\,mc$ – lifting capacity of an overhead crane; crane operation mode – $7K$, single-shift; crane service life $t = 10$ years;
- general: $\overline{q} = 0{,}5q,\ V_q = 0{,}2$ – numerical characteristics of the cargo; cargo – a normal stationary process with an effective frequency of $\omega_\kappa = 71{,}0\ 1/h$ for mode cranes $4K - 6K$; $\omega_\kappa = 107{,}0\ 1/h$ – mode $7K$; $\omega_\kappa = 215{,}0\ 1/h$ – mode $8K$; $\overline{\sigma}_V = 220\,МПа$, $V_\sigma = 0{,}10$ – numerical characteristics of the fatigue limit of the steel hook.

7.5.2. Hook load reserve characteristics:

$$\overline{R} = \overline{\sigma}_V A = \frac{22 \cdot \pi \cdot 5^2}{4} = 432\,kN; \quad \hat{R} = 0{,}1 \cdot 432 = 43{,}2\ \kappa H;$$

$$\vec{S} = \overline{q} = 0{,}5 \cdot 300 = 150\,kN; \quad \hat{S} = 0{,}2 \cdot 150 = 30\ \kappa H;$$

$$\overline{Y} = 432 - 150 = 282\ \kappa H; \quad \hat{Y} = \sqrt{43{,}2^2 + 30^2} = 52{,}6\ \kappa H;$$

$$\beta = \frac{282}{52{,}6} = 5{,}36.$$

7.5.3. Hook Failure Rate:

- effective frequency of the random process of the bearing capacity reserve $\tilde{Y}(t)$

$$\omega_q = \frac{\omega_\kappa \hat{S}}{\hat{Y}} = \frac{107 \cdot 30}{52{,}6} = 60{,}9\ 1/h;$$

- broadband coefficient

$$K = \frac{\hat{S}}{\hat{R}} = \frac{30}{43{,}2} = 0{,}694; \quad \beta_\omega = \frac{3\sqrt{1 + 0{,}694^2}}{0{,}694} = 5{,}26,$$

where the multiplier 3 is the crane bandwidth coefficient;

- the probability of the hook failure by the time according to the formula (7.10)

$$Q(t)=\frac{\omega_q e^{\left(-0,5\beta^2\right)}t}{2\pi\beta_\omega}=\frac{60,9\cdot e^{\left(-0,5\cdot 5,36^2\right)}\cdot 10\cdot 8\cdot 365}{2\pi\cdot 5,26}=0,053>[Q]=10^{-3},$$

where $t=10\cdot 8\cdot 365(year)$ – operating hours, which is determined by multiplying the specified number of service years by the number of work hours day (taking into account the specified number of work shifts per day) and the annual days number.

The calculation showed that the hook reliability is insufficient.

7.5.4. Selection of a new hook diameter. We obtain the required value β by subs tituting numerical values in the formula (7.11):

$$\beta=\sqrt{2\ln\frac{60,9\cdot 10\cdot 8\cdot 365}{2\pi\cdot 5,26\cdot 0,001}}=5,97.$$

The mathematical expectation of the hook load-bearing capacity $\overline{R}$ is found from the quadratic equation derived from formula (7.11):

$$\overline{R}^2-b\overline{R}+C=0;\ b=\frac{2\overline{S}}{1-\beta^2V_\sigma^2};\ C=\overline{S}^2\frac{1-\beta^2V_S^2}{1-\beta^2V_\sigma^2}. \tag{7.17}$$

We substitute the numerical values and solve the equation:

$$b=\frac{2\cdot 150}{1-5,97^2\cdot 0,1^2}=466,1;\ C=150^2\frac{1-5,97^2\cdot 0,2^2}{1-5,97^2\cdot 0,1^2}=-14894,0;$$

$$\overline{R}=\frac{466,1}{2}+\sqrt{\left(\frac{466,1}{2}\right)^2+14894}=496,0\ \kappa H.$$

The new diameter of the cylindrical part

$$d=\sqrt{\frac{4\overline{R}}{\pi\overline{\sigma}_V}}=\sqrt{\frac{4\cdot 496}{\pi\cdot 22}}=5,36\approx 54mm.$$

Checking the new cross-section area:

$$\overline{R}=\frac{22\cdot\pi\cdot 5{,}4^2}{4}=503{,}8kN;\quad \hat{R}=0{,}1\cdot 503{,}6=50{,}4\ \kappa H;$$

$$\overline{Y}=503{,}8-150=353{,}8\ \kappa H;\quad \hat{Y}=\sqrt{50{,}4^2+30^2}=58{,}7\ \kappa H;$$

$$\beta=\frac{353{,}8}{58{,}7}=6{,}03;\quad \omega_q=\frac{107\cdot 30}{58{,}7}=54{,}6\ 1/h;$$

$$K=\frac{30}{50{,}4}=0{,}595;\quad \beta_\omega=\frac{3\sqrt{1+0{,}595^2}}{0{,}595}=5{,}87;$$

$$Q(t)=\frac{54{,}6\cdot e^{-0{,}5\cdot 6{,}03^2}\cdot 10\cdot 8\cdot 365}{2\pi\cdot 5{,}87}=0{,}55\cdot 10^{-3}<[Q]=10^{-3}.$$

The reliability of the hook is insufficient.

7.6. Numerical example: structural reliability calculation according to the scheme of independent tests

7.6.1. Initial data:

- individual: mathematical expectation of load $\overline{F}=100\kappa N$; beam span $l=2m$; number of loading cycles $n=10^6$; $[Q]=10^{-2}$;
- general: numerical characteristics of the steel fatigue strength $\overline{\sigma}_V=300MPa$; $V_\sigma=0{,}1$; load variation coefficient $V_F=V_S=0{,}2$; a rolled-steel joist with parallel faces of shelves according to the assortment of rolling I-beams.

7.6.2. Characteristics of the force (moment) in the beam:

$$\overline{S}=\overline{M}=\frac{\overline{F}l}{4}=\frac{100\cdot 2}{4}=50\,\kappa N\cdot м;\quad \hat{S}=50\cdot 0{,}2=10\,\kappa N\cdot м.$$

7.6.2. Safety feature β. We use the independent testing scheme and the approximate formula β (7.14)

$$Q_n(\beta)\cong nQ_1(\beta),$$

where Q_1 – probability of failure with a separate download (test); Q_n – the same with n tests.

Let us solve the selection of equation (7.16), based on the condition $\mu(\beta) \approx \beta$, for this example we have

$$\beta = \sqrt{2\ln\frac{10^6}{5.63\sqrt{2\pi}\cdot 10^{-2}}} = 5{,}63.$$

7.6.3. Selection of the beam section. We use the quadratic equation (7.17):

$$b = \frac{2\cdot 50}{1-(5{,}63\cdot 0{,}1)^2} = 146{,}4; \qquad C = 50^2\cdot\frac{1-(5{,}63\cdot 0{,}2)^2}{1-(5{,}63\cdot 0{,}1)^2} = -980{,}5;$$

$$\overline{R} = \frac{146{,}4}{2} + \sqrt{\left(\frac{146{,}4}{2}\right)^2 + 980{,}5} = 152{,}8\ kN\cdot m.$$

Required resistance moment of beam

$$W_{\text{р}} = \frac{\overline{R}}{\overline{\sigma}_V} = \frac{152{,}8\cdot 100}{30} = 509{,}4\ sm^3.$$

Accept the I-bean №30Б3, W_K= 521 sm^3.

7.6.4. Checking the accepted beam section:

$$\overline{R} = 30\cdot 521 = 156{,}3\ kN\cdot m; \quad \hat{R} = 0{,}1\cdot 156{,}3 = 15{,}63\ kN\cdot m;$$

$$\overline{Y} = 156{,}3 - 50 = 106{,}3\ kN\cdot m; \quad \hat{Y} = \sqrt{15{,}63^2 + 10^2} = 18{,}6\ kN\cdot m;.$$

$$\beta = \frac{106{,}3}{18{,}6} = 5{,}73.$$

According to the normal distribution *table, D.3* finding out

$$Q_1 = Q(\beta = 5{,}73) = 6\cdot 10^{-9}.$$

According to the formula (7.14) for $n = 10^6$ tests have

$$Q_{\bullet} = 10^6\cdot 6\cdot 10^{-9} = 0{,}6\cdot 10^{-2} < [Q] = 10^{-2}.$$

The reliability of the beam is insufficient.

Control questions

1. What does it mean a reserve of bearing capacity?
2. What does determine the safety characteristic? Depict graphically this determination.
3. What is it a safety characteristic and how it is affected?

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LECTURE 8. WIND LOAD ON BUILDING STRUCTURES

8.1. Stochastic properties of wind loads
8.2. Probabilistic models of the wind load
8.3. Comparison of probabilistic models of wind load
8.4. Mean wind parameters of Ukrainian districts
8.5. Provision of design values of the wind load
8.6. Trend coefficient K_{tr}
8.7. Auxiliary coefficient δ
8.8. Temporary factor γ_T
8.9. Conclusions

The evolution of reliability computation theory and design Codes of building structures are still of interest because of the complexity of a problem on the one hand and on the other of the ignoring the random loads including the wind ones. The numeric values of wind loads probabilistic parameters were not determined for different districts of Ukraine. The traditional approach does not enable to get the exact structure reliability evaluation for the practical purposes especially for some building processes as reconstruction, erection and setting up the unique structures. There were used different probabilistic models for wind load description but the most of them are based on the deficient statistic material and presented in the forms of samples.

In a previous paper [1] only some problems on wind load description were examined. In this lecture we'll give more detailed information on this matter. The given below results are treated as the integral part of reliability estimation method which was developed in the previous works [2, 3].

8.1. Stochastic properties of wind loads

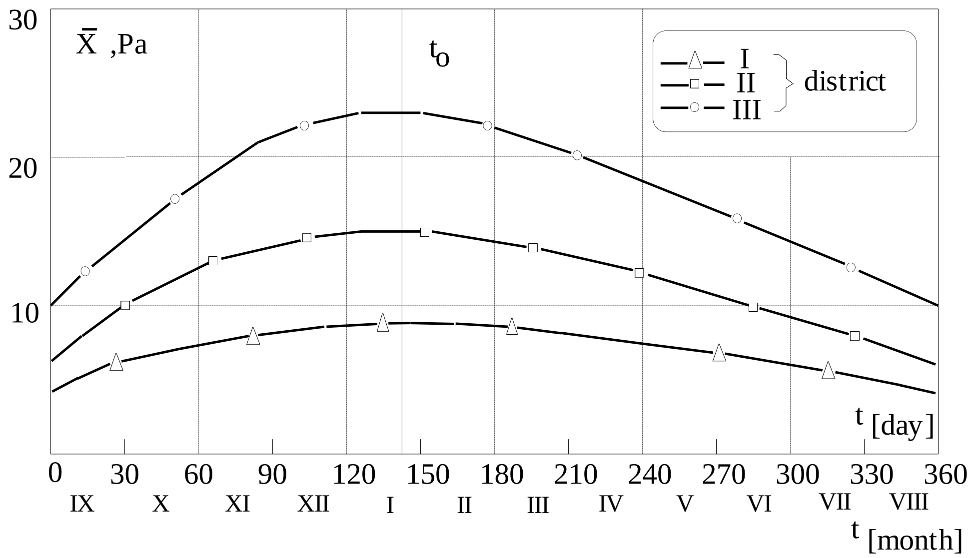

Fig. 8.1. Season change of the wind load mathematic expectation

The systematic results of the wind velocity measurements are quite informative and allow getting the ordinate distribution, the numeric characteristics and frequent parameters of the mean wind value. This value corresponds to the macro-meteorological peak of Vanderkhoven wind spectrum [4], with the period of 1 up 4 twenty-four hours.

Wind load has the season change of the mathematic expectation *X* and the standard deviation *X* during the year, which can be described approximately with polynomial of the 3rd degree as it has illustrated in *Fig. 8.1*. At the same time the coefficient of variation *V*, relative skewness *S* and excess *E* can be treated as time independent.

The wind load for examined territory is of the stationary frequency nature. This solution is based on the fact that its normalized correlation function and effective frequency have no significant distinctions during the year.

The experimental distributions of the wind load are well corresponded to the Veibull's law (see *Fig. 8.2*).

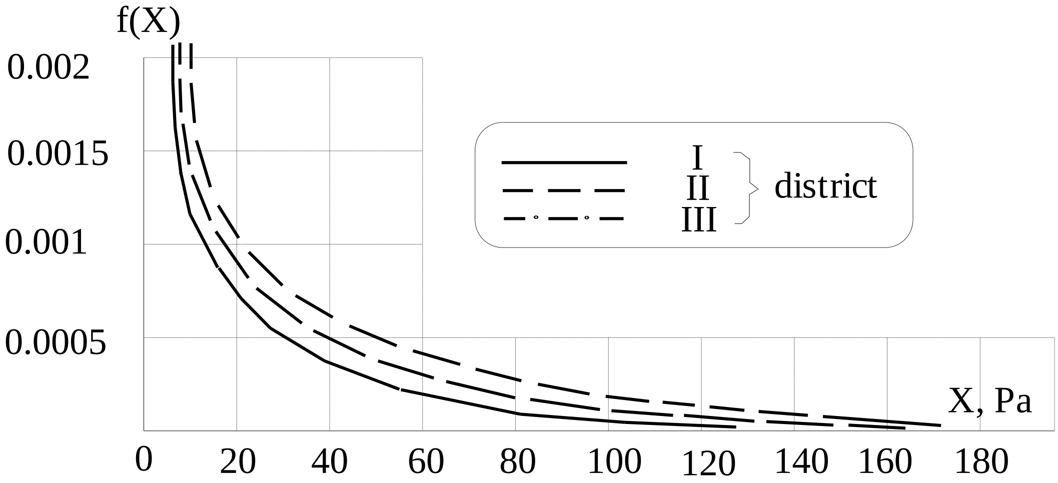

Fig. 8.2. Veibull's law for the wind ordinate (January)

Its integral and differential functions are usually written in the form of:

$$F(X) = 1 - \exp\left[-(X-\varepsilon)^{\beta}/\alpha\right]; \tag{8.1}$$

$$f(X) = (\beta/\alpha)(X-\varepsilon)^{\beta-1}\exp\left[-(X-\varepsilon)^{\beta}/\alpha\right],$$

where ε – parameter of a position of the distribution, when $X \geq 0$ $\varepsilon = 0$; α and β – the parameters of a scale and the form of distribution.

Let us dwell upon the normalized Veibull's distribution. The standardized variable $\gamma=(X-X)/X$, the expression $X=X(\gamma V+1)$ will be used but α should be neglected:

$$F(X) = 1 - \exp\left\{-\left[(\gamma V + 1)\Gamma\left(1 + \beta^{-1}\right)\right]^{\beta}\right\}; \tag{8.2}$$

$$f(\gamma) = \beta V \Gamma\left(1 + \beta^{-1}\right)^{\beta} (\gamma V + 1)^{\beta - 1} \exp\left\{-\left[(\gamma V + 1)\Gamma\left(1 + \beta^{-1}\right)\right]^{\beta}\right\},$$

where $\Gamma\left(1 + \beta^{-1}\right)$ – gamma-function.

As the coefficient of variation V remains constant the parameters of β and gamma-functions are not changeable. So the approximate normalized presentation in the form of equation (8.2) remains unchangeable per year.

The analysis of the received results has demonstrated that the wind load is of a quasi-stationary origin with the stationary not only on frequency but on the normalized distribution of ordinate.

8.2. Probabilistic models of the wind load

The systematic analysis of five most commonly used probabilistic presentations of random loads with the account of unspecified and normal laws of distribution were introduced in our previous work [2]. This article gives the general analysis, which is applied to the wind load described with Veibull's distribution, partially these results are represented in work [1]. For this model the solutions of the direct problem of the calculation of the wind level load $\gamma(t)$ corresponding to the given probability $Q(t)$ and opposite problem of the determination of $Q(t)$ of exceeding γ-level during t were obtained. The evident advantage of all these solutions is the obvious account of a time factor "*t*".

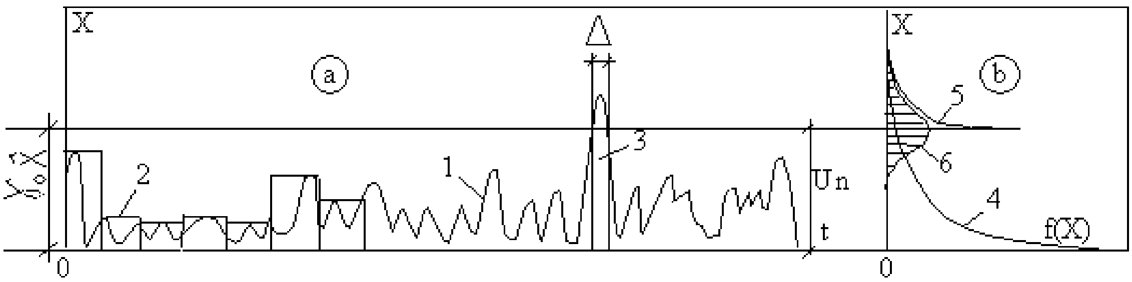

Fig. 8.3. Probabilistic models of the wind load:
a – realization of load: 1 – random process; 2 – random sequence of independent loads; 3 – discrete presentation; b – density distribution: 4 –process ordinate; 5 – absolute maxima; 6 –extremes.

