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The allowable stress method is the basis of the modern method of calculating building structures according to limit states

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The article consistently examines the evolution of building construction calculation methods and their reflection in regulatory documents for the period from the 17th-18th centuries to the middle of the 20th century. Attention is drawn to the continuity of the method of limit states and the method of allowable stresses, which dominated the calculations of building structures until the 1950s. It is noted that foreign experience was taken into account when developing the first domestic design codes. A comparative analysis of the codes for calculating building structures based on allowable stresses with modern design codes is carried out. The general conclusion is substantiated that the method of allowable stresses, despite all its shortcomings, over the 200-year period of application still ensured the necessary reliability and safety of construction objects around the world. In the background of the method, a large and valuable baggage of scientific results was acquired, which were later laid as the basis of a new method of limit states

Keywords: design codes, reserve factor, allowable stresses, limit states, material strength, steel grade

Метод допустимих напружень – базис сучасного методу розрахунку будівельних конструкцій за граничними станами

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Незважаючи на тисячолітній досвід будівництва, проблема міцності споруди існувала завжди, актуальна вона і зараз. Починаючи з XVII століття, працями основоположників будівельної механіки розпочався розвиток методів розрахунку будівельних конструкцій, які з середини XIX століття почали оформлюватися у норми проєктування, обов'язкові для будівельників. Актуальність вивчення розвитку вітчизняних і зарубіжних норм проєктування пов'язана не тільки з тим, що історія дає фактичні знання про минулий досвід будівництва, але у певній мірі дозволяє прогнозувати тенденції розвитку будівельного нормування. У статті проведений послідовний огляд методів розрахунку будівельних конструкцій, починаючи з класичних досліджень XVII-XVIII століть до середини XX століття, коли домінував метод допустимих напружень. Детально розглянуто Урочне положення, яке регламентувало будівельну діяльність з середини XIX століття до початку XX століття. Оглянуті наукові дослідження, пов'язані з розвитком методики розрахунку за допустимими напруженнями, націлені на виявлення природи коефіцієнту запасу, врахування пластичної роботи матеріалу, дійсної роботи конструкцій і з'єднань. статистичної природи коефіцієнта запасу міцності конструкцій. Описана еволюція методів розрахунку будівельних конструкцій та їхнє відображення у нормативних документах. Відмічається врахування закордонного досвіду при розробці перших вітчизняних норм проєктування. Проводиться порівняльний аналіз норм розрахунку будівельних конструкцій за допустимими напруженнями із сучасними нормами проєктування. Обґрунтовано загальний висновок, що метод допустимих напружень, при всіх його недоліках, за 200-річний термін застосування все ж забезпечував необхідну надійність і безаварійність будівельних об'єктів по всьому світу. У підґрунті методу був набутий великий і цінний багаж наукових результатів, які згодом були покладені в основу нового методу граничних станів

Ключові слова: норми проєктування, коефіцієнт запасу, допустимі напруження, граничні стани, міцність матеріалу, марка сталі

Introduction

Despite thousands of years of construction experience, the problem of building strength has always existed, and it is still relevant today. For a long time, building mechanics and design codes did not exist, therefore, even in the most perfect ancient buildings, you can find gross errors, which testify to their ignorance of the basics of the strength of materials and the theory of buildings. The builders of the past were mainly guided by traditions and recipes acquired over the centuries. Since ancient times, the construction profession was considered very responsible, and possible construction errors had very serious consequences for those who made them. Let us recall here the Laws of Hammurabi, compiled back in 1750 BC. e., the text of which was found on a stone ceiling at the beginning of the 20th century on the territory of Persia [1]. These Laws quite strictly regulated the responsibility of the builders of ancient Mesopotamia. Starting from the 17th century, with the works of the classics of construction mechanics, the development of methods for calculating building structures began, which from the middle of the 19th century were formalized into design codes, which are mandatory for builders. The relevance of studying the development of domestic and foreign design codes is not only related to the fact that history provides factual knowledge about the past experience of construction, but it allows predicting trends in the development of construction codes.

Review of research sources of and publications

Individual stages of the development of building mechanics and building structures are described in one of the first works on this topic [2], quite vividly supplemented in a brochure [3], illuminated in a domestic monograph [4] and in foreign publications [5, 6]. Construction activity in the Russian Empire from the middle of the 19th century to the beginning of the 20th century was regulated by the capital Urgent position, published in several editions [7]. Foreign building codes, in particular, German, were illustrated by the multi-volume HŰTTE handbook, which was quite popular until the 1930s [8]. During this period, scientific research related to the development of the methodology for calculating allowable stresses, aimed at identifying the nature of the reserve factor [9-11], taking into account the plastic work of the material [12, 15], the actual work of structures and connections [13, 14, 16]. The end of the 1930s was marked by an active scientific attack by domestic and foreign scientists to identify the statistical nature of the safety factor of structures [17-19]. A detailed analysis of the design codes of 1930-40 was carried out in the capital course of metal structures [20]. In recent years, there has been renewed interest in the history of domestic design standards, which has been reflected in a series of publications [21-24].

Definition of insolved aspects of the problem

To date, there are no scientific publications in which the chronological development of the method of calculating the allowable stresses, which was the basis of the design of building structures for more than 200 years, until the middle of the 20th century, is analyzed in detail. Therefore, the questions of how the builders of the past ensured the reliability of buildings and structures that have been safely preserved to our time are not clear. Issues of substantiation of some calculated parameters and coefficients of the allowable stress method, which later became part of the codes of the limit state method, remained unclear. It can be considered that the long-term positive potential of the allowable stress method, on the basis of which the transition to the modern calculation of building structures according to limit states was carried out in the 1950s, was generally overlooked.

Problem statement

The article consistently examines the evolution of building construction calculation methods and their reflection in regulatory documents for the period from the 17th-18th centuries to the middle of the 20th century. Attention is drawn to the continuity of the method of limit states and the method of allowable stresses, which dominated the calculations of building structures until the 1950s. It is noted that foreign experience was taken into account when developing the first domestic design codes in the 1930s. A comparative analysis of the codes for calculating building structures based on allowable stresses with modern design codes is carried out.

Basic material and results

The history of the development of construction science preserves many famous names of scientists who, to one degree or another, developed the issue of the strength of structures. Let's name here only some of them, which created the basis of the future standards for the design of building structures. It is generally known that the science of strength was initiated by Galileo Galilei (1564-1642) in the first half of the 17th century. For the first time, he considered two types of deformation of the rod: tension and bending, and in both cases he looked for the value of the destructive load. The next step was the establishment 40 years later by Robert Hooke (1635-1703) of the law of proportionality between load and deformation and the properties of elasticity of bodies. An important stage in the development of construction mechanics was the work of Leonard Euler (1707-1783), dedicated to solving the problem of stability of compressed elements.