The main probabilistic model of wind load is the model presented in the form of a quasi-stationary random process, it is illustrated in *Fig. 8.3,* position 1. The number of outliers of this process gives the estimation *Q(t):*

$$Q(t) \cong N(\gamma, 0 \le \tau \le \mathrm{t}) = 0.4\,\omega\, t\, \mathrm{K}_{\mathrm{tr}} \beta \sqrt{V} \left[\Gamma(1+\beta^{-1})^{\beta}\right] (\gamma V+1)^{\beta-0.5} \times$$
$$\times \exp\left\{-\left[(\gamma V+1)\Gamma\left(1+\beta^{-1}\right)\right]^{\beta}\right\}, \tag{8.3}$$

where K_{tr} – the coefficient accounts the load wind trend during the year (see p. 8.6).

For the simplification of a design computation the parameter δ is introduced, which numeric values are in the intervals of 0.54 – 0.66 and can be determined as follows (see p. 8.7):

$$\delta = -\ln\left[0.4\beta\sqrt{V}\Gamma\left(1+\beta^{-1}\right)\right] - (\beta-0.5)\ln(\gamma V+1). \tag{8.4}$$

As a result, the equation (8.3) will be transformed into

$$Q(t) = \omega\, t\, K_{tr} \exp\left\{-\left[(\gamma V+1)\Gamma\left(1+\beta^{-1}\right)\right]^{\beta} - \delta\right\}. \tag{8.5}$$

The model of the absolute maxima of random process treats the largest wind loads as random values. These loads are higher than characteristical maximum level γ_0 [5] as it's shown in *Fig. 8.3*, position 5. Its density distribution differs from Veibull's one:

$$f(\gamma) = \left(\frac{K_{tr}}{K_{tro}}\right)\left[\frac{V}{(\gamma V+1)}\right]\left\{\beta\left[\Gamma(1+\beta^{-1})(\gamma V+1)\right]^{\beta} - \beta + 0.5\right\} \times$$
$$\times \exp\left\{\Gamma\left(1+\beta^{-1}\right)\left[(\gamma_0 V+1)^{\beta} - (\gamma V+1)^{\beta}\right] + \delta_0 - \delta\right\}. \tag{8.6}$$

In this case the solution of a direct problem is as follows:

$$\gamma = \left\{\left[(\gamma_o V+1)^{\beta} + \left(\ln\left\{\frac{K_{tr}}{Q(t)K_{tr}}\right\}\right) - \delta_o + \delta\right]^{1/\beta} - 1\right\}\mathrm{V}^{-1}, \tag{8.7}$$

where K_{tro}, δ_0 – refer to γ_0 loadlevel.

Very often the wind loading is introduced in the form of a random sequence of independent loads as it's shown in *Fig. 8.3*, position 2. In this case the frequent parameter is the intensity λ, which equals the number of a load in per-unit time t_λ. The possibility of overloading of γ is:

$$Q(t) = \lambda t \exp\left\{-\left[(\gamma V+1)\Gamma\left(1+\beta^{-1}\right)\right]^{\beta}\right\}. \tag{8.8}$$

The probabilistic model of the wind load in the form of discrete presentation is depicted in *Fig. 8.3*, position 3. It uses the time parameter-mean duration of overloading Δ, connected with the intensity by the ratio $\Delta = t_\lambda/\lambda$; in accordance with it the direct and opposite problems are solved like:

$$\gamma(t) = \left[\left(\ln\{ t / [Q(t)\bar{\Delta}] \} \right)^{1/\beta} \Gamma\left(1+\beta^{-1}\right) - 1 \right] \mathrm{V}^{-1}; \tag{8.9}$$

$$Q(t) = \left(t/\Delta \right) \exp\left\{ -\left[(\gamma V + 1)\Gamma\left(1+\beta^{-1}\right) \right]^{\beta} \right\}. \tag{8.10}$$

In order to adopt the statistics of extremes [6], it's necessary to confirm that the wind load values samples which as presented in accordance with Veibull's distribution correspond to the exponential type. In this case the following condition should be satisfied:

$$\frac{f(X)}{[1-F(X)]} = -\frac{f'(X)}{f(X)}. \tag{8.11}$$

If the Veibull's distribution function is performed in the form of (8.1) when $\varepsilon=0$ the left part of equation (8.11) will be written as follows:

$$\frac{(\beta/\alpha) X^{\beta-1} \exp\left(-X^{\beta}/\alpha\right)}{\exp\left(-X^{\beta}/\alpha\right)} = \frac{\beta}{\alpha X^{\beta-1}}.$$

The derivative of Veibull's distribution density equals:

$$f'(X) = \left(\beta/\alpha\right) \exp\left(-X^{\beta}/\alpha\right) X^{2(\beta-1)} \left[(\beta-1) X^{-\beta} - \beta/\alpha \right]$$

The first term in square brackets asymptotically approaches to zero with X growth. The right part of the condition (8.11) can be written like:

$$\frac{-\left(\beta/\alpha\right)^2 \exp\left(-X^{\beta}/\alpha\right) X^{2(\beta-1)}}{\left(\beta/\alpha\right) X^{\beta-1} \exp\left(-X^{\beta}/\alpha\right)} = -\frac{\beta}{\alpha\beta^{-1}}.$$

Thus, the exponential condition of Veibull's sampling asymptotically performs. So the wind maximum values can be described correctly by the double exponential Gumball distribution of the I type [6], which is shown in *Fig. 3*, position 5. In this case the direct problem is solved with the help of the function from the volume of body sampling n_e:

$$\gamma(t) = \left\{ (\ln n_e)^{1/\beta} \left[1 - \ln Q(t) / (\beta \ln n_e) \right] / \left(\Gamma\left(1 + \beta^{-1}\right) - 1 \right) \right\} V^{-1}. \qquad (8.12)$$

The formulae for necessary extreme parameter computation were represented in our paper [1].

8.3. Comparison of probabilistic models of wind load

All the examined models are close to its sense and forms of *Q(t)* evaluation:

$$Q(t) = \omega t K_{tr} f(\gamma)/\sqrt{2\pi} = f(\gamma)/f(\gamma_0) = \lambda t[1 - F(\gamma)] = \\ = t[1 - F(\gamma)]/\overline{\Delta} = 1 - \exp[-\exp(-y)], \qquad (8.13)$$

where *F(γ)* and *f(γ)* are integral and differential functions of a load distribution in accordance with the equation (8.2).

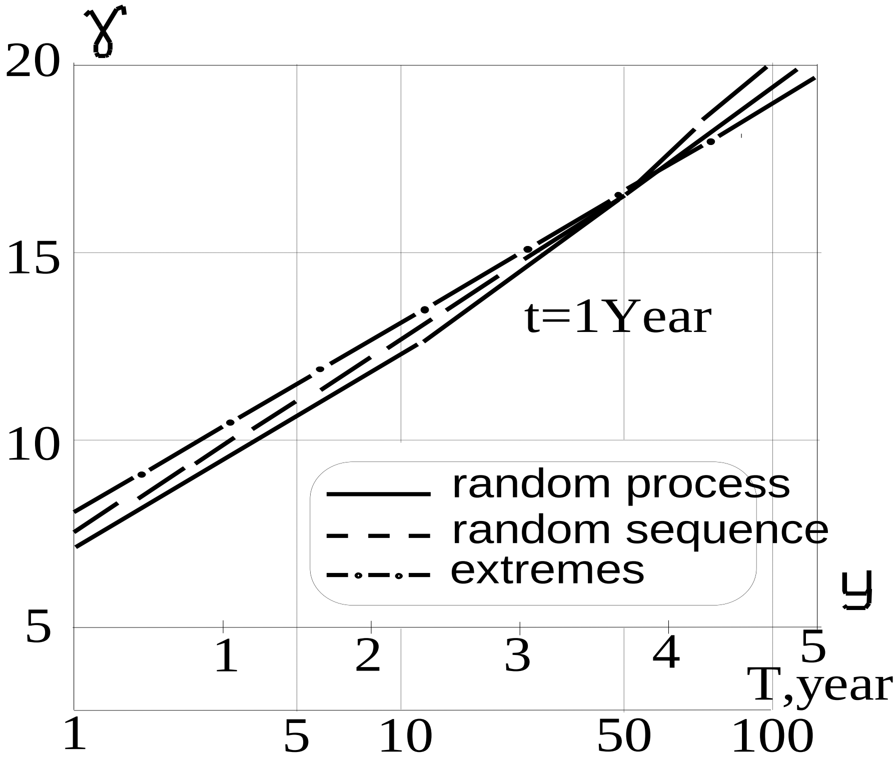

Fig. 8.4. The comparison of probabilistic models of the wind load on the extreme scale

Provided with the equation (8.13) the formulae, which connect the parameters of different probabilistic models of the wind load were derived in the paper [1].

Load probabilistic comparison will be well performed at the extreme scale [6], which is illustrated in *Fig. 8.4.* On the axis of ordinate of a scale the standardized load is laid off, on the axis of abscissas the Gumbel's distribution argument $y = -\ln[-\ln F(t)]$ is laid off which is connected with the load return period *T*.

Gumbel's distribution of the first type is described on the scale in the straight lines form. The random process models, random independent loads sequences and discrete representation are introduced as different curves.

In the case these curves are of a faint concaved character as it's shown in *Fig. 8.4.* The main scale advantage is visual effects of the load distributions tail parts, which have rather small distinctions in the usual presentation forms. It enables to present the visual comparison and different parameters load models correspondation.

8.4. Mean wind parameters of Ukrainian districts

Table 8.1

Probabilistic parameters of wind load

No	Model, parameter	Symbol	Unit	Numeric values for wind districts of Ukraine		
				I	II	III
	Common parameters					
	Mathematic expectation	$\overline{X}$	kPa	8.6	14.6	22.3
	Coefficient of variation	V	-	1.81	1.78	1.73
	Veibull's form parameter	β	-	0.5862	0.5941	0.6078
	Gamma-function	$\Gamma(1+\beta^{-1})$	-	1.5518	1.5243	1.4796
1	**Quasi-stationary random process**					
	Effective frequency	ω	$\frac{1}{24}$ hours	6.58	5.16	5.42
2	**Absolute maxima of random process**					
	Caracteristical maximum level	γ_0	-	8.058	7.327	7.154
3	**Random sequence**					
	Load intensity	λ	$year^{-1}$	650	480	465
4	Discrete presentation					
	Mean duration of overloading	$\overline{\Delta}$	hour	13.55	18.31	18.92
5	**Veibull's extremes per year**					
	Body sampling	n_e	-	977	738	963
	Characteristical extremum	u_n	kPa	0.149	0.230	0.333
	Extreme intensity	α_n	kPa^{-1}	27.1	17.1	12.0

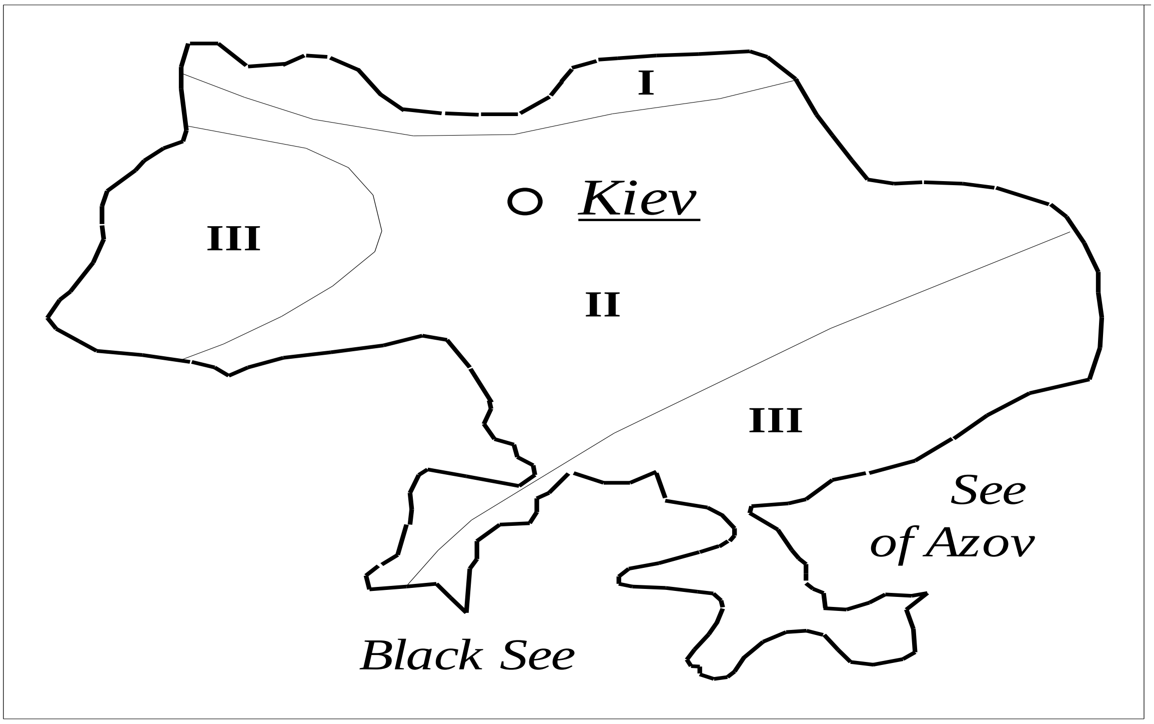

Fig. 8.5. The map of districts of Ukraine according to the wind pressure [7].

In accordance with existing Codes [7] Ukraine is divided into three wind districts as it shown in *Fig. 8.5*. The II wind district occupies the main part of a country, the III district is situated in the south and south-east parts and some part of the Carpathian Mountains, the I district stretches along a narrow part of the north-west board of Ukraine.

The experimental wind probabilistic parameters of all examined models were approximated in accordance with the Ukrainian districts and tabulated in *Table 8.1*. Some of these results were given in work [8]. The values β, $\Gamma\left(1+\beta^{-1}\right)$ and coefficients of variations, which fully determine the average district Veibull's distributions of wind load derived from equation (8.2) are tabulated there. In *Table 8.1* the corresponding frequent characteristics are shown. The numeric values of parameters of different models were derived from the condition of close evaluation *Q(t)* for the working life *t = 50* years. This idea is well illustrated in *Fig. 8.4*.

8.5. Provision of design values of the wind load

The results of the problem are tabulated in *Table 8.2*. They demonstrated that specified and design values of the wind load correspond differently to the experimental statistic data for the Ukrainian territory. For the I-st district the design values are larger then the real ones, for the III district it is just on the contrary: the real loads can exceed the design ones. These data justify the necessity of the further Ukraine wind loads National Codes researches and development.

Table 8.2.

Probabilistic provision of the design wind loads on [7] for Ukraine

Wind district	*Specified load*			*Design load*		
	w_m, Pa	γ_n	T_n, *years*	$w_m\gamma_f, Pa$	γ_f	T_d, *years*
I	*230*	*14.22*	*11.6*	*322*	*20.13*	*86.8*
II	*300*	*10.98*	*5.1*	*420*	*15.60*	*28.2*
III	*380*	*9.27*	*2.5*	*532*	*13.21*	*13.4*

Where: w_m – specified mean wind load; $w_m\gamma_f$ – the same design load; γ_f – load factor; $\gamma_n = (w_m - X)/X$, $\gamma_d = (w_m\gamma_f - X)/X$; T_n, T_d – standard deviation and the return period correspondingly to the specified design load.

The worked out method, which examined the model and obtained parameters gives possibility to estimate reliability of a wide range structures under wind and other loading. Unfortunately, the range of this paper doesn't allow representing all the results. We'd like to point out that the received data show that the reliability differs considerably for structures designed in accordance with the Building Codes [7]. This method also allows specifying some coefficients of design standards.

8.6. Trend coefficient K_{tr}

The character of season change of wind load mathematic expectation $\bar{X}$ for Ukraine is examined in the work [9]. Taking into account the mentioned above facts Weibull distribution for the mean wind load also depends on calendar time t:

$$F(x) = 1 - \exp\left[-x^{\beta}/\alpha(t)\right], \qquad (8.14)$$

where β and $\alpha(t)$ – parameters of a distribution correspondently form and a scale.

From the developed probabilistic wind models, the main one is the representation in the random process form. The formula of the wind random process outliers number is in the form of:

$$\nu_+(x,t) = 0.4\omega[\beta/\alpha(t)]x^{\beta-0.5}[\hat{x}(t)]^{0.5}\exp\left[-x^{\beta}/\alpha(t)\right], \qquad (8.15)$$

where: $\hat{x}(t)$ – wind standard deviation; ω – constant effective frequency.

The estimation of probability of level excess is very important for reliability computation and requires the integration:

$$Q(x,t)=\int_0^t \nu_+(x,t)dt\,. \tag{8.16}$$

For design simplification the trend coefficient is introduced in the form of:

$$K_{tr}=\left[\int_0^{t_1}\nu_+(x,t)dt\right]\Big/\left[\nu_+(x,t_0)t_1\right], \tag{8.17}$$

where: $t_0 = 135$ days. It's a base date in January. It corresponds to the trend top of the mathematic expectation since the conventional date is the 1st of September; t_1 – the design time interval which equals to 1 year.

Let's substitute the numeric integration to the summation of monthly outliers. Then the formula (8.17) will be as follows:

$$K_{tr}=\left\{\sum_{i=1}^{12}(\alpha_0/\alpha_i)^{1-0.5/\beta}\exp\left[-x^{\beta}\left(\alpha_i^{-1}-\alpha_0^{-1}\right)\right]\right\}\Big/12, \tag{8.18}$$

where: α_i and α_0 – Weibull parameters of a scale for i-month and the base one correspondently.

The obtained numeric values K_{tr} for the wind districts of Ukraine, which correspond to Building Codes [5] are tabulated in *Tab. 8.3* in the function of a return period of the wind load T. As it's presented in figure 1-a, the coefficient K_{tr} decreases if T is increased and it changes slightly in the interval $T = 20 \div 50\, years$.

8.7. Auxiliary coefficient δ

For the transition to a normalized Weibull distribution the normalized deviation $\gamma=(x-\bar{x})/\hat{x}$ and the ratio $\bar{x}=\alpha^{1/\beta}\Gamma\left(1+\beta^{-1}\right)$ ($\Gamma\left(1+\beta^{-1}\right)$ – gamma function) were used. Then formula (8.14) is presented in the form of:

$$F(\gamma)=1-\exp\left\{-\left[(\gamma V+1)\Gamma\left(1+\beta^{-1}\right)\right]^{\beta}\right\}. \tag{8.19}$$

The same transformation is applied to formula (8.15) and with the account of K_{tr} for the evaluation (8.16) the formula (8.20) is derived:

$$Q(\gamma,t)=\omega t K_{tr}\exp(-\delta)\exp\left\{-\left[(\gamma V+1)\Gamma\left(1+\beta^{-1}\right)\right]^{\beta}\right\}. \tag{8.20}$$

In this formula the coefficient δ equals:

$$\delta = -\ln\left[0.4\beta V^{0.5}(\gamma V+1)^{\beta-0.5}\Gamma\left(1+\beta^{-1}\right)^{\beta}\right]. \qquad (8.21)$$

All the arguments of this formula are constant except γ. So the numeric values δ for wind districts of Ukraine are in the narrow range of 0.54 – 0.66 for *T=1÷50* years (see *Tab. 8.3* and *Fig. 8.6, b*).

Table 8.3.

Numeric values of coefficients K_{tr}, δ and γ_T

Coefficient	***Wind district of Ukraine***	***Return period of the wind load T, years***						
		1	***2***	***5***	***10***	***20***	***30***	***50***
K_{tr}	*I*	*0.5205*	*0.4994*	*0.4749*	*0.4587*	*0.4434*	*0.4351*	*0.4257*
	II	*0.4920*	*0.4720*	*0.4490*	*0.4340*	*0.4200*	*0.4130*	*0.4040*
	III	*0.4406*	*0.4206*	*0.3980*	*0.3832*	*0.3698*	*0.3627*	*0.3541*
δ	*I*	*0.6591*	*0.6451*	*0.6280*	*0.6164*	*0.6042*	*0.5989*	*0.5918*
	II	*0.6508*	*0.6333*	*0.6144*	*0.6010*	*0.5888*	*0.5821*	*0.5739*
	III	*0.6224*	*0.6041*	*0.5824*	*0.5677*	*0.5540*	*0.5465*	*0.5373*
γ_T	*All districts*	*0.46*	*0.54*	*0.66*	*0.76*	*0.86*	*0.92*	*1.00*

The same transformation is applied to formula (8.15) and with the account of K_{tr} for the evaluation (8.16) the formula (8.20) is derived:

$$Q(\gamma,t) = \omega t K_{tr}\exp(-\delta)\exp\left\{-\left[(\gamma V+1)\Gamma\left(1+\beta^{-1}\right)\right]^{\beta}\right\}. \qquad (8.20)$$

In this formula the coefficient δ equals:

$$\delta = -\ln\left[0.4\beta V^{0.5}(\gamma V+1)^{\beta-0.5}\Gamma\left(1+\beta^{-1}\right)^{\beta}\right]. \qquad (8.21)$$

All the arguments of this formula are constant except γ. So the numeric values δ for wind districts of Ukraine are in the narrow range of 0.54 – 0.66 for *T=1÷50* years (see *Tab. 8.3* and *Fig. 8.6, b*).

The use of coefficients K_{tr} and δ reduces the design and excludes the numeric integration. As a result the closed solution of normalized load level has been derived which corresponds to a chosen excess probability *Q(t)*:

$$\gamma(t) = V^{-1}\left\{\Gamma\left(1+\beta^{-1}\right)^{-1}\left[\ln\left(\omega t K_{tr}/Q(t)\right)-\delta\right]^{1/\beta}-1\right\}. \qquad (8.22)$$

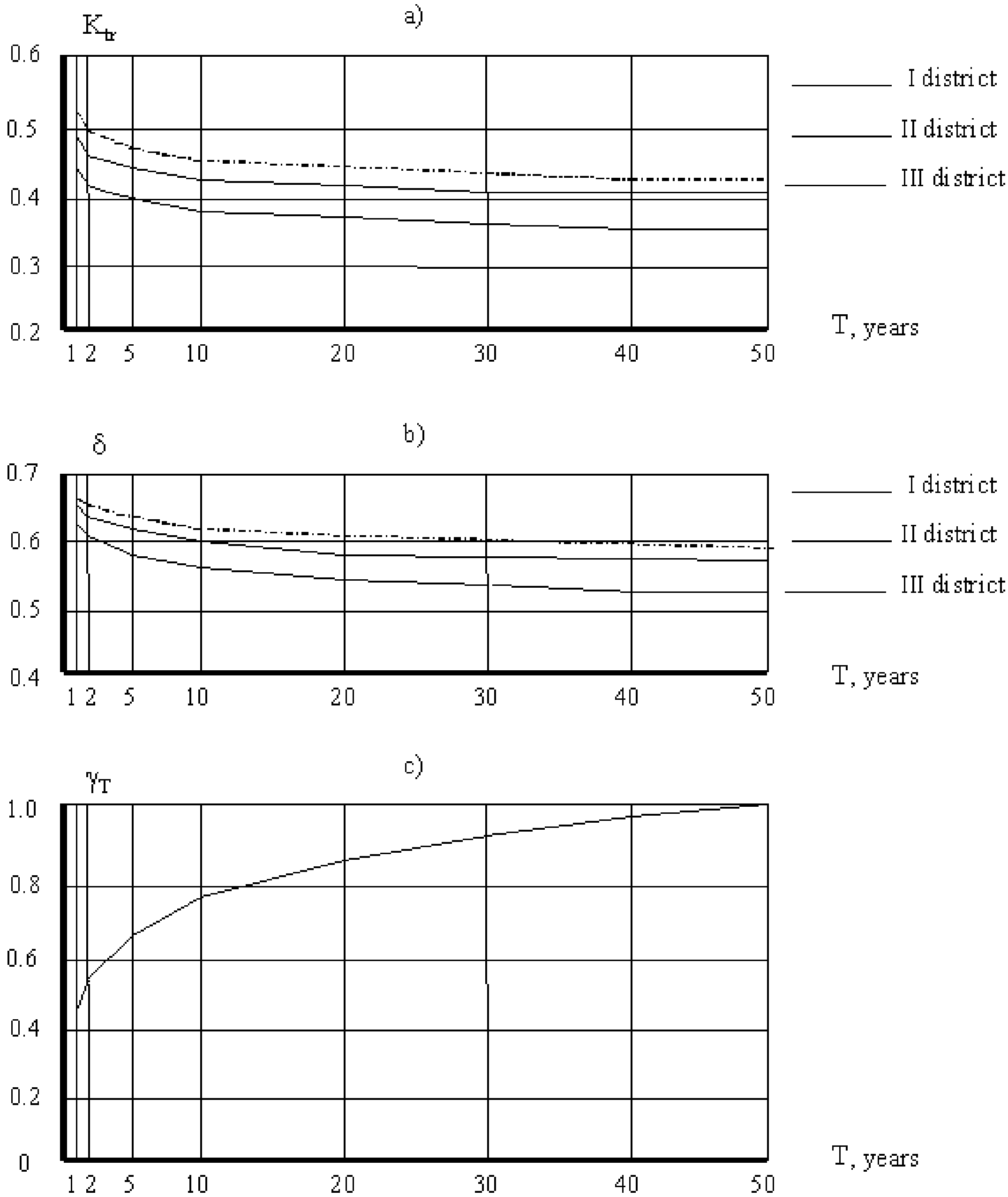

Fig. 8.6. Design coefficients of wind load in the function of its return period: a) trend coefficient K_{tr}; b) auxiliary coefficient δ; c) temporary coefficient γ_T.

Average wind parameters ω, V, β, and $\Gamma\left(1+\beta^{-1}\right)$ for Ukrainian territory are presented in the work [9]. The computation of normalized wind load values $\gamma(T)$ which are connected with the return period T (in years) was significantly simplified with the account of the mentioned above parameters and proposed coefficients K_{tr} and δ. For this purpose the next simplified formula was offered:

$$\gamma(T) = a[\ln(bT)]^c, \tag{8.23}$$

where: *a*, *b* and *c* — numeric coefficients from *Tab. 8.4*. The formula can be applied if $T>20$ years.

Table 8.4

Coefficients of formula (8.23)

Wind district of Ukraine	*a*	*b*	*c*
I	*0.356*	*556.1*	*1.706*
II	*0.369*	*428.5*	*1.684*
III	*0.391*	*409.2*	*1.647*

8.8. Temporary factor γ_T

The obtained results allow computing the mean wind load which corresponds to different return period *T*:

$$w_m(T) = \bar{x}[1 + \gamma(T)V], \tag{8.24}$$

where: $\bar{x}$, V — mathematic expectation and wind load variation coefficient represented in [9]; $\gamma(T)$ — standard deviation, derived from formula (8.22) or (8.23).

Temporary factor γ_T determined as (8.25) is recommended for practical application

$$\gamma_T = w_m(T)/w_m(T_0), \tag{8.25}$$

where: $T_0=50$ years — return period in accordance with the Building Code [5] for design load chose.

The corresponding arguments about γ_T for Ukrainian territory are given in *Tab. 8.4* and *Fig, 8.6, c*. Design wind load is multiplied by these coefficients in the computation of time strength of reconstructed members and some other objects with the limited term of serviceability. For the design of structures under erection it's allowed to apply specified wind load multiplied to the reduced coefficient $\gamma_m=0.55$. This recommendation is based on the value γ_T for $T=1$ year.

Conclusions

The large amount of statistics results on wind loads were examined for the territory of Ukraine. The wind load is of quasi-stationary origin with the constant frequent parameters and normalized distribution. For the description of ordinate density Veibull's law and double exponential Gumbel's distribution were used. The most widely spread probabilistic models of wind load were observed. They are as follows: quasi-stationary random process and its absolute maxima, random sequence of independent loads, discrete presentation and extreme model. Having integrated some initial data all necessary mean parameters of mentioned probabilistic wind models were determined for three districts of Ukraine. Proposed design coefficients significantly simplify the computation process and allow obtaining the wind load values in the form of closed ones for different return periods. A derived temporary coefficient γ_T allows increasing the structure economical efficiency under strength, reconstruction and erection. Numeric wind parameter values and design coefficients are developed for Ukraine. General approaches and design formulae are worked out and are of universal character and can be applied to any geographical districts. Probabilistic wind load model was successfully introduced into the computation of usual and specific structures [3,11].

Control questions

1. What's the point of the Veibull's law? What is the main idea of this law?
2. What is the main advantage of the scale probabilistic models of the wind load on the extreme scale?
3. How many are Ukraine districts according to the wind pressure? How are they divided?
4. What is it the auxiliary coefficient? How to calculate it?
5. What is it the temporary factor? How to calculate it?

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Lecture 9 Snow load on building structures

9.1. Snow load probabilistic distinctions
9.2. Snow load as a random process
9.3. Snow load probabilistic models
9.4. Comparison of probabilistic snow models
9.5. Mean snow load parameters of Ukrainian districts
9.6. Provision of design values of ground snow load
9.7. Conclusions

Snow load probabilistic research for Ukraine is of great importance now. This is a problem, which deals with the necessity for developing National Codes of atmospheric loads on building structures as well as the need for reliability design of buildings and structures, preventing structure failures, which routinely occur every winter in Ukraine. Snow loads are described with different probabilistic models by many authors. However, these approaches lack both systematic analysis and model comparison that can cause different results in structure reliability design. This problem has been studied for several years at Poltava State Technical University. The results presented in this lecture are the integral part of a structure reliability estimation method developed through the author's work [6,7].

9.1. Snow load probabilistic distinctions

The results of regular snow measurements for 15 – 40 years at 62 Ukrainian meteorological stations have been taken as the reference statistic material. Ground snow load realisations were obtained, their intervals run 5 or 10 whole days. This statistic analysis has demonstrated some specific characteristics of snow loading in the territory of Ukraine (*Fig. 9.1*).

Snow drifts on the streets | Snow on the roof of the building

Fig. 9.1. Examples of snowfall

9.1.1. Snow season cycle. During winter, snow load has two little transitional irregular parts. The beginning of winter is the period of snow accumulation; the end of winter is the snow melting stage. The main winter period lies between the winter average beginning date t_S and the average end date t_F . The 10th–15th of November is the starting point of winter for Ukraine (t_S) and the 10th of April can be treated as the end of it (t_F). The period $t_W = t_F - t_S$ is the most important stage of stable snow cover. At this time snow load has relatively high values, which are of interest to the structure reliability design. Within this period (t_W) some general rules of snow load can be traced and they will be described later.

9.1.2. Season winter period. Seasonal changes of mathematical expectation $\overline{X}(t)$ and standard $\hat{X}(t)$ of snow load (t – is the time interval which is calculated from 1 September) are of skewness nature and its top corresponds to the middle of February (*Fig. 9.2*). This trend is described approximately as a polynomial of the 3rd degree. Meanwhile, the district coefficients of variation V and relative skewness S can be roughly treated as constants [4]. In this lecture we use the snow loads for different districts of Ukraine in accordance with the Ukrainian Code [3].

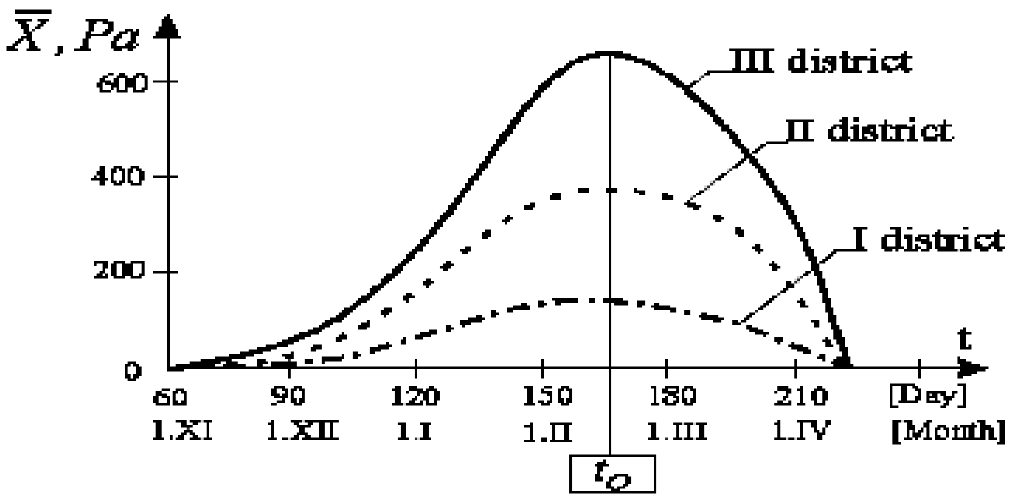

Fig. 9.2. Season change of the snow load mathematical expectation.

9.1.3. Frequent character of snow loads. Stable snow load (in the interval $t_W = t_F - t_S$) is of a stationary frequent character. This conclusion is based on the fact that snow normalized correlation function and effective frequency have no significant distinction during winter.