The famous French engineer and scientist Louis Navier (1785-1836) first introduced the reserve factor into construction science. Already an academician, Navier in 1826 published a course of lectures in which he laid the foundation of the theory of elasticity and introduced the concept of stresses. In contrast to Galileo and his followers, who focused on the destructive (limit) state of the structure, Navier proposed to establish working stresses at which the structure can work reliably, and to calculate these stresses. It is obvious that these stresses should be much less destructive. "Resistance to destruction", Navier wrote, "is not enough for design, because one needs to know not the destructive force, but the one with which the element can be loaded without the changes occurring in it increasing over time". This approach became the basis of the reform of construction mechanics based on the calculation by working state, later called the calculation by allowable stresses. It is interesting to remember that it was Navier who introduced the designation of allowable stress with the letter *R*, which is still used today. For iron with an ultimate strength of 4000 kgf/cm^2, Navier recommendded an allowable bending stress of 1300 kgf/cm^2 (reserve factor 3.08), which is quite close to the codes of the middle of the 20th century for the same steel. For tension, he reduced this value to 1000 kgf/cm^2 (reserve factor 4) and even 600 kgf/cm^2 (reserve factor 6.67).

Subsequently, the design calculation was developed by the outstanding research engineer D. Zhuravsky (1821-1891), who solved the problem of tangential stresses during bending, his famous formula was published in 1855. At the end of the 19th century, scientists from various countries conducted research on the stability of structures, among which the work of F. Yasynskyi (1856-1899), who solved the problem of stability of elements in the inelastic region and proposed the coefficient φ of reduction of allowable stress during longitudinal bending (1894). Thus, the problem of stability was reduced to an equivalent problem of strength by establishing variable allowable stresses that depend on flexibility. It can be assumed that with this Yasynskyi finally completed the transition of material strength to the principle of calculation based on allowable stresses, which Navier had begun. Thus, this transition took 70 years [2,3].

The fundamental question remained unclear: what exactly should be the reserve factor and the value of permissible stress? It depended on whether the building would withstand the loads applied to it. Accidents and destruction occurred not only in the ancient and middle ages, they continued later and occur even in the present time. Each accident added new knowledge to the builders, set new tasks. When there was a lack of knowledge, a reserve factor was introduced into engineering calculations. For example, the load that the element can withstand during operation was determined, and its dimensions were selected that allowed it to withstand loads greater than the operating load, say, 100 times. This meant that the created element had a reserve factor of 100. Since no one knew what unpredictable, unknowable phenomena this factor accounted for and whether it should be exactly that, and not 10 times less, for example, it was called the factor of ignorance. The famous Scottish mechanic V. Rankin (1820-1872) proposed the reserve factor equal to 4.0. This coefficient was taken into account at the end of the 19th - at the beginning of the 20th century in building codes for various materials and building structures. At the same time, the allowable stress of different countries codes, in particular, for iron and steel, had significant differences: in England they were equal to 1080-1240 kgf/cm^2, in Germany 1150 kgf/cm^2 for tension and 950 kgf/cm^2 for compression, in Russia, respectively, 800 and 650 kgf/cm^2 [4-6].

As construction science developed, the reserve factor, essentially the factor of ignorance, was changed, so in fact the entire history of strength science was a history of the struggle to reduce this factor of ignorance. Now this factor has become relatively small (we will talk about it later), but it took centuries. So, starting from the 19th century, the reserve factor was established on the basis of engineering intuition, experience in the design and operation of structures and reigned undividedly in construction mechanics until the 50s of the 20th century.

The first normative document of pre-revolutionary Russia, which included some provisions of the method of calculation of structures, was "Urgent position: a guide for drawing up and checking estimates, designing and performing works" [7]. It is interesting to note that its author was the Russian Count N. de Rochefort (1846-1903), civil engineer and architect, builder of railways and highways, author of the palaces of St. Petersburg. Urgent position is a unique manual that served as a reference for builders and architects, a textbook for teachers, and a guide for construction contractors. It first explained the building codes and regulations and contained the necessary reference material on construction. Urgent position first came into force in 1869, were reprinted with changes 13 times, the last edition was printed in 1930. The edition to this day is a universal table book on construction, it is the only edition published in Tsarist Russia that can still serve many civil engineers. The Urgent position had an important state status, it was mandatory for use throughout the state, the permission for the first edition was signed by Alexander II, the permission for the sixth edition, which is cited further in the text, was signed by Nicholas II.

For the main structural materials, mechanical characteristics that could be used in the calculations of structures based on allowable stresses were given in the Urgent position. For comparison, the text also contained similar information from German codes. The Table 1 contains systematized data on the strength of construction steels, which were then called "welded iron" (intended for further hot processing – strip, bars, round profile) and "cast iron" (not subject to hot processing – corners, bars, rails, rolled beams). The Table 1 shows that the strength of pre-revolutionary cast iron was similar to the strength of modern steel grade St3, and the higher-quality metal of rails, springs, and springs is associated with the strength of modern low-alloy steels. The level of allowable steel tensile stresses for truss structures and bridges (average values of 860 and 820 kgf/cm^2) corresponds to the reserve factors of 4.56 and 4.81, respectively, in relation to the average destructive stresses of cast iron. The specifics of the work of compressed elements of rafter trusses were taken into account by reducing the allowable stress to an average of 660 kgf/cm^2 for short elements and to 175 kgf/cm^2 for long elements (a reserve factor of 22.5), which generally took into account the influence of flexibility on the stability of compressed elements of rafter trusses. As can be seen from the Table 1, the allowable stresses of Germany for structures of truss and bridges were bolder, as they prevailed in Russia by about 1.5 times and approached the domestic level of allowable stresses of the 50s of the 20th century, $[\sigma]$ = 1600 kgf/cm^2. The shear strength of rivets was assumed to be equal to 4/5 of the tensile strength of structural elements.

The Urgent positionalso contained some instructions regarding loads on structures, in particular, on rafter trusses (Table 2). It can be assumed that the regulated variable load on the roofs of 160 kgf/m^2 with a margin took into account the snow load in the main territory of pre-revolutionary Russia, and the total calculated load of 180-230 kgf/m^2, together with a strength margin, ensured a certain level of safety of structures made according to competent calculations. At the same time, we note that the Russian recommendations regarding the total loads on the roof correspond to the German standards for flat roofs (Table 2), which additionally contain increased load values for steep roofs common in Germany.

The Urgent position [7] also contains strength data (temporary strength) of wood - a common building material then and now (Table 3). For comparison, the average values of the strength limit of modern wood (small clean samples) taken from a review [21] are given in the same table.