9.1.4. Distribution of snow load values

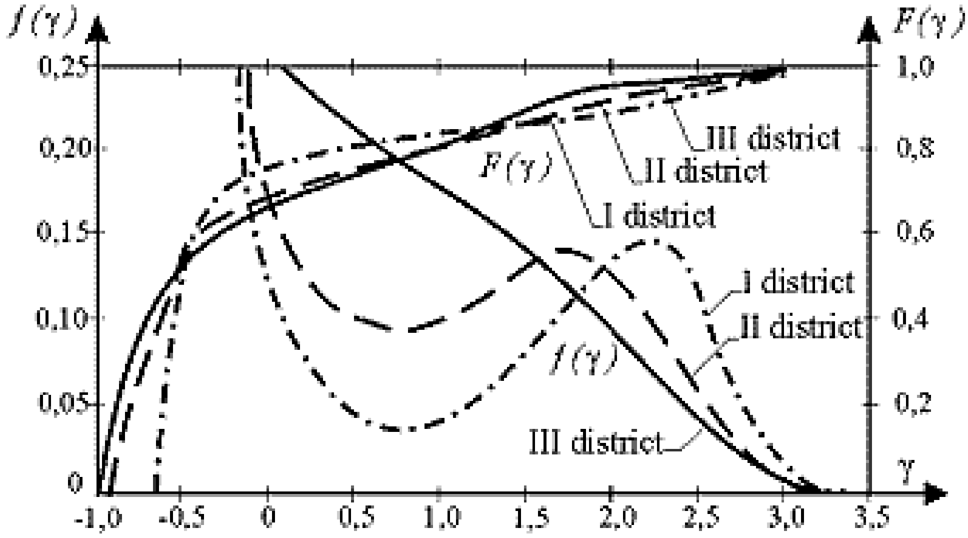

Fig. 9.3. Normalized district snow load distributions

Experimental snow distributions for Ukrainian territory are of bimodal character. Therefore, (*Fig. 9.3*), the normal low that satisfactory describes the snow loads of many snow districts cannot be applied in these cases. That is why so-called polynomial-exponential distribution was substantiated and used. Its normalised presentation is as follows:

$$f(\gamma) = \exp\left(C_0 + C_1\gamma + C_2\gamma^2 + C_3\gamma^3\right), \qquad (9.1)$$

where $\gamma = \left(x - \overline{X}\right) / \hat{X}$ – normalised deviation of snow load.

If V and S are constant the coefficient of exponential index in expression (9.1) remains constant and does not depend upon the date (*t*). As a result, snow load is stationary not only in frequency, but in normalized distribution of ordinate (9.1).

9.1.5. Specific characters of snow polynomial-exponential distribution. This distribution (Fig. 9.3) has an exponential maximum to the left at the original coordinates, which means absence of snow during some winter periods. This is a specific characteristic of changeable Ukrainian winters with little snow. The second top of the curve is determined by the period of stable snow loads. It is interesting to analyse the influence on distribution of coefficients of the polynomial argument in expression (9.1). Term C_0 determines the original ordinate, C_1 determines down - going exponential part, positive coefficient C_2 – existence and height of a curve top. The last negative multiplier, C_3, suppresses the effect of C_2 to the level of γ = (3–3.5) and it makes a curve tag down to the X-axis. Thus, it is not by coincidence that the distribution tags (9.1) are located lower then the normal distribution ones. It is necessary to note that charts of integral functions of snow load distribution $F(\gamma)$, are of more stable and smooth character in comparison with differential functions $f(\gamma)$ (Fig. 9.3).

9.2. Snow load as a random process

9.2.1. Outliers of snow random process. Mentioned above, probabilistic specific characteristics were taken into account while presenting ground snow load in the form of quasi-stationary differentiated random process. Mathematical expectation, $\overline{X}(t)$, and the standard, $\hat{X}(t)$, of the process change during a seasonal cycle just as the effective frequency, ω, and normalized distribution of ordinate (9.1) remain constant. The outlier frequency of this process for the moment (t) of a seasonal cycle can be derived from a stationary process [6]

$$\nu_+(x,t) = \omega f(\gamma)/\sqrt{2\pi} = \omega \exp\left(C_0 + C_1\gamma + C_2\gamma^2 + C_3\gamma^3\right)/\sqrt{2\pi}, \qquad (9.2)$$

where $x = \overline{X}(t)\cdot(1+\gamma \ V)$ is the chosen level of snow load; $\gamma = \left[x - \overline{X}(t)\right]/\left[\overline{X}(t)V\right]$ — normalized load deviation.

Let us estimate the probability of snow load excess of level x per year (opposite problem). It is determined by the quantity of outliers of quasi-stationary random process over that level. It is calculated with the help of the integral expression (9.2) at the interval t_W

$$Q(x, t = 1 year) = N_+(x, t_w) = \int_0^{t_w} \nu_+(x,t)dx. \qquad (9.3)$$

The mean annual curve of snow load outliers for different districts was obtained by summation of numbers of outliers for the fortnight intervals. These curves are presented in *Fig. 3*. They are characterised by sharp peaks at the beginning and long extended parts at the level of N_+ (1 year) = 0.2 – 1.0. These parts are formed by ground snow storage and the snow-melting period during different months. This specific characteristic is typical for rather warm Ukrainian winters. The nature of outlier curves has no simple analytical description, but it is much simpler than the distribution (9.1).

Let us take into account the proportion between the number of outliers per year and return period $T=1/N_+$ (1 year). Than, it is possible to determine the loads of different return periods directly using the outlier curves. These loads are shown on the additional low scale in *Fig. 9.4.* Because of specific characteristics of snow loads an outlier comparatively low normalised snow load value $\gamma = 0.2 - 0.4$ correspond to the return period T (which equals 1 year). These values are located at the beginning of the rise. The transition to T period, which is 5 – 10 years, occurs as the shift along the stretch curves to level with $\gamma = 1.5 - 2.7$. The loads of higher return periods (T = 20 – 50 years) are closely grouped, located on descending outlier curve parts $\gamma = 2.5 - 3.0$.

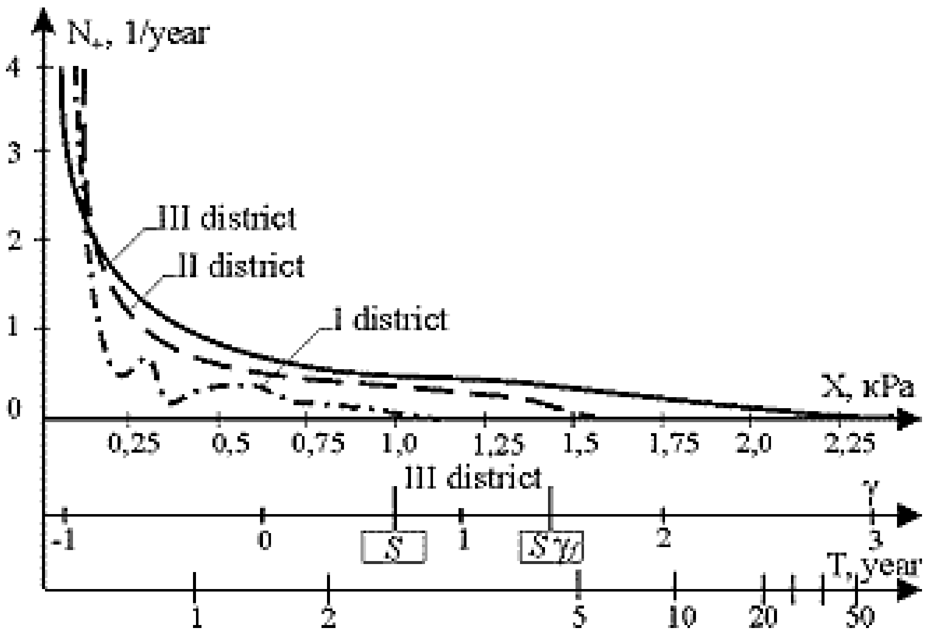

Fig. 9.4. Number of snow load outliers (per year)

***9.2.*2. Trend coefficient.** The trend coefficient, K_{tr}, was introduced to simplify computation and to get closed decisions. This coefficient equals the proportion of numbers of quasi-stationary random process outliers during the t_W period to the number of outliers during the same stationery process period. It corresponds to the trend top for t_0=165 whole days:

$$K_{tr} = \frac{\int_0^{t_w} \nu_+(x,t)\,dt}{\nu_+(x,t_0)t_w}. \qquad (9.4)$$

Fig. 9.5 illustrates the district values K_{tr}. They are in the intervals between 0.13 – 0.70 for T=1–50 years. If $T>10$ years, they slowly descend.

Instead of expression (9.3) for probabilistic description of snow load (inverse problem), K_{tr} application allows one to use the dependence for stationary process in the form of

$$Q(x,t) = \frac{t t_w K_{tr}}{\sqrt{2\pi}} \exp\left(C_0 + C_1\gamma + C_2\gamma^2 + C_3\gamma^3\right), \qquad (9.5)$$

where $\gamma = \left[x - \bar{X}(t_0)\right] / \left[\hat{X}(t_0)\right]$; t — is the serviceability term in years.

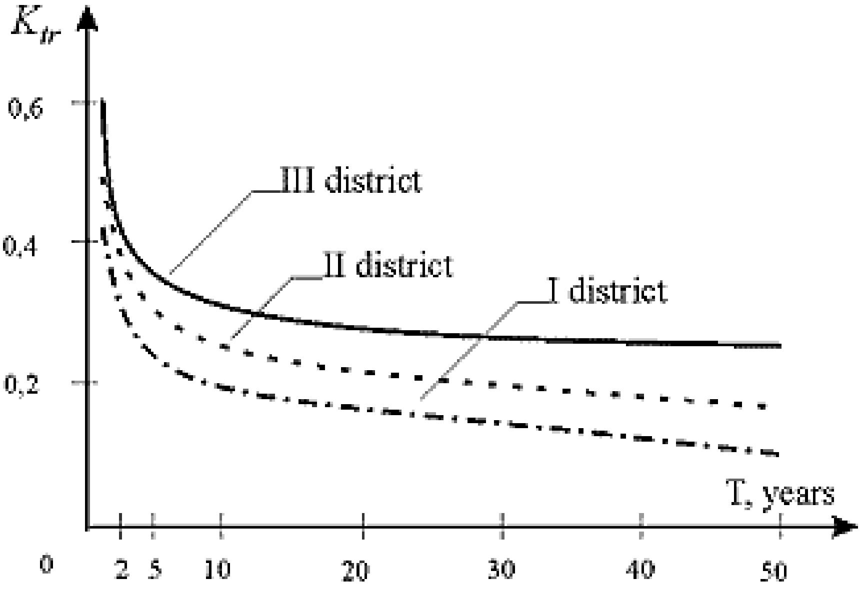

Fig. 9.5. Trend coefficient of snow load

This approach allows, one to derive the normalised level of snow load γ (from (9.5). It corresponds to a given probability of exceeding [*Q(t)*] (direct problem). This decision is a root of a cube equation

$$C_1\gamma + C_2\gamma^2 + C_3\gamma^3 + \left\{ C_0 + \ln\left[\frac{tt_w K_{tr}\,\omega}{Q(t)\sqrt{2\pi}} \right] \right\} = 0. \qquad (9.6)$$

9.3. Snow load probabilistic models

9.3.1. Absolute maxima of a random snow process. As mentioned above, models in the form of quasi-stationary random process fully characterise ground snow load, but it takes a lot of initial statistical information which is difficult to access. The model, in the form of random snow load maxima, is more laconic and accessible. Distribution of maxima determined by the tag part of this random process is located higher than the level of characteristical normalised maximum γ_0. The letter is derived from the equation $N_+(\gamma_0; 0 \le \tau \le t) = 1$, where $N_+(\bullet)$ — the number of outliers of random process [1]. This distribution for absolute snow maxima has the integral and differential function:

$$F(\gamma, \gamma_0) = 1 - \frac{K_{tr}}{K_{tr0}} \exp\left[C_1(\gamma - \gamma_0) + C_2\left(\gamma^2 - {\gamma_0}^2\right) + C_3\left(\gamma^3 - {\gamma_0}^3\right)\right], \qquad (9.7)$$

$$f(\gamma;\gamma_0) = \frac{K_{tr}}{K_{tr0}}\left(C_1 + 2C_2\gamma + 3C_3\gamma^2\right) \times \exp\left[C_1(\gamma - \gamma_0) + C_2\left(\gamma^2 - {\gamma_0}^2\right) + C_3\left(\gamma^3 - {\gamma_0}^3\right)\right], (9.8)$$

where K_{tr} and K_{tr0} are trend coefficients corresponding to the γ and γ_0 levels. It's easy to see that the distribution (9.8) is normalised (neglecting K_{tr}/K_{tr0}). The example of a distribution like that is given in *Fig. 9.6.*

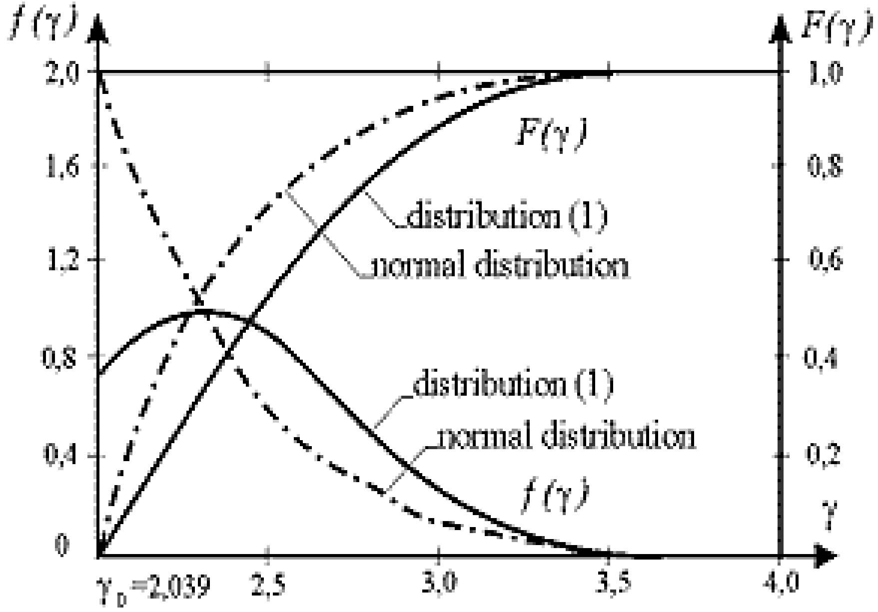

Fig. 9.6. Absolute maxima of snow load random process (II district, *t*=5 years)

9.3.2. Random sequence and discrete presentation on snow load. Snow load is presented rather often as a random sequence of independent loads. This model is sometimes called a scheme of independent tests. The load intensity, $\lambda = N / t_\lambda$, is the frequent characteristic here, N is the number of loads; t_λ is the chosen time interval. The answer to inverse problem is derived from the use of the integral function $F(\gamma)$

$$Q(\gamma,t) = \lambda t[1 - F(\gamma)] = \lambda t \int_{\gamma}^{\infty} \exp\left(C_0 + C_1\gamma + C_2\gamma^2 + C_3\gamma^3\right) d\gamma . \qquad (9.9)$$

Direct problem can be reduced to the solution of a cube equation

$$C_0 + C_1\gamma + C_2\gamma^2 + C_3\gamma^3 + \ln\left[\frac{\lambda t}{\mu(\gamma) Q(t)}\right] = 0 , \qquad (9.10)$$

where $\mu(\gamma) = f(\gamma)/[1 - F(\gamma)]$ – distribution intensity of a snow load.

Applying to snow load, this model advantage is the possibility to obtain a priori λ parameter (for example for month or annual intervals) and relative access of meteorological data in the intervals mentioned above. Besides, this integral function $F(\gamma)$ is of a smoother character and is more suitable for computation in comparison to the differential function $f(\gamma)$ (*Fig. 9.3*). Discrete presentation of snow load is of the same form, its frequent parameter is mean duration of overloading $\overline{\Delta}(\gamma)$. In this case the design formula are derived from (9.9) and (9.10) with the help of substitution $\lambda = \overline{\Delta}^{-1}$.

9.3.3. Snow extremes. This presentation is widely used and it describes well annual snow maxima. Gumball's well known distribution of the I-st type is used [2]

$$F(y) = \exp[-\exp(-y)]; y = \alpha_n(\gamma - u_n), \tag{9.11}$$

where u_n and α_n are the characteristically extremum and experimental intensity correspondingly, which depend upon body sampling n_e.

Let us check the possibility of applying experimental presentation for maxima from snow load samples, subjected to polynomial and exponential distribution (9.1). With this purpose, we will check the validity of condition:

$$\mu(\gamma) = f(\gamma)/[1 - F(\gamma)] = \mu'(\gamma) = -f'(\gamma)/f(\gamma).$$

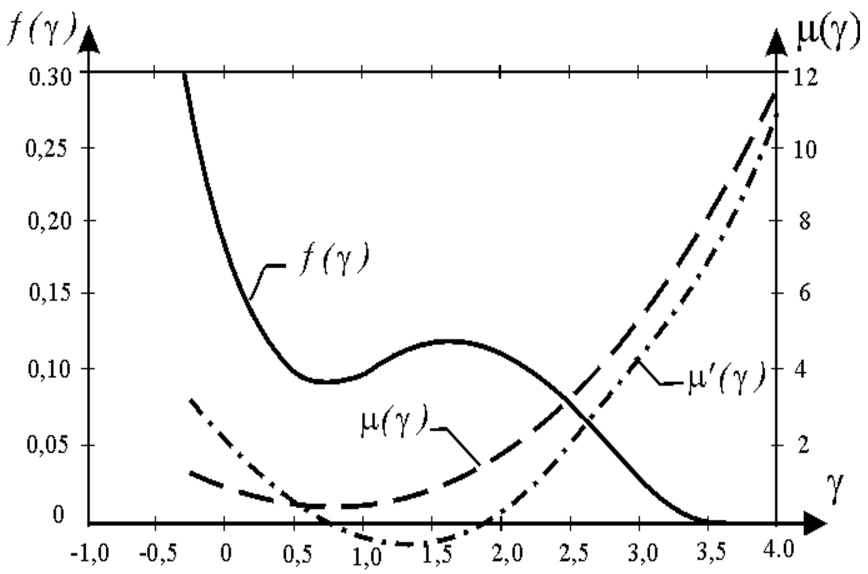

Fig. 9.7. Estimation of the exponential condition for snow body sampling (II district)

Its left part is evaluated numerically; the right part is of closed type $C_1 + 2C_2\gamma + 3C_3\gamma^2$. As it is possible to see in *Fig. 9.7* the given condition is carried out from the level $(3.5 - 4)\gamma$. Thus, snow load sample is of an exponential type and the extremum use is quite correct in that case.