Table 1 – Mechanical characteristics of metals, kgf/cm^2 (Urgent position)

<table>
<tr><td colspan="2">Destructive stresses</td><td colspan="6">Allowable stresses</td></tr>
<tr><td colspan="2" rowspan="2">Russia</td><td colspan="3">Russia</td><td colspan="3">Germany</td></tr>
<tr><td colspan="6">Rafter structures</td></tr>
<tr><td>Welded iron</td><td>3000-3910</td><td>Tension</td><td colspan="2">810-915</td><td rowspan="3">Tension Comp-res-sion</td><td>Welded iron</td><td>до 1440</td></tr>
<tr><td>Cast iron</td><td>3035-4850</td><td rowspan="2">Compres-sion short elts long elts</td><td colspan="2" rowspan="2">610-710
150-200</td><td rowspan="2">Cast iron</td><td rowspan="2">до 1600</td></tr>
<tr><td rowspan="2">Beams, rails</td><td rowspan="2">3730-5300</td></tr>
<tr><td colspan="6">Bridges</td></tr>
<tr><td>Springs</td><td>5300-8000</td><td rowspan="2">Tension Compres-sion</td><td>Welded iron</td><td>600-725</td><td rowspan="2">Tension Compres-sion</td><td>Welded iron</td><td>750-1000</td></tr>
<tr><td>Winders</td><td>8000-9015</td><td>Cast iron</td><td>690-950</td><td>Cast iron</td><td>1000-1200</td></tr>
</table>

Table 2 – Loads on rafter trusses (Urgent position)

<table>
<tr><td>Country</td><td colspan="4">Loads, kgf/m^2</td></tr>
<tr><td rowspan="3">Russia</td><td>Own weight</td><td colspan="3">20 – 70</td></tr>
<tr><td>Variables</td><td colspan="3">160</td></tr>
<tr><td>Total</td><td colspan="3">180 – 230</td></tr>
<tr><td rowspan="4">Germany</td><td rowspan="4">Total loads</td><td rowspan="2">Steep roofs (1:3)</td><td>Heavy (shingles)</td><td>300</td></tr>
<tr><td>Light (iron, zinc)</td><td>250</td></tr>
<tr><td rowspan="2">Sloping roofs (1:4)</td><td>Heavy (shingles)</td><td>225</td></tr>
<tr><td>Light (iron, zinc)</td><td>185</td></tr>
</table>

Table 3 – Comparative data on the strength of wood, kgf/cm^2

<table>
<tr><td rowspan="2">Wood Breeds</td><td colspan="2">Tension</td><td colspan="2">Compression</td><td colspan="2">Chipping</td></tr>
<tr><td>[7]</td><td>[21]</td><td>[7]</td><td>[21]</td><td>[7]</td><td>[21]</td></tr>
<tr><td>Larch</td><td>1120</td><td rowspan="3">940</td><td>560</td><td rowspan="3">470</td><td>–</td><td rowspan="3">74</td></tr>
<tr><td>Pine</td><td>1020</td><td>510</td><td>100</td></tr>
<tr><td>Spruce</td><td>960</td><td>480</td><td>–</td></tr>
<tr><td>Oak</td><td>810</td><td>1130</td><td>530</td><td>526</td><td>150</td><td>103</td></tr>
</table>

It can be noted that the tensile, compressive and chipping strength of modern coniferous wood is of the same order and even somewhat less (by 10-20%) compared to wood used in construction more than 100 years ago. The Urgent position regulated the reserve factor for wood equal to 10, i.e. the basic allowable tensile stress of wood was equal to an average of 100 kgf/cm^2. In modern codes for the design of wooden structures, the design strength of wood is also significantly less than the initial strength of small clean samples due to the consideration of decreasing reliability coefficients, long-term strength of wood and working conditions. Let's illustrate the transition to the design strength using the example of the tensile strength of coniferous wood along the fibers: pure wood - average value R_w^u = 100 MPa, normative value R_w^n = 60 MPa; lumber of the 1st grade - average value R^u = 34 MPa, normative R^n = 20 MPa; stretched elements of the 1st grade – design strength R_p = 10 MPa = 100 kgf/cm^2 [20].

Somewhat unexpectedly, it turned out that the design strength of wood at the beginning of the 21st century coincides with the allowable stress of pre-revolutionary wood, which can be considered an example of the continuity of current design codes.

For bricks, the Urgent position provide several values of compressive strength, called "temporary strength to fragmentation", which was equal to 70 kgf/cm^2 for masonry of monumental and tall buildings and 56 kgf/cm^2 on average for masonry. Note that the first of these values is close to the average strength $\overline{R}$ = 75 kgf/cm^2 of the M75 brand of modern expanded bricks. The allowable stress of the brick, taking into account the reserve factor equal to 10, was assumed to be equal to 5.6 kgf/cm^2, and the builders of that time were probably guided by this rather low value when calculating the strength of the brickwork. At the same time, in most cases, brick walls were laid without calculation, based on the recommendation of the Urgent position, that in Russian climatic conditions the walls have a thickness of at least 2.5 bricks. As the number of floors of buildings increased, the thickness of the walls increased, in particular, to 3.5 bricks on the lower floors of five-story buildings. The strength ("temporary strength to fragmentation") of brickwork at that time was quite high: "good" masonry - 60 kgf/cm^2, "weak" - 38 kgf/cm^2. To calculate the strength of load-bearing walls, significantly lower allowable stresses with significant safety factors were used: loaded walls – 2.0-5.0 kgf/cm^2 (reserve factors 12-30); loaded pillars and columns - 1.2-1.5 kgf/cm^2 (reserve coefficients 40-50). It can be assumed that this is why the brick walls of pre-revolutionary houses had a large thickness. Meanwhile, modern tests and calculations show significantly greater strength of brickwork. For example, masonry made of M75 bricks on M75 solution has an average strength of 28 kgf/cm^2, which, with a homogeneity factor of k = 0.5, gives a value of the design strength of the masonry of 14 kgf/cm^2 [21].

The Urgent position also gives the value of the limit of strength ("strength to fragmentation") for natural stones, in particular, granite, basalt, limestone, etc., together with a general reserve factor of 10.

In the second half of the 19th century and the beginning of the 20th century, there was practically no Russian technical literature. Considering the fact that the industrial development of tsarist Russia in the specified period was largely provided by foreign companies, translated technical publications, mainly German, were popular. Such a publication was the Hütte handbook - a multi-volume "Handbook for engineers, technicians and students". The first German edition of the Handbook was published in Germany in 1857, it included sections: mathematics and mechanics, mechanical engineering and construction. Soon, in 1863, the first Russian translation of the Handbook was published. Before the Second World War, Hütte was one of the most common technical reference books in the USSR. The Handbook continued to be published in the post-war years, the last 34th edition was printed in 2012. During its more than 150-year history, the format of the Handbook was constantly changing: from 1890 it was published in two volumes, from 1908 - in three volumes, and finally from 1922 - at four. Considering the topic of the article, the 26th German edition of the Handbook and its 15th Russian translation, which was published in the early 1930s and was supplemented with information about Soviet standards and materials of that time, are of interest [8].

The Hütte handbook [8] contains data on the allowable stresses of metals that were officially allowed in Germany for use in building structures in 1925 (Tables 4 and 5). German codes used the modern term "steel" for metals in building structures, leaving the outdated and imprecise name "iron" mainly for existing structures. Two grades of steel were recommended for use: St 37 and high-grade St 48 with basic allowable stresses of 1200 and 1540 kgf/cm^2, respectively. The table of allowable stresses for civil structures, regulated in Prussia (region of Germany) (Table 4) had an expanded form, contained stress values for steel structures for tension, bending and shear, as well as data for rivets, clean and black bolts, anchor bolts. It is interesting to note that the Prussian codes of 1925 used designations for allowable stresses, which later became in the method of limit states (Table 4). As it was already indicated above, the allowable stresses of the German codes prevailed over the Russian codes of the Urgent position (Table 1).