For selection procedure simplification of u_n let's connect general extreme proportions with snow load distribution

$$Q(u_n) = n_e^{-1} = f(u_n)/\mu(u_n) = \exp\left(C_0 + C_1 u_n + C_2 u_n^2 + C_3 u_n^3\right)/\mu(u_n). \quad (9.12)$$

Cube equation for u_n estimation is derived from (9.12)

$$C_0 + C_1 u_n + C_2 u_n^2 + C_3 u_n^3 + \ln\left[n_e / \mu(u_n)\right] = 0. \quad (9.13)$$

If relation $u_n = \gamma - y/\alpha_n$ is substituted into (9.12) we will obtain expression (9.14) for the body sampling (by α_n).

$$n_e = \alpha_n \exp\left[-C_0 - C_1(\gamma - y/\alpha_n) - C_2(\gamma - y/\alpha_n)^2 - C_3(\gamma - y/\alpha_n)^3\right]. \quad (9.14)$$

Another expression for n_e is derived from (9.12) by u_n

$$n_e = \mu(u_n) \ \exp\left(-C_0 + C_1 u_n - C_2 u_n^2 - C_3 u_n^3\right). \quad (9.15)$$

9.4. Comparison of probabilistic snow models

The connection of models in the form of random process, random sequence, and discrete presentation is described as follows

$$\lambda = \Delta^{-1} = \omega\mu(\gamma) K_{tr} / \sqrt{2\pi}. \quad (9.16)$$

The selection of corresponding extreme parameters can be performed numerically or with the help of extreme scale (*Fig. 9.8*). Normalized deviation of γ load is graphed on the ordinate axis and distribution argument (9.11) $y = -\ln\left[-\ln F(t)\right]$ or corresponding probabilities *Q(t)* of γ level exceeding connected with the return period of *T* load is graphed on the abscissa axis. Random process transition to the scale (*Fig. 9.8*) is carried out by annual outliers, the number of which equals $N_+(\gamma, 1year)$

$$y = -\ln N_+(\gamma, 1year). \quad (9.17)$$

As it can be seen in *Fig. 9,8*, a random process chart is of a complicated and irregular character if *T*<5 years. The curves which illustrate random sequences and discrete presentations are of a smoother character and approach to random process curves at the top quite closely if $T \geq 5$ years. Straight lines in *Fig. 9,8* show gumball's extremes. The analysis demonstrated that the most suitable snow extremes are from the 10-year samples. The corresponding line is tangent to the random process curve at point T = 10 years, it exceeds, insignificantly, snow loads for T = 10 – 50 years and it is higher than the irregular part of the curve for T < 10 years, which is of no importance.

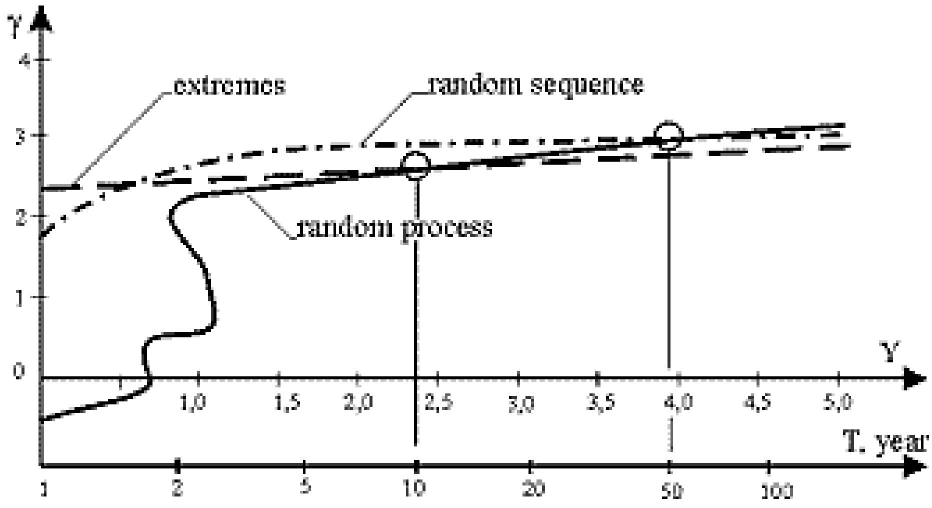

Fig. 9.8. Exposition of snow load models at the extreme scale

9.5. Mean snow load parameters of Ukrainian districts

All necessary mean snow load parameters were determined, for the mentioned above probabilistic models of ground snow loads, for districts of Ukraine (*Table. 9.1*). In accordance with existing Codes [3] the territory of Ukraine can be divided into three snow districts. The first district stretches along southern and western Ukrainian parts; the second one is situated in the northern and western parts of Ukraine and the third one is the narrow area in the northeast territory. Analytical expressions, numeric methods, and extreme scales (*Fig. 9.8*) were used. Snow load probabilistic model parameters were determined on the basis of equality of normalized load level γ (*T=50* years) (*Fig. 9.8*).

There are, in *Table 9.1*, the largest values of mathematical expectation for all examined snow load models corresponding to the top of a seasonal trend, coefficient of variations and coefficients of polynomial-exponential distribution of the ordinate (9.1). In addition, some particular characteristics for every model are tabulated there (*Table 9.1*). These data allow one to give a full description of every possible snow model. It is necessary to pay attention to the fact that all the models give close results in normalising snow load and structure reliability estimation. The author describes wind load in the same way [8].

Table 9.1.

Probabilistic parameters of snow load

Model, parameter, symbol, unit	***Numeric values for snow districts of Ukraine***		
	I	***II***	***III***
Common parameters			
Mathematics expectation, $\overline{X}$*, kPa*	*0.164*	*0.344*	*0.631*
Coefficient of variation, V	*1.60*	*1.26*	*0.920*
Coefficients of equation (1)			
C_0	*-2.265*	*-1.736*	*-1.313*
C_1	*-3.885*	*-1.926*	*-0.725*
C_2	*3.855*	*1.885*	*0.445*
C_3	*-0.920*	*-0.506*	*-0.178*
Quasi-stationary random process			
Effective frequency ω*, 1/24 hours*	*0.141*	*0.095*	*0.073*
Absolute maxima of random process			
Characteristically maximum level, γ_0	*2.688*	*2.459*	*2.113*
Random sequence			
Load intensity, λ*, year*$^{-1}$	*7.90*	*4.86*	*3.20*
Discrete presentation			
Mean duration of overloading, $\overline{\Delta}$*, hour*	*13.55*	*18.31*	*18.92*
Extremes per year / per 10 years			
Body sampling, n_e	$\frac{7.14}{71.4}$	$\frac{3.98}{40.0}$	$\frac{2.42}{24.2}$
Characteristically extremum, u_n***, kPa***	$\frac{0.738}{0.86}$	$\frac{1.074}{1.409}$	$\frac{1.163}{1.858}$
Extreme intensity, α_n*, kPa*$^{-1}$	*17.883*	*6.860*	*3.310*

9.6. Provision of design values of ground snow load

Generalised snow load parameters are tabulated in table 1 and they make it possible to vary the use of probabilistic structure design. In particular, using the obtained results, it was possible to estimate quantitatively the existing Codes of snow load for Ukrainian territory. The obtained results are given in *Table 9.2* where S – is normative snow load; $S_{\gamma f}$ – design snow load; γ_f – load factor; $\gamma_n = \left(S - \overline{X}\right)/\,\hat{X}$ –standard deviation of normative load; $\gamma_d = \left(S \quad \gamma_f - \overline{X}\right)/\,\hat{X}$ – the same of design load; T_n, T_d – return period correspondingly to the normative and design snow load.

The data given in *Table 9.2* demonstrate that normative and design snow loads are of short return periods like T= 2.53 – 3.85 years. It justifies that design loads in accordance with the Codes [3] are not ensured enough and are much lower than real ground snow loads for Ukrainian territories. Everything mentioned above gives evidence that the development of a National Ukrainian Snow Load Code is an urgent task. This fact was pointed out by the author in his previous works [5]. As a temporary measure the increasing of snow load factors can be proposed i.e. from γ_f =1.4 and 1.6 to γ_f = 2.4 –3.0.

Table 9.2.

Probabilistic provision of the design snow load for Ukraine

Load value	***Snow district***	***S, Pa***	γ_n	***T_n, years***
Normative load	*I*	*500*	*1.29*	*2.72*
	II	*700*	*0.82*	*2.94*
	III	*1000*	*0.64*	*2.53*
Design load		$S_{\gamma f}$, ***Pa***	γ_d	***T_d, years***
	I	*700*	*2.06*	*2.43*
	II	*980*	*1.47*	*3.15*
	III	*1400*	*1.26*	*3.85*

Conclusions

Ground snow load is of a quasi-stationary origin. Its mathematical expectation and standard have a seasonal trend. At the same time snow frequent characteristics and normalised ordinate distribution remain constant during the season. Different probabilistic models can describe Snow load faithfully: quasi-stationery differentiated random process with its absolute maxima, random sequence, discrete presentation, and extremes. System comparative analysis of these snow load models was performed, and parameters for Ukrainian districts were validated. These results can be used for practical design of structure reliability. It is substantiated that existing Codes considerably underestimate real snow loads and they are badly in need of updating.

Control questions

1. What is the character of the experimental snow distributions for Ukrainian territory?
2. What is the specific characters of snow polynomial-exponential distribution?
3. What designation does the trend coefficient have and what its introduction?
4. How is it possible to describe the connection of models in the form of random process, random sequence, and discrete presentation?
5. How to determine the probabilistic parameter of snow load?

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LECTURE 10. STATISTICAL DESCRIPTION OF THE COSTRUCTION STEEL STRENGHT

10.1. Problem statement
10.2. Statistical strength characteristics of low-carbon steel St3
10.3. Statistical strength characteristics of low-alloy steels
10.4. Conclusions

10.1. Problem statement

The steel strength is a crucial load-bearing capacity parameter of metal structures. Therefore, a steel strength objective assessment is of great importance for ensuring and calculating the structures reliability and the design standards proper justification. It is known that the smelting steel process is quite complex and not perfectly controlled (high temperature, melting process time, the content of alloying impurities, etc.). Subsequently, during rolling, the metal is compressed, the grains are crushed and their orientation along and across the rolled metal is changed, which affects the mechanical properties of the metal. The properties of steel are also affected by the rolling temperature and subsequent cooling. In addition, with increasing rolling thickness, the mechanical properties of the metal decrease. In the presence of such numerous factors that affect the steel strength, it is natural that the strength indicators have a certain variability, a clear idea that is given by statistical different steel characteristics distribution curves. The yield strength and other mechanical modern steels characteristics have a statistical variance, which is well described by normal law, which has been repeatedly confirmed by test steel samples data. Therefore, the undoubted relevance of regular statistical steel strength studies is associated with the constant design standards revision.

The initial data on the mechanical steel characteristics are obtained because of standard acceptance molten steels samples tests in the metallurgical plants laboratories. The main data purpose is to assess the quality and rejection of substandard metallurgical products. In addition, statistical steels test results are used in the design standards preparation and revision. This process was especially intensified with the introduction of the calculation limit states method [1, 2]. Numerous domestic researchers' publications since the 1940s [3-16] are devoted to the statistical description of the mechanical characteristics of steel, in particular its strength. This problem is actively discussed by foreign experts [17-20]. Reliable statistical parameters of steel strength are especially needed to assess the reliability of metal structures. This is emphasized, in particular, in the publications prepared by the scientific school "Reliability of building structures" of the National University " Yuri Kondratyuk Poltava Polytechnic " [21-24].

Factory tests of steel strength are performed for many years on a large scale, creating a significant array of statistical information. However, there is no common information database for these data. Some of them have been published in various scientific and technical journals, collections of articles, conference proceedings. Access to these publications is difficult, especially since some institutions have begun to destroy paper magazines in recent years, citing the transition to electronic publications. However, in reality, the translation into electronic form has so far occurred only for publications published after 2000.

The lecture contains a systematic publications review in leading scientific and technical journals on the problem of statistical description of the strength of construction steels. The main attention is paid to the selection of statistical steels strength characteristics of different periods, such as mathematical expectation, standard deviation (standard), coefficient of variation, etc. These data are intended for use in numerical calculations of reliability of designs. In addition, the evolution of the norms of design of steel structures is analyzed in terms of changes in the purpose of normative and design resistances and the involvement of experimental statistics.

The content of the lecture is an organized review of publications of such scientific and technical journals as "Industrial and civil construction" (formerly "Construction industry" and "Industrial construction"), "Industrial construction and engineering structures", "Construction mechanics and calculation of structures", "News University. Construction and architecture", "Building materials", "Automatic welding", etc. The review is compiled for the period from the 40s of the twentieth century to the present. The paper version was mainly used for journals published before 2000, which were in the scientific and technical library of the National University "Poltava Polytechnic Yuri Kondratyuk ", one of the most complete book storages in Ukraine. Information on later editions digitized from electronic libraries and electronic versions of journals.

10.2. Statistical strength characteristics of low-carbon steel St3

Statistical studies of the strength of low-carbon steel were initiated before World War II V.V. Kuraev under the leadership of prof. N.S. Streletsky [1]. Based on them, the minimum value of yield strength and allowable stress was determined for steel grade St3 equal to $\left[\sigma\right]$=1600 kg/cm^2.

With the transition in 1955 of the calculation of steel structures to the method of limit states NiTU 121-55 "Standards and technical conditions for the design of steel structures" were introduced. For steel grade St3 in these standards was introduced normative resistance $R^н$ = 2400 kg/cm^2, which was equal to the defective minimum in the acceptance tests of steel samples according to the relevant GOST. The possibility of deviation of strength from the normative resistance in the smaller direction due to the selectivity of control and variability of the size of the rolled product was taken into account by the coefficient of homogeneity $k = 0{,}9$. The design tensile, compression and bending resistance was defined as $R = kR^н = 0{,}9 \cdot 2400 = 2100$ kg/cm^2. The design resistance was equal to the minimum probable value of the yield strength of steel, which was defined as

$$R = \overline{\sigma}_T - 3\hat{\sigma}_T\,, \tag{10.1}$$

where $\overline{\sigma}_T$ and $\hat{\sigma}_T$ – mathematical expectation and standard deviation (standard) of the yield strength.

The design resistance was determined on the basis of statistical processing of 6 thousand results of factory tests of steel grade St3 of different plants [2].

In the 60s of the last century in the metallurgical industry there were significant changes in the production of low-carbon steel: developed oxygen-converter smelting, mastered new deoxidation schemes (semi-quenched steel), increased capacity of open-hearth furnaces, increased ingot weight. This is reflected in the next edition of SNiP II-B.3-62 "Steel structures. Design standards ". They introduced two calculated supports – the yield strength of $R = 2100$ kg/cm^2 (as before) and the temporary resistance of R_p = 2600kg /cm^2. It were separated open-hearth and converter steels, as well as degrees of deoxidation of steel: calm (sp), boiling (kp) and semi-calm (ps).

The mentioned development of metallurgical technology and revision of design norms had a certain influence on the mechanical properties of steels. Therefore, CNDIBK carried out statistical processing of the results of acceptance tests of open-hearth thick steel St3 according to GOST 380-60 with a thickness of 2 – 60 mm at three metallurgical plants: Magnitogorsk Metallurgical Plant (MMK), Kommunarsky Metallurgical Plant and Ilyich Metallurgical Plant (Mariupol) [4]). The obtained results are given in *Table 10.1*. These studies have shown that the mechanical properties of rolled low-carbon steel St3 in these years have decreased significantly (especially in terms of yield strength and toughness). Therefore, it was concluded that the method of acceptance testing of steel at that time (especially the determination of yield strength) needed significant improvement.

With the data given in *Table 10.1* the results of statistical processing of results of mechanical tests of steel VSt3 of various metallurgical enterprises published a little earlier are connected [3]:

- yield strength: $\overline{\sigma}_y$ = 281,0 MPa; $\hat{\sigma}_y$ =23,4 MPa; $\sigma_{y\,\max}$ = 350,0 MPa;
- temporary resistance: $\overline{\sigma}_u$ = 456,4 MPA; $\hat{\sigma}_u$ = 216 MPa; $\sigma_{u\,\max}$ = 520,0 MPa.

It is known that in cases where the results of control tests meet the standards of GOST and TU, the consumer can get a metal with values of strength characteristics below the standard resistances. In the article [5] the probabilistic analysis of these deviations which are considered in norms by coefficient of homogeneity (coefficient of reliability on material) is executed.

Table 10.1

Statistical data on the mechanical characteristics of sheet steel St3

Yield strength				
Steel	*Date, source*	$\bar{\sigma}_y$, *MPa*	$\hat{\sigma}_y$, *MPa*	V_y, *%*
Ct3кp	*1968 p [4]*	*284,1 – 310,7*	*21,9 –25,7*	*7,55*
	1980 p [7]	*266,0*	*29,0*	*10,9*
Cm3ps	*1968 p [4]*	*293,6 – 312,2*	*21,5 – 26,8*	*7,30*
	1980 p[7]	*265,0 – 289,0*	*25,0 – 30,0*	*9,9*
Cm3sp	*1968 p [4]*	*232,6 – 294,0*	*15,9 – 25,9*	*5,8 – 9,1*
	1980 p [г]	*268,0 – 294,0*	*22,0 – 27,0*	*8,7*
Temporary resistance				
Steel	*Date, source*	$\bar{\sigma}_u$, *МПа*	$\hat{\sigma}_u$, *МПа*	V_u, *%*
Cm3кp	*1968 p[4]*	*422,4 – 433,0*	*23,4 – 29,1*	*5,83*
	1980 p [7]	*410,0*	*30,0*	*7,32*
Cm3ps	*1968 p [4]*	*441,8 – 436,0*	*20,6 – 27,1*	*4,75*
	1980 p[7]	*420,0 – 437,0*	*25,0 – 27, 0*	*6,07*
Cm3sp	*1968 p [4]*	*417,0 – 459,0*	*19,2 – 23,4*	*5,54*
	1980 p [7]	*433,0 – 440,0*	*20,0 – 25,0*	*5,15*
Designations: $\bar{\sigma}_y, \hat{\sigma}_y, V_y$ *– accordingly average value, standard, coefficient of variation of yield strength;* $\bar{\sigma}_u, \hat{\sigma}_u, V_u$ *– the same of limit of strength (temporary resistance).*				

In preparation for the next revision of steel structures, in the late 70's CNIDBK conducted a large-scale data processing of 26 thousand acceptance tests of steel St3 [7], the results of which are partially given in *Table 10.1*. They generally correspond to the results of previous tests and confirm a smaller statistical scatter of data on the temporary resistance compared to the yield strength. The resulting array of statistical information was linked to the main provisions of the calculation of steel structures at the limit states. In particular, the estimation of probabilistic provision of normative and design resistances of steel St3 was carried out (*Table 10.2*).

Table 10.2

Probabilistic provision of normative and design resistances of steel St3

Profile	***Steel***	***Normative resistance***		***Design resistance***			
		$P(R_{yn})$	$P(R_{un})$	***after yield strength***		***after limit of strength***	
				γ_y	$P(R_y)$	γ_u	$P(R_u)$
Sheets	*Ct3кp*	*0,893*	*0,841*	*1,94*	*0,974*	*5,00*	≈ 1
	Ct3ps	*0,894-0,991*	*0,929-0,989*	*1,97-3,12*	*0,976-0,9986*	*5,92-7,07*	≈ 1
	Ct3sp	*0,921-0,998*	*0,984-0,996*	*2,15-3,82*	*0,984-0,9999*	*6,96-8,65*	≈ 1
Steel angu-lar	*Ct3кp*	*0,989*	*0,913*	*3,09*	*0,999*	*5,65*	≈ 1
	Cm3ps	*0,999*	*0,985*	*4,05*	*0,99997*	*7,72*	≈ 1
	Ct3sp	*0,999*	*0,993*	*3,92*	*0,9998*	*7,07*	≈ 1
Chan-nels, beams	*Ct3кp*	*0,999*	*0,985*	*4,04*	*0,99997*	*7,95*	≈ 1
	Ct3ps	*0,9999*	*0,9996*	*5,24*	≈ 1	*8,95*	≈ 1
	Ct3sp	*0,9999*	*0,999*	*5,67*	≈ 1	*6,46*	≈ 1
Designations: γ_y, γ_u *– normalized deviations of calculated resistances from average values (safety characteristics)*							

Data analysis of *Table 10.2* allowed substantiating the following conclusions:

• the provision of standard sheet metal resistances up to 10 mm thick made of St3ps and St3kp steels is low, which is explained by a significant share of less strong rolled steel;

• high security of normative resistances R_{yn} and R_{un} of angle steel, channels and beams from steel of the St3ps and St3sp brands;

• the requirement to ensure the values of the normative resistance of building materials with a probability of 0.95 for steel St3 in most cases is met;

• the security of the calculated resistances values after strength limit is higher, for which the security in all cases is close to $P \approx 1{,}00$, and the safety characteristic $\gamma = 5 - 9$;

• the probabilistic provision of design resistances of rolled steel from St3sp and St3ps steels is always higher than the probability of 0.999, with safety characteristics $\gamma = 4 - 6$. Therefore, CNDIBK proposed to increase the design resistance BSt3sp to 230 MPa and BSt3kp to 220 MPa, which was implemented during the revision of design standards.

In the new edition of SNiP II-23-81 "Steel structures" it were introduced for steel St3 two strength groups (at the suggestion of the Institute of Electric Welding named after EA Paton), grades were replaced by classes (steel St3 was assigned to classes C235, C245 and C255 depending on the degree of deoxidation and strength groups), differentiation was introduced depending on the type of rolling (sheet or shaped) and the thickness of the profiles. To move from the normative to the design resistance instead of the coefficient of homogeneity now the coefficient of reliability for the material is used:

$$R_y = R_{yn} / \gamma_m ; \qquad R_u = R_{un} / \gamma_m , \tag{10.2}$$

where R_{yn}, R_{un} – normative resistances, respectively, after the yield strength and temporary resistances; R_y, R_u – similar design resistances.

Substantiated statistically new reliability coefficients on the material differ insignificantly from the unit: $\gamma_m = 1{,}025 - 1{,}100$.

The article of the staff of CNDIBK [8] summed up the results of the first years of implementation of SNiP II-23-81, which led to significant savings in steel in construction. Subsequent editions of the norms of Ukraine DBN B.2.6-198: 2014 "Steel structures. Design standards" and Russia's SP 16.13330.2017"SNiP II-23-81*" do not differ in principle from SNiP II-23-81 in terms of strength rating of steels [16].

Recently, the use of light thin-walled steel structures has expanded. It was found that the cold formation of steel profiles leads to their strengthening. To detect it, the statistical processing of test results of samples of two steels was performed [9]. The obtained strengthening factor is well described by normal law and has the following parameters:

- 14G2 – $\overline{m}$ =1,17, $\hat{m}$ = 0,082, V=6,4%;
- BCt3sp – $\overline{m}$ =1,31, $\hat{m}$ = 0,066, V=5,0%.

10.3. Statistical strength characteristics of low-alloy steels

It is no coincidence that statistical studies of the properties of low-carbon steel of ordinary strength were the most extensive. According to the data from the end of the 1980s, 80% of rolled steel of this type with yield strength of up to 245 MPa was used for the production of building steel structures. Low-alloy steels of high strength with yield strength of 325 –345 MPa were 15%, rolled high-strength steels with yield strength of at least 390 MPa – only 5% [8]. Therefore, it was important to deploy research on strength steels.

Back in the postwar period of 1955 – 1957, the Chelyabinsk Plant of Metal Structures performed large-scale statistical mechanical tests of natural alloy steel NL2 (15HSND) (30 thousand tons), supplied by MMK, Kuznetsk Metallurgical Plant (KMK), Nizhny Tagil Metallurgical Plant and Plant named after Dzerzhinsky [10]. The yield strength distribution of NL2 steel was well described by the normal law with the characteristics of $\overline{\sigma}_y = 382{,}0\,\text{MPa}$; $\hat{\sigma}_y = 27{,}3\,\text{MPa}$. The author of the publication, a well-known specialist BI Belyaev calculated the coefficient of homogeneity K_{cp} = 0.757 according to the author's method, taking into account the minus tolerances on the dimensions of the cross sections of rolled sections, which gave the following value of the design resistance of steel NL2 (15HSND):

$$R = K_{cp}\overline{\sigma}_y = 0{,}767 \cdot 382 \approx 290\,\text{MPa}.$$

Therefore, a reasonable conclusion was made that the design resistance of 290 MPa adopted in the norms of NiTU 121-55 is in full compliance with the actual mechanical properties of NL2 steel. However, BI Belyaev criticized the system of rejecting this steel, because the then rejection minimum of 340 MPa was at a distance of 1.43 of the standard from the average value, which led to the probable rejection of 7.6% of steel. Therefore, the author of the article proposed to accept the rejection minimum at the level of 3 standards, ie 382 - 3 • 27.3 = 300 MPa.

In the mid-1960s, statistical processing of the mechanical tests results of low-alloy construction steels 14G2, 15 HSND, 10 HSND in the amount of 225, 575 and 507 factory tests, respectively, at MMK, NTMZ, KMK, Orsko-Khalilovsky (OHMK) and other metallurgical enterprises was performed [3]. The obtained results are summarized in *Table 10.3*, the data of which for steel 15HSND differ from the previous ones [10] by a higher standard – 34.5 MPa compared to 27.3 MPa at the same average values.

Table 10.3

Statistical data on the mechanical characteristics of low-alloy steels

Steel	*Yield strength σ_y, MPa*			*Temporary resistance σ_u, MPa*		
	$\bar{\sigma}_y$	$\hat{\sigma}_y$	$\sigma_{y\max}$	$\bar{\sigma}_u$	$\hat{\sigma}_u$	$\sigma_{u\max}$
14G2	*398,8*	*36,0*	*510,0*	*552,0*	*38,6*	*670,0*
15XSNДD	*389,2*	*34,5*	*500,0*	*562,4*	*30,0*	*660,0*
10XSNДD	*458,7*	*37,6*	*580,0*	*597,5*	*34,6*	*710,0*

A detailed statistical study of low-alloy steel 10G2S1 was conducted in the late 60's B.Yu. Uvarov at the Ilyich Metallurgical Plant (Mariupol) [11]. Sheets with a thickness of 26 – 119 mm were studied, the number of samples was 1200. The distribution curves of the mechanical characteristics were close to normal with a slight asymmetry. A decrease in the mechanical characteristics of steel with increasing sheet thickness was found. This general trend was described by the following regression equations:

- average value:

$\bar{\sigma}_y = 41{,}3 - 0{,}085\delta$; $\bar{\sigma}_u = 56{,}5 - 0{,}039\delta$; $\bar{\delta}_5 = 27{,}7 - 0{,}019\delta$;

- standard:

$\hat{\sigma}_y = 2{,}67 - 0{,}006\delta$; $\hat{\sigma}_u = 2{,}70 - 0{,}014\delta$; $\hat{\delta}_5 = 2{,}29 - 0{,}007\delta$.

Here the strength σ is in kg/mm^2; thickness δ in mm; relative elongation δ_5 in %.

The standards of yield strength and temporary resistance decrease with increasing thickness due to the alignment of mechanical properties with slow cooling of thicker sheets.

The coefficient of homogeneity was determined in the usual way

$$k = \frac{1 - \gamma\sqrt{V_y^2 + V_f^2}}{1 - \gamma^2 V_f^2},$$

where γ – safety factor (accepted in the norms equal to 3); V_y – coefficient of variation of yield strength; $V_f = 0{,}043$ – coefficient of variation by area.

After substituting the numerical values into the formula, the coefficient of homogeneity was obtained $k = 0{,}79$. The formula for the design resistance was obtained by the regression line equation

$$R = 32{,}6 - 0{,}068\delta.$$

It turned out that with increasing sheet thickness for every 15 mm, the design resistance decreases by 10 MPa. This was taken into account in *Table 10.4*.

Table 10.4

Recommended design resistances of sheet steel

Thickness, mm	***До 38***	***39 –52***	***53 – 68***	***69 – 82***	***83 – 98***	***99 – 110***
R, *MPa*	*300*	*290*	*280*	*270*	*260*	*250*

The recommended division of rolled products into groups narrower than the norms could have some economic effect, but was not fully implemented.

In the early 80's, experts from the Moscow Institute of Civil Engineering (MIBI) conducted statistical studies of high-strength steels [12]. Data on steel 12GN2MFAYU of strength class C70/60 were obtained by the results of acceptance tests at OHMK, the sample size – 4 thousand tests. Sheet metal with a thickness of 12 – 40 mm was tested. The obtained results: the average value of the yield strength $\bar{\sigma}_y$ = 710.4 MPa; temporary resistance $\bar{\sigma}_u$ = 806.4 MPa; average elongation $\bar{\delta}_5$ = 16.11%. Steel within the batch is heterogeneous (327 tested batches): the standard of properties distribution within the batch in the shares of the general distribution standard is: 0.53 after the yield strength and 0.48 after the temporary resistance. The investigated rolled metal meets the requirements for steel of class C 70/60: $\sigma_y \geq 60\,\text{MPa}; \sigma_u \geq 70\,\text{MPa};\ \delta_5 \geq 12\,\%$. According to the test results, high-strength steel grade 12GN2MFAYU can be considered promising for responsible welded metal structures operating under dynamic loads and operated at negative temperatures below -40 ° C.

Statistical analysis of the properties of the new high-strength steel with nitride hardening grade 16G2AF was performed at OHMK on the basis of a sample of 6.5 thousand tests [13]. Sheet metal with a thickness of 10 – 40 mm was tested. Steel in the normalized state had an average value of yield strength $\bar{\sigma}_y$= 470 MPa; temporary resistance $\bar{\sigma}_u$ = 650 MPa. Heat-treated steel had slightly higher characteristics – the average value of yield strength $\bar{\sigma}_y$= 550 MPa; temporary resistance $\bar{\sigma}_u$= 680 MPa. Steel within the batch is heterogeneous (816 batches were tested): the standard of distribution of properties of normalized steel within the batch in the shares of the general distribution standard is: 0.518 after the yield strength and 0.607 after the temporary resistance. It was concluded that the developed steel in terms of both strength and plastic characteristics meets the requirements for high-strength steels.

Recent publications describe new high-strength steels of large thickness [14, 15]. Rolled steels C345, C375, C390 and C440 have high engineering properties and good weldability. Thermomechanical hardened steels of high purity can be referred to the third generation of construction steels and be used in building metal structures of the most responsible and unique structures.

Conclusions

A systematic review of works on the problem of statistical description of the strength of construction steels is realized. The main attention is paid to a sample of statistical characteristics of steels strength of different periods, such as mathematical expectation, standard deviation (standard), coefficient of variation, etc. These data are intended for use in numerical calculations of reliability of structures. The evolution of steel structure design norms is analyzed in terms of changes in the purpose and provision of normative and design resistances and the involvement of experimental statistics.

Control questions

1. How does cold forming of steel profiles affect their strength?
2. What is it the coefficient of homogeneity and what its value?
3. Give definition of the thermomechanical hardened steels of high purity.

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LECTURE 11. CAUSES AND CONSEQUENCES OF STEEL VERTICAL TANKS

11.1. Problem statement
11.2. Background to the creation of statistics on accidents of high-risk construction objects
11.3 Causes and consequences of accidents steel vertical tanks
11.4. Review of past research
11.5. Statistics of accidents steel vertical tanks

11.1. Problem statement

The causes of accidents at high-risk facilities have been studied for dozens of years, but the need to improve accident statistics and data processing methodology is also quite relevant today. The reason for this is a series of accidents, which resulted in the dead and wounded, and the company suffered huge economic losses.

Studying the causes of accidents based on the methodology allows solving the most important practical issues of industrial safety. The identification of hazardous production factors and zones, their impact on residential buildings adjacent to enterprises contributes to the introduction of new safety technologies and the optimization of measures and means of suppressing the development and localization of accidents.

To improve state building codes, more and more attention is paid to accident statistics, a method for predicting a possible accident and progressive destruction. Thus, the processing and analysis of tank accidents also occupies an important position in this topic (*Fig. 11.1*).

Fig. 11.1. Steel vertical tanks

Nowadays, the accidents prediction at the design construction site stage the begins to be introduced into government regulations in developing countries. This fact makes it clear the need to develop an appropriate methodology in this matter. Studying the historical experience of creating algorithms various types, it is necessary in turn to engage in their improvement. The reasons for this are technical and informational construction industry development as a whole, increasing the engineering tasks complexity, updating the architectural forms concept and buildings designations.

The topic of analyzing the reasons for the complete or partial destruction of reservoirs has been relevant for quite a long time; the reason for this is the great responsibility of this type of objects and their increased danger. The list of works devoted to this issue is headed by such scientists as Hanukhov Kh.M. [3], Konovalov P.A., Mangushev R.A. [4,5], Galeev V.B. [6], Rozenshtein I.M. [7], Zemlyansky A.A. [8], Berezin V.L. [9], Belyaev B.I. [10], Tarasenko A.A. [11], Konovalov A.P. [12], Athos P. [13] and others. The analysis of such accidents was carried out with information provided from technical literature, periodicals, personal experience of the authors, Internet sources, etc.

11.2. Background to the creation of statistics on accidents of high-risk construction objects

On Saturday 10th July 1976, the control system of a chemical reactor for the production of trichlorophenol, a component of several herbicides, was damaged, and the temperature rose beyond the limits (*Fig.11.2*).