In the German codes for railway bridges of 1925 (Table 5), allowable stresses under the action of "main forces" and somewhat larger values were already distinguished when taking into account wind pressure and additional loads.

Hütte [8] also contains strength data (temporary strength) of wood, a mass construction material of its time (1925) (Table 6). For comparison, the average values of the limit strength of modern wood (small clean samples) taken from a review [21] are given in the same Table 6.

As can be seen from the Table 6, only the tensile strength along the fibers is close to the German wood of the beginning of the 20th and Soviet wood of the beginning of the 21st centuries, other German indicators - compression, bending and chipping - are much smaller than modern domestic ones. It can be assumed that this is a consequence of different methods of wood samples testing.

The strength characteristics (temporary strength σ_B) of bricks, mortar and brickwork, given in the Hütte handbook [8], can be found in the Table 7. For comparison, the average values of the strength limits of modern mortar, brick, and brickwork, taken from a review [21], are given in the same table.

The comparison shows that the German brick had a strength similar to the modern one: for the 2nd grade $\sigma_B \geq$ 100 kgf/cm^2, which corresponds to the average strength of the M100 brick [21]. The average strength of modern cement mortars of the M50 - M200 brands is 50-200 kgf/cm^2, slightly less than the strength of German cement mortars, but much higher than the strength of lime mortars. At the same time, the strength of modern brickwork turned out to be much lower than that of the German at the beginning of the last century: our masonry made of M100 brick on M150 mortar has an average strength of 38 kgf/cm^2 [21], which is significantly less than the German indicators for masonry on lime mortar (54 kgf /cm^2) and especially on cement mortar (128 kgf/cm^2) (Table 7). Perhaps such great strength of German masonry is connected with the high quality of bricks and work, as well as with the use of high-strength mortars.

Table 4 – Allowable metal stresses for civil structures (Codes of the Prussian Ministry of 1925) [8]

Form of products	Designation	Allowable stresses, kgf/cm^2		Welded iron
		Cast steel brand St37	High-grade steel brand St48	
Rolled shaped beams, articulated parts of structures, support racks, etc	R_b, R_z	1200	1560	Less by 10%
	R_s	1000	1300	
Rivets and driven bolts with screw cutting	R_s	1000	1300	
	R'_d	2000	2600	
Ordinary bolts with a hread (raw, black bolts)	R_s	800	1040	
	R'_d	1600	2080	
Anchor bolts	R_z	800	1040	
Designation: Allowable stresses: R_b – bending; R_z – tension; R_s – shear; R'_d – crumpling of the inner surface of the hole				

Table 5 – Allowable metal stresses for railway bridges, kgf/cm^2 (Instructions of the German Railway Department, 1925) [8]

Sort steel	Medium yield strength kgf/cm^2	Allowable tensile and bending stresses of the main beams and beams of the carriageway under the action	
		ain forces	main forces, wind pressure and additional loads
Bridges under construction			
Cast steel St 37	2400	1400	1600
High-grade steel St 48	3120	1820	2080
Existing bridges			
Welded and cast iron, construction time before 1895	2200	1400	1600
Cast iron, construction time after 1895	2400	1600	1700

Table 6 – Mechanical characteristics of wood, kgf/cm^2 (Germany, 1925)

Wood breeds	Tension		Compression		Bending		Chipping	
	Hütte	[21]	Hütte	[21]	Hütte	[21]	Hütte	[21]
Fir	800	940	280	470	470	806	45	74
Pine	750		250		420		40	
Spruce	750		250		400		40	
Oak	1000	1130	400	526	600	970	75	103
Beech	1340		350		670		85	

Table 7 – Mechanical characteristics of brickwork, kgf/cm^2 (Germany, 1925)

Brick		Brickwork, σ_B, kgf/cm^2			Mortar, σ_B, kgf/cm^2		
Germany		Germany		[21]	Germany		[21]
Сорт	σ_B, kgf/cm^2	Lime mortar	Cement mortar		Lime	Cement	
First	≥ 150	54	128	38	15	200 – 350	50 – 200
Second	≥ 100	32	–				

Much attention was paid in the German Rules of 1925 and, accordingly, in the Hütte handbook [8] to the calculation of compressed elements for longitudinal bending "according to the method ω". Tetmayer's studies were taken as a basis, according to which for St37 steel in the plastic region for elements with flexibility $\lambda < 60$, the critical stresses were assumed to be constant, equal to σ_k = 2400 kgf/cm^2; the elastic-plastic area at $60 < \lambda < 100$ was described by the linear dependence of $\sigma_k = (2890.5 - 8{,}175\lambda)$ kgf/cm^2 and the elastic area at $\lambda > 100$ - by the Euler hyperbola $(20726000/\lambda^2)$ kgf/cm^2. The critical stress and allowable compressive stress R_d were related as $\sigma_k / R_d = \nu_0$, where ν_0 is the reserve coefficient for longitudinal bending (called the "strength reserve"). For $\lambda = 0$, obviously, $R_d^0 = 1400$ kgf/cm^2 was taken into account, i.e. the allowable tensile or bending stress by the main forces (*Table 5*) with a reserve factor ν_0 =1.71; for the elastic region with $\lambda \geq 100$, a constant reserve coefficient ν_0 =3.5 was assumed; for the site of $0 \leq \lambda \leq 100$, a parabolic transition between allowable stresses R_d^0 and R_d^{100} was assumed in the form of $R_d = (1400 - 0{,}0808\lambda^2)$ kgf/cm^2, which corresponded to the range of the reserve factor ν_0 =1.71…3.50. The ratio of the allowable tensile or bending stress to the variable allowable compressive stress $\omega = R_z\ (R_b) / R_d \geq 1$ was called the "longitudinal bending factor". It was tabulated in the form of a table, in which values ω in the range of 1.00 - 5.32 were given for steel St37, depending on the flexibility $\lambda = 0 - 150$.

A similar approach, i.e. "method ω", was regulated by the German railway specification for compressed timber structures with a different longitudinal bending factor ω table in the range 1.00 - 7.60 for flexures $\lambda = 0 - 160$.

The actuality of developing construction design regulations became especially urgent in the 20s of the 20th century with the beginning of industrialization of the country. The process of construction standardization began with the fact that individual departments began to implement own construction standards. Thus, in 1925, the NKVS (People's Commissariat of Internal Affairs) issued its codes. They regulated the allowable stresses for cast iron (Table 8), which were reduced by 10% for welded iron.

Table 8 – Allowable stresses for cast iron (codes of the NKVS of 1925)

Elements	Structures	Rivets		Bolts		Anchor bolts
Allowable stresses, kgf/cm^2	Bending	Shear	Crumpling	Shear	Crumpling	
	1200	1000	2000	750	1500	800

As we can see, given in the *Table. 8* allowable stresses practically coincide with the allowable stresses for steel St37 (*Table 4*).

For centrally compressed steel elements, the following calculation of allowable stresses R_k was standardized:

• for flexibility $\lambda > 105$, Euler's formula with a fivefold reserve factor was used;

• in the case of flexibility $\lambda \leq 105$, the Tetmeier formula $R_k = 0.20(3100 - 11{,}4\lambda)$ kgf/cm^2 was also recommended with a fivefold margin factor.

Instead of the given formulas, a table of allowable stress values in the range $R_k = 600 - 105$ kgf/cm^2 for flexibilities $\lambda = 10 - 200$ was recommended.

One of the first recommendations regarding the permissible deflections of steel beams was given in the NKVS Codes: "The height of the section of steel I-beams should be at least 1/32 of the span, and in this case the definition of the deflection arrow is not required, except in cases where the beams and girders, in length not less than 7 m, are building elements that give the necessary rigidity and replace the usual transverse capital walls in public buildings (for example, beams in the interfloor ceilings of industrial buildings). In this last case, the deflection arrow should not exceed 1/500 of the free span of the beams".

In 1931, the first state codes were introduced - the Unified Rules for Building Design, in 1934 - the Technical Rules for the Design of Metal Structures. The basis for their development was the research of domestic scientists, some of which are mentioned above, the existence of the Course provision, which, with changes and additions, still ensured a certain technical level and safety of buildings. The experience of American and German engineers, who were then building giant metallurgical and machine-building plants in the country, was also important. As an example, we can mention the famous American entrepreneur Albert Kahn, who in three years built 570 objects in the USSR, including the Stalingrad Tractor Plant, which was completely designed and manufactured in the USA and later transported and assembled in the city.

The Unified Rules for Building Design are a capital collection of mandatory codes, more than 200 pages long. The section of codes devoted to metal structures was compiled by the classic of domestic metal structures N. Streletsky (1885-1967). In the 1920s and 1930s, he devoted a cycle of publications [9-11] and a capital course of metal structures with a volume of about 1000 pages to the development of the general principles of the allowable stress method and the justification of the reserve factor [12].

N. Streletsky noted that the allowable stress [n] for steel is chosen to be smaller than the ultimate destructive stress, the ratio between them, called the reserve factor or the safety factor, "... is the main technical and economic data on which the consumption of metal in structures directly depends, and therefore the setting is quite justified value of this coefficient is significantly important, especially in the modern era of sharp economy of metal". Note that this provision is valid to this day. And then the classic characterized the situation in the 1930s: "However, until now, despite the importance of the safety factor, the numerical values of this factor cannot be determined mathematically; they are given rather as an average resulting general idea of the structure's operation". The capital course of metal structures [12] brought some clarity to the important question of the nature of the reserve factor: "The reserve factor does not guarantee us against destruction, but it indicates the probability of occurrence of destruction within the limits that satisfy our technical, economic and household demands. Thus, the reserve factor depends not only on the nature of stresses that develop in structures under the influence of forces, but also on the approach to this stressed state, on the execution of the structure, on the maintenance and operation of the structure, and therefore on the probability of the appearance of defects and accidental excesses in it force influences over the calculated and allowable ones".

For high-quality structures and main stresses, the natural limit is the yield strength, which is equal to $R_T = 2400$ kgf/cm^2 for steel 3 (Table 9). Thus, the reserve in relation to the usual allowable stresses accepted under the action of the main loads $[n] = 1400$ kgf/cm^2 (Table 10) is:

$$\xi_1 = \frac{R_T}{[n]} = \frac{2400}{1400} = 1.71\,. \qquad (1)$$

This value is a stock of random growth of the loads that is possible during the operation of the structure. These excesses and differences of actual stresses with calculated ones are possible from various reasons:

• actual stresses from this load are not equal to the calculated, which determines the structural correction; these deviations depend on the scheme and conditions of production;

• forceful actions may be more loads taken in the calculation; with insufficient care during repair or other special circumstances structures are often overloaded (bridges during stairs from rails or other adventures);

• the material itself may be different from standard properties that are attributed to it.

All these differences should be covered by the reserve factor. The combination of the most unfavorable values of these factors may not be covered by this factor, which is proved by the presence of catastrophes. However, the reserve factor should be constructed so that the probability of coincidence of these factors satisfies the necessary conditions for operation and safety.

Expanded tables of allowable stresses for metals marked [n] were given in the Unified Rules (Tables 10, 11). They resembled German codes in form (Table 4), but were more complete and detailed. In these Rules, the term "iron" was finally replaced by the more accurate name "steel" for the material of building structures. The introduction of the division of rolled and cast steels into grades, the mandatory mechanical characteristics of which are given in the Table 9. The most common was and remains steel St3, which used to be called (before 1924) cast iron grade *G*: temporary strength 3800 - 4500 kgf/cm^2, yield strength on average 2400 kgf/cm^2.

Table 9 – Mechanical characteristics of steels

Object	Material	Temporary strength, kgf/cm^2	The smallest yield limit, kgf/cm^2	Elongation %
Structures	Steel 3	3800 – 4500	2400*	22
	Steel 5 increased	5000 – 6000	3000	18
	Special steel	4800 – 6200	3600	20
Rivets	Steel 2	3400 – 4200	2000*	25
	Steel 3	3800 – 4500	2300*	22
	Steel increased	4500 – 5500	2700	22
	Special steel	4500 – 5500	3600	22
Note: * the yield limit was determined on an optional basis				

It is known that up to the limit of proportionality (for steel St3 equal to $R_n = 2000$ kgf/cm^2), steel works as a completely elastic isotropic body, the elongation of steel during this period is insignificant. This makes it possible to develop very high stresses in steel structures with relatively small deformations. In addition, steel in the elastic stage works most accurately according to Hooke's law, as most structures are calculated; therefore, the real and theoretical stresses in steel structures during elastic operation are the closest, which allows us to take the smallest reserve coefficients in steel structures and accept the allowable stresses very close to the limit ones. Indeed, according to the Unified Rules for Building Design, when the main loads are applied, the reserve factor in relation to the limit of proportionality is 2000/1400 = 1.43; under the action of main and accidental loads, only 2000/1700 = 1.18 was taken into reserve; in exceptional cases of unfavorable loads, it was allowed to bring the stresses to the limit of proportionality, that is, there is no reserve. The latter was possible because when the stress reaches the limit of proportionality, no destruction or damage occurs in the structure, only residual deformations begin to accumulate and steel begins to work as a not entirely elastic body.

The ability of steel to support the load is exhausted at the yield limit, therefore, the yield limit is, in any case, a level above which working stresses cannot rise. Since it is easily determined and quite constant, the reserve coefficients are calculated in relation to it; for steel 3, for which $R_T = 2400$ kgf/cm^2, they will be: at main loads, $\xi_1 = 2400/1400 \approx 1.71$, at main and accidental loads $\xi_2 = 2400/1700 \approx 1.4$. The reserve coefficients here cannot be equal to 1, since in order for the structure not to accumulate residual deformations for a long time, but to work elastically according to our calculation, the working stresses should not exceed the limit of proportionality, which are below the yield limit.