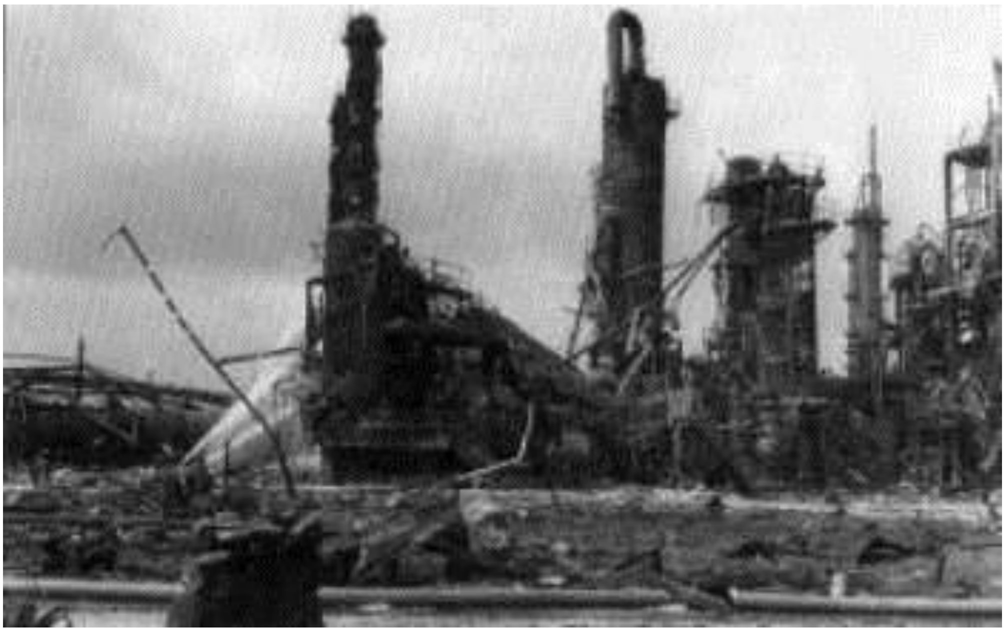

Fig. 11.2. Seveso disaster, Italy, 1976

The explosion of the reactor was avoided by the opening of safety valves, but the high temperature reached had caused a change in the reaction that led to a massive formation of 2,3,7,8-tetrachlorodibenzo-p-dioxin (TCDD), substance commonly known as dioxin, a high toxic compound.

This event became internationally known as the Seveso disaster since Seveso is the name of a neighbouring municipality that was the most severely affected [25].

The catastrophic accident in Seveso (Italy) in 1976 led to the adoption of European Union legislation aimed at preventing accidents in certain industries with the use of hazardous substances and, thus, limiting the impact on workers, the population as a whole and the environment. The resulting standard was Directive 82/501 / EEC [18], better known as Seveso I. This regulatory framework established that a manufacturing company that used in its process hazardous substances listed in Appendix A or stored hazardous substances listed in Appendix B, or both, should develop (among other documents) internal and external protection plans and emergency plans, including a risk assessment.

With the introduction of Seveso I in Europe, more than 130 serious accidents have occurred, and as a result of technological advances, new risks have appeared. So the European Commission introduced Directive 96/82 / EC (called the Seveso II Directive) in 1996. This directive classified risks as “minor”, “low risks” and “high risks” depending on the amount of hazardous substances. Seveso II has been revised in Directive 2012/18 / EU or Seveso III to increase the level of protection for people, property and the environment.

In Spain, in 2016, according to the Directorate General for Civil Defense [19], in accordance with the Seveso Directive there were 422 high-risk facilities and 470 low-risk facilities. The geographical distribution is similar to the distribution of goods turnover: Catalonia was the first (23.9%), Andalusia from 70 (16.6%), the Valencian community of 39 (9.2%) and the Basque Country (6, 6%).

The chemical industry has implemented improvements in process safety and environmental protection through four strategies: a safer design; risk assessment processes; use of instrumental security systems; and the introduction of security management systems. In the risk assessment process, the HAZOP method is the method most used to identify risks [23]. The HAZOP study developed with Imperial Chemical Industries (ICI) as a “critical study” method, formulated in the mid-1960s. A decade later, HAZOP was officially published as a disciplined procedure for identifying deviations in the manufacturing industry by Kletz in 1978 [21], as well as in some publications [22], corporate benefits, standards (IEC 61882 [23]) and national guidelines (Nota Técnica Prevención (NTP) 238 [24]) was developed after.

11.3. Causes and consequences of accidents steel vertical tanks

Speaking about the reliability of steel vertical tanks (*Fig. 11.3*) is provided by the following parameters:

- characteristics of sections of the main bearing structures, properties of steel;
- quality of welded joints;
- tolerances in the manufacture and installation of structural elements.

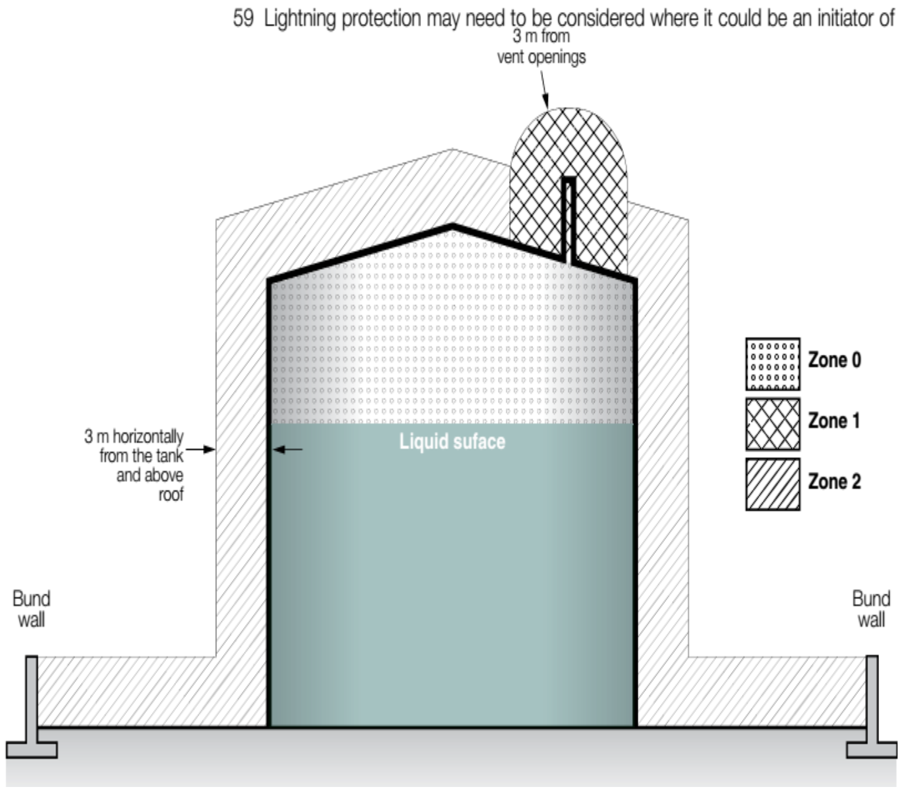

Fig. 11.3. Vertical storage tank – typical hazardous area classification

Meeting all standards and ensuring the reliability of the design with regulatory documents, in our time steel vertical tanks act as one of the most dangerous industrial facilities. This is due to a number of reasons, such as:

- a large length of the welds of the structure, which is quite difficult and difficult to fully control;

- high fire and explosion hazard produced;

- imperfect geometric shape, there are still at the stage reservoir hydrotesting;

- significant movement of the tank wall as in the process operation, and in the process of technological operations;

- high corrosion rate of structural elements;

- low-cycle fatigue of individual zones of the structure;

- the complex nature of the load structure in the zone of the loop joint [1].

Accident reservoirs lead to severe material, environmental and social consequences. Among the main consequences of accidents are the following:

- full or partial destruction of the emergency tank itself, as well as other nearby tanks, buildings and structures;

- soil and water bodies pollution with oil and oil products, as well as air pollution by combustion products;

- injuries and deaths.

According to statistics in extreme cases, material damage from tank accidents is 500 times more than the initial cost of their construction [2].

11.4. Review of past research

Following causes destruction of vertical steel tanks were set on the basis of years of research: direct and indirect. Direct causes include brittle cracks, viscous cracks, pre-rolled steel, defects in welds and uneven sludge. Indirect causes, in turn, include: unsuccessful rolling solutions, unsatisfactory quality of work, poor quality of materials, violation of installation technology and poor quality control of works [14].

Moreover, uneven sediment becomes the main cause of the destruction of tanks of this type. It accounts for 33% of the total number of accidents considered.

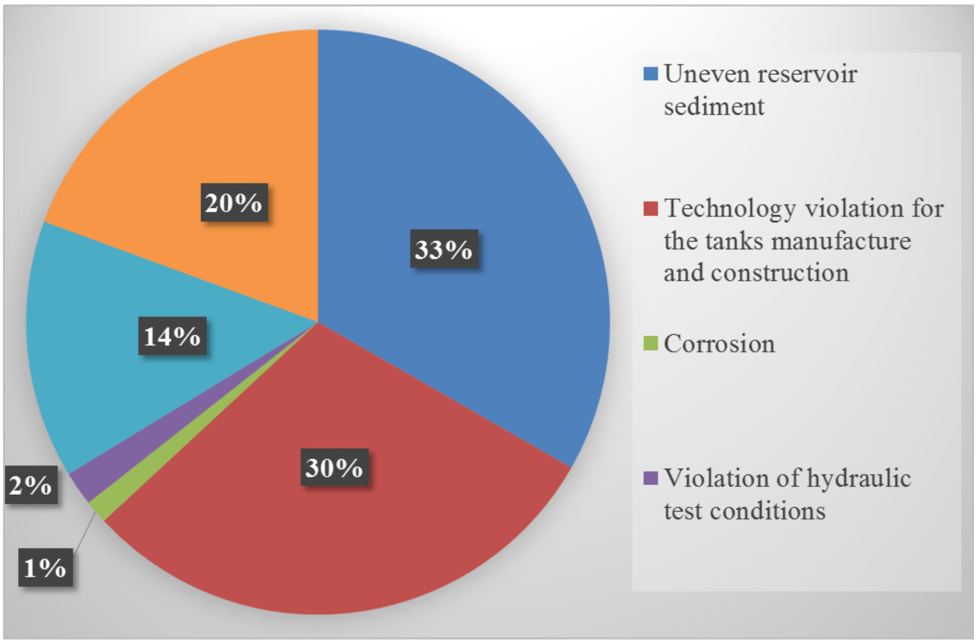

Diagram 1. The main destruction reasons VST

Uneven base sediment is one of the main destruction causes and is distinguished by such global companies as ESSO and Chevron [15].

Such accidents include a 10m long bottom breakdown with a 0.15 m opening to the tank of Mitsubishi Corporation (Japan, 1974), two accidents at an oil depot near London, several accidents at the ESSO tank farm (Foley, England). It is characteristic that at the tank farm in the town of Foley the first accident occurred during their testing (1955). The cause of the tank bottoms destruction was a large local base subsidence [18].

In the early 1970s, three more major accidents occurred with tanks with a diameter of 53 m. Two tanks were filled with water, one - with oil. During testing, one of the damaged tanks received an average draft of 254 mm, and the peripheral draft on the bottom area in a section 2.0 m wide from the wall to the center was 150 mm, while in non-destroyed areas it was 40-50 mm. A detailed examination of acts of investigation of tank accidents over the past 30 years shows that in 38 cases out of 44 there was an uneven settlement of the foundation, which, in combination with other factors, caused the destruction. It is difficult to judge the quantitative ratio of the influence of precipitation and other factors, because there is no true picture of the precipitation of these tanks. However, a number of cases are known when only sediment was the cause of the damage, in one case it was different in sediment size of the tank body and process pipelines, which resulted in the latter breaking off from the wall and subsequent rupture of the latter; in the other - an uneven draft, which reached 320 mm, which led to the rupture of the wall and complete destruction of the reservoir in the third - an uneven draft, which led to the rupture of the bottom. It should be noted that accidents are usually caused by a complex of reasons, one of which is the uneven settlement of certain base areas [3].

Every year the accidents number on tanks increases due to the fact that a large percentage of tanks have already developed their design resources. The wear of the operated vertical steel tanks (RVS) is 60 - 80%. On the basis of a survey by TSNIIPSK [16], it was established that the total accidents number is 3-5 times more than that recorded. The emergencies remains intensity quite high over the last 30 years, about 0.0003 tank destruction per year.

Analysis of the risk of destruction [2] showed that the actual accidents risk is two orders of magnitude higher than the standard value and is $1{,}6 \cdot 10^{-3}$. The accidents danger is estimated by the amount of damage, depends on how the accident manifests itself: in the form of explosions and fires from the spilled product in the form of fragile damage or local failure of tanks. As practice shows, accidents RVS in most cases accompanied by a significant loss of products, poisoning the area and death. In extreme cases, according to statistics, the total material damage exceeds by 500 times or more the initial costs for the construction of reservoirs [17].

11.5. Statistics of accidents steel vertical tanks

On the basis of the conducted research, *Table 11.1* of steel vertical tanks accidents statistics was created. The data were retrieved using Internet sources, scientific publications and other media resources.

Table 11.1

Accidents of steel vertical tanks

N	Description of accident	City, country	Date
1	The collapse of the tank	Muravlenko, Russia	17.08.2018
2	Fire at the refinery	Ugra, Russia	30.10.2018
3	The collapse of the tank	India	28.08.2006
4	Destruction of the reservoir	Chesapeake, Virginia, USA	12.11.2008

N	Description of accident	City, country	Date
5	Six oil tanks descend from the rails	California, USA	09.05.2014
6	The tank has burned	Colorado, USA	17.04.2015
7	Fire in the reservoir for bursting of the well	Colorado, USA	08.05.2017
8	Explosion of the oil reservoir	North Colorado, USA	25.05.2017
9	The explosion resulting in damaged or destroyed up to six oil reservoirs	Colorado, USA	19.06.2018
10	Tank explosion	Delaware, USA	17.07.2001
11	Fire in an oil tank	Kansas, USA	26.06.2018
12	Tank burst	Mississippi, USA	31.10.2009
13	The explosion, which led to 36 oil reservoirs	New Mexico, USA	11.07.2016
14	Several oil reservoirs burned	South dakota	18.08.2018
15	Tank explosion	Texas, USA	17.10.2017
16	Fire in an oil reservoir in Cherokee	Texas, USA	27.12.2017
17	Fire in an oil tank in Madison County	Texas, USA	03.01.2018
18	Fire in a crude oil storage tank across the US pipeline east of Wichita Falls	Texas, USA	28.08.2018
19	Several oil storage tanks were involved in a fire in Campbell's northern constituency	Wyoming, USA	16.03.2018
20	Due to a malfunction of the reservoir at the winemaking plant, 30,000 liters of prosecco spilled	Veneto, Italy	28.09.2018
21	Overpressure of one of the reservoirs	Bila Tserkva, Ukraine	28.08.2017
22	Sulfuric acid spilled out of the reservoir	Oberhausen, Germany	16.02.2017
23	The tank was lit when cleaning	Samara, Russia	31.08.2018

N	Description of accident	City, country	Date
24	During the drain of fuel oil into an underground reservoir an explosion with destruction of the capacity occurred	Russia	26.12.2010
25	The "dead" residue of oil was burning	Russia	24.09.2010
26	A reservoir burned during welding	Tamur district, Dagestan, Russia	23.04.2010
27	Tank explosion	Russia	28.03.2010
28	In the whitewash shop for cellulose production, a dust-gas-air mixture in the hydrochloric acid reservoir exploded	Irkutsk region, Russia	03.09.2009

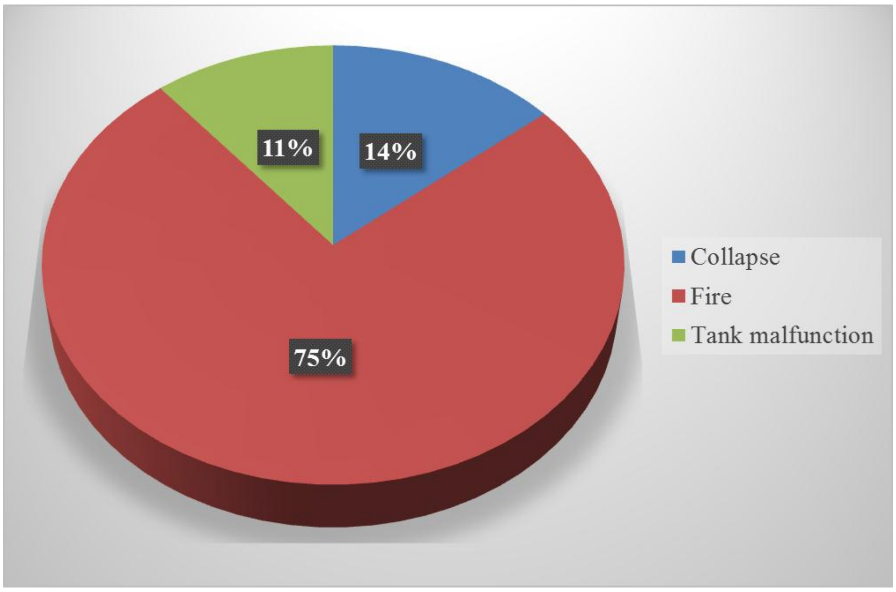

Diagram 2. Percentage ratio of the type of accident at the high security facility from 2009-2019.