The above considerations were taken into account when justifying the set of allowable stresses for steels of different grades (Table 10). The values of allowable stresses given in the table were used for class 2 structures; for buildings of the 1st class they decreased by 10% and for buildings of the 3rd class they increased by 10%. Thus, the 1930 codes introduced a progressive classification of buildings by liability classes, which was excluded from subsequent editions of the codes and restored only 50 years later.

Static tensile or compressive stress was taken as the main allowable stress [*n*] for rolled and cast material. For the most common steel 3, the basic allowable stress, taking into account the main loads, was taken equal to $[n] = 1400$ kgf/cm^2, which coincided with the German standard for steel St37 for bridges (Table 5) and exceeded the allowable stress according to the code of the NKVS of 1925 (Table 8).

Table 10 - Basic allowable stresses for structures [*n*] of transport, hydrotechnical, industrial and public buildings (kgf/cm^2)

Stresses \ Materials	Rolled			Cast				
	Steel 3	Steel 5 increased	Steel special	Steel L1	Steel 2 lowered	Steel L2	Steel L5	Cast iron CH1
Main stresses under the action of main loads	1400	1750	2100	1200	1500	1800	2000	1000
Main stresses under the combined action of main and accidental loads	1700	2100	2500	1500	1800	2100	2500	1200

In the Unified Rules for Building Design, two calculation combinations of loads were distinguished:

• main loads that regularly operate during the operation of structures or are directly related to the purpose of the structure; these are: useful loads of normal intensity, force effects from moving loads for bridges and cranes for industrial buildings, atmospheric loads (snow load in our climate), own weight, etc.; when calculating only for these loads, a higher safety factor is adopted and the basic allowable stress is taken below (for example, for industrial structures and steel St3 $[n] = 1400$ kgf/cm^2);

• main and random loads, i.e. the coincidence of main and irregularly acting additional loads in the form of hurricane wind pressure, the effects of support draft and temperature, the useful load or the inertial effects of the moving load of the greatest possible intensity, the greatest ice on the wires, etc.; when calculating the main and accidental loads, taking into account the rare occurrence of their combination, a smaller safety factor and a higher basic allowable stress are taken (for example, for industrial structures and steel St3 $[n] = 1700$ kgf/cm^2).

This classification has been formulated and developed for the first time, it is associated with the German approach for bridges (Table 4), and implicitly takes into account the different probability of the combined action of random loads. Considering these combinations of loads from modern positions, it is possible to note the absence in the list of useful loads, loads on the floor, classification of wind load as additional loads, emphasis on dynamic effects and shocks, some of which are now related to emergency effects.

In the Unified Rules, for the first time, detailed recommendations were given for crane, snow and wind loads acting on any building structures, the consideration of which is beyond the scope of this article. The evolution of the development of standardization of these loads is described in detail in the author's articles [21-23].

Other allowable stresses for various deformed states of structures, rivets and bolts, called "derivatives", are listed in Table 11, they are connected by the corresponding transition coefficients to the main allowable stresses.

For centrally compressed elements, the Unified Rules were based on an approach similar to the German standards, but using the inverse coefficient of longitudinal bending $\varphi = 1/\omega$ (recall that this coefficient was proposed by F. Yasinsky). Therefore, the adjusted allowable stress $[n]_{kr} = \varphi[n]$ was introduced into the calculation of compressed elements. The coefficient φ was determined using the coefficient of reduction of the allowable stress during longitudinal bending $\varphi' = R_{кр} / R_T$ (theoretical) and the coefficient φ'' that took into account additional eccentricities and other side circumstances. The final coefficient curve $\varphi - \varphi'/\varphi'' = R_{кр} / R_T\, \varphi''$ was given in the Unified Rules in the form of a simple table, in particular, for steel 3 in the range of values 1.00 - 0.191 for flexibilities 0 - 200, close to modern regulatory data.

By its very nature, the reserve factor must be time-dependent. But in the 30s of the last century, this dependence had not yet been discovered and was not regulated in the Unified Rules. Therefore, N. Streletsky involved in this a qualitative analysis of two classes of accidental hazards for structures during their service.

1. Accidents are external, which do not depend on the state of the building, but only on their care, which are accidental loads during repairs, unusual situations, etc. Their analysis shows that the fewer years a structure has been in service, the less likely it is that something will happen to it.

2. Contingencies depending on the condition of the building, for example, on the presence of defects. The probability of the appearance of these accidents increases slightly over time, because the condition of the structure deteriorates and it wears out.

Table 11 - Derivative allowable stresses for rolled materials (kgf/cm^2)

Objects	Materials and allowable stresses			Steel 3		Steel 5 increased		Steel special		Transition coef.
				1400	1700	1750	2100	2100	2500	
Main structures	Shear		$[t]$	1050	1270	1300	1570	1570	1880	0,75
Rivets in buildings with strong seams	Shear	*B*	$[t]_{sh}$	1100	1360	1400	1680	1680	2000	0,80
		C		950	1180	–	–	–	–	0,67
	Crumpling	*B*	$[n]_{cr}$	2800	3400	3500	4200	4200	5000	2,00
		C		2400	2900	–	–	–	–	1,70
	Breakaway	*B, C*	$[n]_{br}$	840	1020	1050	1260	1260	1500	0,60
Turned bolts	Shear		$[t]_{sh}$	1100	1360	1400	1680	1680	2000	0,80
	Crumpling		$[n]_{cr}$	2800	3400	3500	4200	4200	5000	2,00
Unsharpened bolts	Shear		$[t]_{sh}$	840	1020	–	–	–	–	0,60
	Crumpling		$[n]_{cr}$	1700	2050	–	–	–	–	1,20
Anchor bolts	Tension		$[n]_t$	1050	1270	1300	1570	1570	1880	0,75
Note: B – rivets installed in drilled holes; *C* – rivets installed in pressed and undrilled holes										

Combining both factors, it can be assumed that the real hazards at the end of a structure's service life can be dramatically reduced if it is in a reasonably good condition. This allows to increase the stress for structure. This technique was used by the National Railways of Ukraine for old bridges before their replacement: when the main loads were applied, instead of the usual stress of 1300 kgf/cm^2, 1700 kgf/cm^2 was used, and accidental loads (hurricane wind and braking) were not taken into account. Perhaps this was the answer of N. Stryletsky at the beginning of the Stakhanovites-Krivonosivites of the 30s, who introduced heavy-duty railway trains that overloaded the existing bridges.

The designation of allowable stresses under the action of repeated and variable loads was implemented in the Unified Rules differently for two possible cases.

1. The case of loads changing according to a complex periodic law, act for a short time with a small oscillation period and a small amplitude - on bridges, crane tracks, etc. The influence of these loads is not very different from the static one, it is taken into account by the fact that the loads are multiplied by a factor greater than one, called the dynamic factor, after which the calculation is carried out with the usual static allowable stresses.

2. The impact of continuously changing loads, which causes resonance and metal fatigue. The danger of resonance is removed by directly prohibiting the action of loads with an oscillation period close to the structure's oscillations. To eliminate the danger of fatigue, it is necessary that the stress is below the working strength of the structure, which in this case is the same stress limit as the yield strength for static loading. Thus, the allowable stress decreases relative to $\gamma = R_{fat} / R_t$, where R_{fat} is the working strength (fatigue limit according to Weller-Weinrauch). Since the real danger of fatigue in the structure was considered to be negligible, the effect of fatigue on principal stresses was ignored in the Uniform Rules. At the same time, the phenomenon of fatigue can dramatically affect the local stresses, which are determined by accidental defects (sharpening, cutting, riveting, etc.). Therefore, in the Unified Rules, the phenomenon of fatigue was taken into account when calculating riveted and welded joints by the corresponding reduction of allowable stresses (Table 14).

Another stock of the reserve factor was discovered during this period by the works of N.S. Stryletsky - taking into account the plastic work of the material beyond the limit of elasticity and yield. In his course [12], he showed how the reserve factor increases in the plasticity hinge of a single-span steel beam ($\xi_n = 1.5\xi$ for a rectangular section and $\xi_n = 1.17\xi$ for a I-beam section) and a two-span beam loaded with concentrated forces ($\xi_n = 1.8\xi$ for a rectangular section and $\xi_n = 1.4\xi$ for a I-beam section). For repeatedly statically indeterminate systems, the picture becomes more complicated, but the effect increases not so significantly. These developments were included in subsequent editions of the design codes.

A great deal of attention was paid in the Uniform Rules to rivet joints, which were the main ones at that time. The allowable stress of rivets for shear was assumed to be equal to 0.8 of the main allowable stress: $[t]_{cp} = 0.8[n]$. This ratio was derived from the theoretical ratio of shear and tensile stresses, but it was well confirmed by mass experience of the destruction of rivets during their static operation (Table 12). As it turned out, this stress depends relatively little on the type of rivet.

The allowable stress for the average stresses along the width of the rivet (for crumpling) was assumed to be equal to twice the main allowable stress $[n]_{см} = 2[n]$. This ratio depends on the strength of the rivet to puncture and therefore on the distance in the direction of force from the first rivet to the edge of the sheet a_1, it can be increased with an increase in this distance. If this distance is taken to be equal to $2d$, then the above recommendation corresponds quite well to the experimental results presented in the form of a generalized linear graph according to Dornan, St.Gallih, Kayser, Weidman [12]. Thus, a simple relationship $n_{см}/n=a_1/d$, where d is the diameter of the rivet, was established between the allowable crumpling stress and the distance from the extreme rivet to the end of the sheet. Commenting on this result, N.S. Streletsky concluded: "Thus, work (and destruction) to crumple is essentially a conditional term; destruction occurs not from crumpling, but from puncture, therefore the strength of the rivet depends primarily on the area of the puncture, i.e. from the distance from the rivet to the edge of the sheet" [12].

The allowable stress during the operation of rivets for breakaway was assumed to be much less than the normal strength $[n]_{отр} = 0.6[n]$ (until 1930, it was taken even less $[n]_{отр} = 0.4[n]$). Foreign tests (Prof. Wilson) on the detachment of rivet heads showed that the initial increase in external force occurs without the elongation of the rod, since all the energy goes to the balancing of the crumpling forces (compression of the sheets after cooling the rivet), only later the elongation of the rivet rod and its additional work begin for tension.

The above ratios between the allowable rivet stresses and the main allowable stress (0.8; 2; 0.6) were related to the high-quality manufacturing of structures with drilling holes for rivets (work *B*). For lower-quality installation of rivets in punctures and undrilled holes (work *C*), which worsened the operation of the connection, the Unified Rules were prescribed to reduce the above ratios to 0.67; 1.70; 0.6 and to use the corresponding allowable stresses in the calculations (Table 13).

Table 12 - Medium shear strength of rivets
(according to the International Bridge Congress of Vienna, 1928) [12]

Stave	Steel St37 (St3) kgf/cm^2		
	$R_з^T$	$R_з$	$\frac{R_з}{R}$
Machine room	1970	3390	0,84
Pneumatic	1560	3383	0,84
Manual	1510	3380	0,82
Designations in the table: $R_з^T$ – shear strength of the connection at the beginning of yield; $R_з$ – shear strength of the connection during destruction; *R* is the temporary strength of the sheet material			

Table 13 - Allowable stresses in the calculation of riveted joints (kgf/cm^2)

Loads	Stresses	Transition coef.	Material of rivets and bolts		
			Steel special	Steel 4	Steel 3
Main	Shear *B*	0,8	1700	1400	1100
	Shear *C*	0,67	It is not used	It is not used	950
	Breakaway *B and C*	0,6	1250	1050	850
	Crumpling *B*	2,0	4200	3500	2800
	Crumpling *C*	1,70	It is not used	It is not used	2400
Main and accidental	Shear *B*	0,8	2000	1700	1350
	Shear *C*	0,67	It is not used	It is not used	1150
	Breakaway *B and C*	0,6	1500	1250	1000
	Crumpling *B*	2,0	5000	4200	3400
	Crumpling *C*	1,70	It is not used	It is not used	2900

The development of electric welding in construction was reflected in the Unified Rules by the first inclusion of allowable stresses for the joints of steel structures using arc welding (Table 14). As noted by experts [12], the metal of the welds of that time (which they called "electrometal") was rather heterogeneous with the presence of bubbles, slags and oxides.

The range of spread of temporary strength was quite wide – 2500 – 4500 kgf/cm^2 – due to the influence of the specified factors. The Unified Rules required a minimum strength of 3000 kgf/cm^2 for seams, allowing 2500 kgf/cm^2 in unfavorable cases, which indicated the uncertainty of the given figures. The yield limit of the deposited metal was also different, but relatively high: according to American, Belgian, German and domestic data (E. Paton, Kyiv [13]), the ratio of yield limit to temporary strength was 0.7–0.9 for seams, then as for the base metal of steel 3, it was constantly equal to 0.66. The increased yield strength interferes with the operation of the weld, as it contributes to the development of local overstresses in the weld metal, which is heterogeneous and associated with shrinkage stresses. The increase in the yield strength corresponded to a decrease in the relative elongation to 10%, and in fact, when welding with bare electrodes (there was such a thing then...) the elongation was significantly lower. These shortcomings forced the developers of the Unified Rules to adopt relatively low values of allowable stresses for welds (Table 14). The lower strength of welding to tension was explained by the influence of unwelded seams. The above ratios between the allowable rivet stresses and the main allowable stress (0.8; 2; 0.6) were related to the high-quality manufacturing of structures with drilling holes for rivets (work B). For lower-quality installation of rivets in punctures and undrilled holes (work C), which worsened the operation of the connection, the Unified Rules were prescribed to reduce the above ratios to 0.67; 1.70; 0.6 and to use the corresponding allowable stresses in the calculations (Table 13).

Under the action of the main and accidental loads, the allowable stresses of the welds increased by 20%, under the constant action of vibration loads they decreased by 30%. For corner seams, the allowable stress [t] = 800 kgf/cm^2 (in German codes – 700 kgf/cm^2) was regulated. Tables of calculated permissible forces per 1 sm of butt, flank and frontal seams were included in the Unified Rules, reference books and textbooks [12].