According to the table, a percentage of the tanks destruction by accident type from 2009–2019 was created. The analysis showed that the highest failure percentage of the structure normal operation accounted for the fire occurrence or a sudden explosion (75%). Such accidents include the ignition of the reservoir internal vapors, the explosion, the reservoir ignition from neighboring objects enveloped in a fire, the ignition of the gas-air mixture during dry cleaning, etc.

The tanks collapse covers 14% of the total accidents number, indicating an improvement in the methods of this type construction. Only 11% accounted for by accidents caused by faulty tanks, their depressurization and subsequent inability to operate as intended.

Fig. 11.4. An accident at a winery in Italy,
(30 thousand liters of prosecco leaked onto asphalt, 2018)

Conclusions

The prerequisites for the implementation of the methodology and algorithm model for the probability of an accident at a high-risk construction site gave impetus to the development of such a direction in the scientific activity of the construction industry as predicting the progressive destruction of structures.

The need to create this algorithm is based on the commercial need for such calculations by the construction customer. The demand for this analysis of a construction object becomes the basis for introducing clear construction standards for a model failure of a potential accident, based on statistics from previous years, the dynamics of the occurrence of certain types of accidents.

Analysis of accidents and causes of accidents of steel vertical tanks showed that at present the most common reservoir destruction are fires, namely explosions, or the construction of fires from external objects covered by the flame.

It should be noted that the percentage of reservoir destruction from precipitation has significantly decreased, which indicates a decrease in the cases of detection of geometric shape defects and uneven precipitation.

It is also worth noting that the solution to the problem of improving the performance of tanks should be reduced to the implementation of constructive-technological, operational and organizational measures. Constructive-technological measures to improve the resource safe operation of tanks are performed at the stages of design, manufacture and installation. One of the most important conditions for ensuring high reliability and safety of tanks is the use of fine-grained steels with high resistance to brittle fracture in their manufacture.

Control questions

1. What is the Seveso disaster reason and effect?
2. Give explanation of the HAZOP method. What is its essence?
3. What are the main causes of steel vertical tanks accidents?
4. Give an example of a tank accidents. What is its cause?

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ANNEXES

Table D.1

Normal Distribution Density Table

$$\varphi(x)=\frac{1}{\sqrt{2\pi}}e^{-\frac{x^2}{2}}$$

x	0	1	2	3	4	5	6	7	8	9
0,00	3989	3989	3989	3988	3986	3984	3982	3980	3977	3973
0,1	3970	3965	3961	3956	3951	3945	3939	3932	3925	3918
0,2	3910	3902	3894	3885	3876	3867	3857	3847	3836	3825
0,3	3814	3802	3790	3778	3765	3752	3739	3726	3712	3697
0,4	3683	3668	3652	3637	3621	3605	3589	3572	3555	3538
0,5	3521	3503	3485	3467	3448	3429	3410	3391	3372	3352
0,6	3332	3312	3292	3271	3251	3230	3209	3187	3166	3144
0,7	3123	3101	3079	3056	3034	3011	2989	2966	2943	2920
0,8	2897	2874	2850	2827	2803	2780	2756	2732	2709	2685
0,9	2661	2637	2613	2589	2565	2541	2516	2492	2468	2444
1,0	2420	2396	2371	2347	2323	2299	2275	2251	2227	2203
1,1	2179	2155	2131	2107	2083	2059	2036	2012	1989	1965
1,2	1942	1919	1895	1872	1849	1826	1804	1781	1758	1736
1,3	1714	1691	1669	1647	1626	1604	1582	1561	1539	1518
1,4	1497	1476	1456	1435	1415	1394	1374	1354	1334	1315
1,5	1295	1273	1257	1238	1219	1200	1182	1163	1145	1127
1,6	1109	1092	1074	1057	1040	1023	1006	0989	0973	0957
1,7	0940	0925	0909	0890	0878	0863	0848	0833	0818	0804
1,8	0790	0775	0761	0748	0734	0721	0707	0694	0681	0669
1,9	0656	0644	0632	0620	0608	0596	0584	0573	0562	0551
2,0	0540	0529	0519	0508	0498	0488	0478	0468	0459	0449
2,1	0440	0431	0422	0413	0404	0396	0387	0379	0371	0363
2,2	0355	0347	0339	0332	0325	0317	0310	0303	0297	0290
2,3	0283	0277	0270	0264	0258	0252	0246	0241	0235	0229
2,4	0224	0219	0213	0208	0203	0198	0194	0189	0184	0180
2,5	0175	0171	0167	0163	0158	0154	0151	0147	0143	0139
2,6	0136	0132	0128	0126	0122	0119	0116	0113	0110	0107
2,7	0104	0101	0099	0096	0093	0091	0088	0086	0084	0081
2,8	0079	0077	0075	0073	0071	0069	0067	0065	0063	0061
2,9	0060	0058	0056	0055	0053	0051	0050	0048	0047	0046
3,0	0044	0043	0042	0040	0039	0038	0037	0036	0035	0034
3,1	0033	0032	0031	0030	0029	0028	0027	0026	0025	0025
3,2	0024	0023	0022	0022	0021	0020	0020	0019	0018	0018
3,3	0017	0017	0016	0016	0015	0015	0014	0014	0013	0013
3,4	0012	0012	0012	0011	0011	0010	0010	0010	0009	0009
3,5	0009	0008	0008	0008	0008	0007	0007	0007	0007	0006
3,6	0006	0006	0006	0005	0005	0005	0005	0005	0005	0004
3,7	0004	0004	0004	0004	0004	0004	0003	0003	0003	0003
3,8	0003	0003	0003	0003	0003	0002	0002	0002	0002	0002
3,9	0002	0002	0002	0002	0002	0002	0002	0002	0001	0001

Table D.2

Laplace function value table $$\Phi\left(x\right)=\frac{1}{\sqrt{2\pi}}\int_0^x e^{\frac{-z^2}{2}}dz$$

x	*Φ(x)*	*x*	*Φ(x)*	*x*	*Φ(x)*	*x*	*Φ(x)*
0.00	*0.0000*	*0.32*	*0.1255*	*0.64*	*0.2389*	*0.96*	*0.3315*
0.01	*0.0040*	*0.33*	*0.1293*	*0.65*	*0.2422*	*0.97*	*0.3340*
0.02	*0.0080*	*0.34*	*0.1331*	*0.66*	*0.2454*	*0.98*	*0.3365*
0.03	*0.0120*	*0.25*	*0.1368*	*0.667*	*0.2486*	*0.99*	*0.3389*
0.04	*0.0160*	*0.36*	*0.1406*	*0.68*	*0.2517*	*1.00*	*0.3413*
0.05	*0.0199*	*0.37*	*0.1443*	*0.69*	*0.2549*	*1.01*	*0.3438*
0.06	*0.0239*	*0.38*	*0.1480*	*0.70*	*0.2580*	*1.02*	*0.3461*
0.07	*0.0279*	*0.39*	*0.1517*	*0.71*	*0.2611*	*1.03*	*0.3485*
0.08	*0.0319*	*0.40*	*0.1554*	*0.72*	*0.2642*	*1.04*	*0.3508*
0.09	*0.0359*	*0.41*	*0.191*	*0.73*	*0.2673*	*1.05*	*0.3531*
0.10	*0.0398*	*0.42*	*0.1628*	*0.74*	*0.2703*	*1.06*	*0.3554*
0.11	*0.0438*	*0.43*	*0.1664*	*0.75*	*0.2734*	*1.07*	*0.3577*
0.12	*0.0478*	*0.44*	*0.1700*	*076*	*0.2764*	*1.08*	*0.3599*
0.13	*0.0517*	*0.45*	*0.1736*	*0.77*	*0.2794*	*1.09*	*0.3621*
0.14	*0.0557*	*0.46*	*0.1772*	*0.78*	*0.2823*	*1.10*	*0.3643*
0.15	*0.0596*	*0.47*	*0.1808*	*0.79*	*0.2852*	*1.11*	*0.3665*
0.16	*0.0636*	*0.48*	*0.1844*	*0.80*	*0.2881*	*1.12*	*0.3686*
0.17	*0.0685*	*0.49*	*0.1879*	*0.81*	*0.2910*	*1.13*	*0.3708*
0.18	*0.0714*	*0.50*	*0.1915*	*0.82*	*0.2939*	*1.14*	*0.3729*
0.19	*0.0753*	*0.51*	*0.1950*	*0.83*	*0.2967*	*1.15*	*0.3849*
0.20	*0.0793*	*0.52*	*0.1985*	*0.84*	*0.2995*	*1.16*	*0.3770*
0.21	*0.0832*	*0.53*	*0.2019*	*0.85*	*0.3023*	*1.17*	*0.3790*
0.22	*0.0871*	*0.54*	*0.2054*	*0.86*	*0.3051*	*1.18*	*0.3810*
0.23	*0.0910*	*0.55*	*0.2088*	*0.87*	*0.3087*	*1.19*	*0.3830*
0.24	*0.0948*	*0.56*	*0.2123*	*0.88*	*0.3106*	*1.20*	*0.3849*
0.25	*0.0987*	*0.57*	*0.2157*	*0.89*	*0.3133*	*1.21*	*0.3869*
0.26	*0.1026*	*0.58*	*0.2190*	*0.90*	*0.3159*	*1.22*	*0.3883*
0.27	*0.1064*	*0.59*	*0.2224*	*0.91*	*0.3186*	*1.23*	*0.3907*
0.28	*0.1103*	*0.60*	*0.2257*	*0.92*	*0.3212*	*1.24*	*0.3925*
0.29	*0.1141*	*0.61*	*0.2291*	*0.93*	*0.3238*	*1.25*	*0.3944*
0.30	*0.1179*	*0.62*	*0.2324*	*0.94*	*0.3264*		
0.31	*0.1217*	*0.63*	*0.2357*	*0.95*	*0.3289*		

Continuation of table D.2

x	*Φ(x)*	*x*	*Φ(x)*	*x*	*Φ(x)*	*x*	*Φ(x)*
1.26	0.3962	1.59	0.4441	1.92	0.4726	2.50	0.4928
1.27	0.3980	1.60	0.4452	1.93	0.4732	2.52	0.4941
1.28	0.3997	1.61	0.4463	1.94	0.4738	2.54	0.4945
1.29	0.4015	1.62	0.4474	1.95	0.4744	2.26	0.4948
1.30	0.4032	1.63	0.4484	1.96	0.4750	2.58	0.4951
1.31	0.4049	1.64	0.4495	1.97	0.4756	2.60	0.4953
1.32	0.4066	1.65	0.4505	1.98	0.4761	2.62	0.4956
1.33	0.4082	1.66	0.4515	1.99	0.4767	1.64	0.4959
1.34	0.4099	1.67	0.4525	2.00	0.4772	2.66	0.4961
1.35	0.4115	1.68	0.4535	2.02	0.4783	2.68	0.4963
1.36	0.4131	1.69	0.4545	2.04	0.4793	2.70	0.4965
1.37	0.4147	1.70	0.4554	2.06	0.4803	2.72	0.4967
1.38	0.4162	1.71	0.4564	2.08	0.4812	2.74	0.4969
1.39	0.4177	1.72	0.4573	2.10	0.4821	2.76	0.4971
1.40	04192	1.73	0.4582	2.12	0.4830	2.78	0.4973
1.41	0.4207	1.74	0.4591	2.14	0.4838	2.80	0.4974
1.42	0.4222	1.75	0.4599	2.16	0.4846	2.82	0.4976
1.43	0.4236	1.76	0.4608	2.18	0.4854	2.84	0.4977
1.44	0.4251	1.77	0.4616	2.20	0.4861	2.86	0.4979
1.45	0.4265	1.78	0.4625	2.22	0.4868	2.88	0.4980
1.46	0.4279	1.79	0.4633	2.24	0.4875	2.90	0.4981
1.47	0.4292	1.80	0.4641	2.26	0.4881	2.92	0.4982
1.48	0.4306	1.81	0..4649	2.28	0.4887	2.94	0.4984
1.49	0.4319	1.82	0.4656	2.30	0.4893	2.96	0.4985
1.50	0.4332	1.83	0.4664	2.32	0.4898	2.98	0.4986
1.51	0.4345	1.84	0.4671	2.34	0.4904	3.00	0.49865
1.52	0.4357	1.85	0.4678	2.36	0.4909	3.20	0.49931
1.53	0.4370	1.86	0.4686	2.38	0.4913	3.40	0.49966
1.54	0.4382	1.87	0.4693	2.40	0.4918	3.60	0.499841
1.55	0.4394	1.88	0.4699	2.42	0.4922	3.80	0.499928
1.56	0.4406	1.89	0.4706	2.44	0.4927	4.00	0.499968
1.57	0.4418	1.90	0.4713	2.46	0.4931	4.50	0.499997
1.58	0.4429	1.91	0.4719	2.48	0.4934	5.00	0.499997

Table D.3

The probability of large deviations *Q(β)* of the normal distribution

β	$Q(\beta)$	β	$Q(\beta)$
3.0	1.35*10–3	5.5	1.90*10–8
3.1	9.68*10–4	5.6	1.07*10–8
3.2	6.87*10–4	5.7	5.99*10–9
3.3	4.83*10–4	5.8	3.32*10–9
3.4	3.37*10–4	5.9	1.82*10–9
3.5	2.33*10–4	6.0	9.87*10–10
3.6	1.59*10–4	6.2	2.82*10–10
3.7	1.08*10–4	6.4	7.77*10–11
3.8	7.23*10–5	6.6	2.06*10–11
3.9	4.81*10–5	6.8	5.23*10–12
4.0	3.17*10–5	7.0	1.28*10–12
4.1	2.07*10–5	7.2	3.01*10–13
4.2	1.33*10–5	7.4	6.81*10–14
4.3	8.54*10–6	7.8	1.48*10–14
4.4	5.41*10–6	8.0	3.10*10–16
4.5	3.40*10–6	8.2	6.22*10–16
4.6	2.11*10–6	8.4	2.23*10–17
4.7	1.30*10–6	8.8	3.99*10–19
4.8	7.93*10–7	9.0	1.13*10–19
4.9	4.79*10–7	5.0	2.87*10–7
5.1	1.70*10–7	5.2	9.96*10–8
5.3	5.79*10–8	5.4	3.33*10–8

Table D.4

Critical distribution points χ^2

Number of freedom degrees	*Significance level α*					
k	*0.01*	*0.025*	*0.05*	*0.95*	*0.975*	*0.89*
1	*6.6*	*5.0*	*3.8*	*0.0039*	*0.00098*	*0.00016*
2	*9.2*	*7.4*	*6.0*	*0.103*	*0.051*	*0.020*
3	*11.3*	*9.4*	*7.8*	*0.352*	*0.216*	*0.115*
4	*13.3*	*11.1*	*9.5*	*0.711*	*0.484*	*0.297*
5	*15.1*	*12.8*	*11.1*	*1.15*	*0.831*	*0.554*
6	*16.8*	*14.4*	*12.6*	*1.64*	*1.24*	*0.872*
7	*18.5*	*16.0*	*14.1*	*2.17*	*1.69*	*1.24*
8	*20.1*	*17.5*	*15.5*	*2.73*	*2.18*	*1.65*
9	*21.7*	*19.0*	*16.9*	*3.33*	*2.70*	*2.09*
10	*23.2*	*20.5*	*18.3*	*3.94*	*3.25*	*2.56*
11	*24.7*	*21.9*	*19.7*	*4.57*	*3.82*	*3.05*
12	*26.2*	*23.2*	*21.0*	*5.23*	*4.40*	*3.57*
13	*27.7*	*24.7*	*22.4*	*5.89*	*5.01*	*4.11*
14	*29.1*	*26.1*	*23.7*	*6.57*	*5.63*	*4.66*
15	*30.6*	*27.5*	*25.0*	*7.26*	*6.26*	*5.23*
16	*32.0*	*28.8*	*26.3*	*7.96*	*6.91*	*5.81*
17	*33.4*	*30.2*	*27.6*	*8.67*	*7.56*	*6.41*
18	*34.8*	*31.5*	*28.9*	*9.39*	*8.23*	*7.01*
19	*36.2*	*32.9*	*30.1*	*10.1*	*8.91*	*7.63*
20	*37.6*	*34.2*	*31.4*	*10.9*	*9.59*	*8.26*
21	*38.9*	*35.5*	*32.7*	*11.6*	*10.3*	*8.90*
22	*40.3*	*36.8*	*33.9*	*12.3*	*11.0*	*9.54*
23	*41.6*	*38.1*	*35.2*	*13.1*	*11.7*	*10.2*
24	*43.0*	*39.4*	*36.4*	*13.8*	*12.4*	*10.9*
25	*44.3*	*40.6*	*37.7*	*14.6*	*13.1*	*11.5*
26	*45.6*	*41.9*	*38.9*	*15.4*	*13.8*	*12.2*
27	*47.0*	*43.2*	*40.1*	*16.2*	*14.6*	*12.9*
28	*48.3*	*44.5*	*41.3*	*16.9*	*15.3*	*13.6*
29	*49.6*	*45.7*	*42.6*	*17.7*	*16.0*	*14.3*
30	*50.9*	*47.0*	*43.8*	*18.5*	*16.8*	*15.0*

ПІЧУГІН СЕРГІЙ ФЕДОРОВИЧ

КЛОЧКО ЛІНА АНДРІЇВНА

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