Listed in the Table 14 stresses were allowed if the weld metal had the following strength parameters: temporary tensile and shear strength – 3000 and 2400 kgf/cm^2, respectively; relative elongation during stretching is 10%.

Table 14 – Allowable stresses for welds (kg/cm^2)

Stresses	Designations	The Unified Rules for Building Design				Foreign codes [12]	
		Under the action of the main loads		Under the simultaneous action of all loads			
		Static load	Vibration load	Static load	Vibration load	USA	Germany
Compression	$[n]_{\text{св.}-}$	1000	670	1200	800	1055	1050
Tension	$[n]_{\text{св.}+}$	900	600	1100	750	914	910
Shear	$[t]_{\text{св}}$	720	450	870	680	795	700

The reserve coefficients of welded joints were as follows:

- in relation to the yield limit R_T =0.8R=2400 kgf/cm^2:
  - in tension ξ_+ = 2400/900 = 2.65;
  - for compression ξ_- = 2400/1000 = 2.40;
- in relation to the strength limit:
  - in tension ξ_+ = 3000/900 = 3.30;
  - on compression ξ_- = 3000/1000 = 3.00

Thus, the reserve coefficients of the Unified Standards for welded joints were significantly higher than the reserve coefficients for the base metal of steel St3, which is explained by the above-mentioned imperfections of welded joints.

As can be seen from the Table 14, domestic allowable stresses of welds in the 1930s practically coincided with the standards of developed foreign countries.

In the following years, after the adoption of the Unified Rules, active research was conducted aimed at improving the methodology of allowable stresses in the part of load standardization [14], taking into account the plastic work of steel in structures [15], the work and calculation of welded and riveted joints [13,16], statistical evaluation strength of steels [19]. These results were included in the Technical conditions for the design of metal structures (1940), the main provisions of which were summarized in the second edition of the Course of metal structures "Part 1. Fundamentals of the design of metal structures", compiled by N.S. Streletsky [20].

These codes reflected the development of construction steel standardization according to the then OST 2897, according to which steel was divided into six grades (Table 15). Starting from 1935, the measurement of the yield point became mandatory for steels, and from 1940 - the assessment of the impact strength of steels.

The most important construction steel remained St3 low-carbon steel, stable in quality and well-developed by metallurgical plants that were actively built in the 1930s. The first statistical studies of this steel (1600 samples [19]) revealed a fairly favorable average value of 2400 kgf/cm^2 and a mean square deviation (standard) of 150 kgf/cm^2 for the yield strength. At the same time, a certain part of the steel St3 that was smelted did not meet the standard requirements, it was transferred to the lower St2, St1, St0 grades and allowed to be used, because at that time it was believed that "... all products of our factories must be used in accordance with their quality " [20].

During this period, low-carbon steels similar to domestic steel grades of St3 and St5 were used abroad (Table 16 [20]).

Table 15 - Characteristics of steel grades according to OST 2897

Steel grades	Temporary strength, kgf/cm^2	Yield strength, kgf/cm^2	Relative elongation, %	Bend in cold state	Allowable stresses, kgf/cm^2	
					$[\sigma]_T$	$[\sigma]_{TI}$
St 1 normal	3200 – 4000	–	28	*d =0*	1200	1450
St 2 normal	3400 – 4100	2100	26	*d =0*	1200	1450
St 3 normal	3800 – 4500	2200	22	*d =0,5δ*	1400	1700
St 3 increased	3280 – 4500	2200	24	*d =0*	1400	1700
St 4 normal	4200 – 5000	2500	20	*d =2δ*	1400	1700
St 5 normal	5000 – 6000	2900	16	*d =3δ*	1750	2100
St 5 increased	5000 – 6000	2900	18	*d =2δ*	1750	2100
St 6 normal	6000 – 7000	3100	12	–	–	–
Note: The bend test is performed for a strip with a thickness of *δ* that bends around a rod with a diameter of *d* or close to it (OST 1863)						

Table 16 - The main characteristics of domestic and foreign steels (1940)

Country	Steel grades	Temporary strength, kgf/cm^2	Yield strength, kgf/cm^2	Relative elongation, %
USSR	St3	3800 – 4200	2200 – 2300	22
	St 5 increased	5000 – 6000	2900	18
Germany	St37	3700 – 4500	2400	20
	St48	4800 – 5800	2900	18
USA	A 7–9	3900 – 4600	2100	22
	A 14	4200 – 5100	2300	22
France	A42	4200	2400	25
	Rombeau	3500 – 4000	2400	30
	Dorombeau	5400 – 6400	3000	20
England	BBS 15	4100 – 5200	–	20

At the end of the 1930s, the development of high-strength steel began. It was a chrome-manganese-copper steel, called DS steel, intended for implementation in the construction of the Palace of Soviets This colossal structure with a height of more than 400 m was planned to be erected in Moscow on the site of the destroyed Cathedral of Christ the Savior. These plans did not have to be implemented, because the Second World War began. DS steel had high mechanical characteristics: temporary strength of 5200 - 6200 kgf/cm^2; yield strength at least 3600 kgf/cm^2: relative elongation 20%.

The technical design conditions of 1940 introduced a new designation for allowable stresses: $[\sigma]$ instead of $[n]$, but left the same allowable stresses for the main steel St3 as before: $[\sigma]_T$ = 1400 kgf/cm^2, taking into account the main loads (reserve factor 1.7 in relation to the calculated yield strength σ_t=2400 kgf/cm^2) and $[\sigma]_{TI}$=1700 kgf/cm^2 taking into account additional loads (reserve factor 1.4) (Table 15). Thus, the steel St3 had a reserve value of 1000 kgf/cm^2 for the first allowable stress and 700 kgf/cm^2 for the second allowable stress. The allowable stresses for the steel St2 were taken equal to $[\sigma]_T$=1200 kgf/cm^2 and $[\sigma]_{TI}$=1450 kgf/cm^2, which at the calculated yield strength σ_t=2200 kgf/cm^2 left 1000 kgf/cm^2 in case of an increase in working stresses, respectively 1000 kgf/cm^2 and 750 kgf/cm^2.

For the steel St1, the same allowable stresses were adopted (Table 15), despite the significantly lower value of the yield strength. Thus, this steel had smaller reserves of strength and was not used for responsible structures. In order for the number of different allowable stresses to be small, the allowable stresses for the steel St4 were left as they were for the steel St3. Increased strength (the yield strength according to OST 2857 is equal to 2500 kgf/cm^2) led to greater reserves of strength of this steel, which partially compensated for its lower viscosity. For the steel St5, the allowable stresses were increased compared to the steel St3, based on the principle of constancy of the reserve factor, and were taken as 1750 kgf/cm^2 and 2100 kgf/cm^2. N.S. Streletsky noted [20] that this principle is not entirely consistent, because it leaves additional reserves for stronger material, although they are useful in view of the lower viscosity and standardization of high-strength steels.

Research into the operation of riveted joints continued during this period in Germany, Switzerland, USA and USSR [16]. They confirmed that the actual work of this connection on a shear is quite complex and includes the following main stages: elastic work due to friction between the connecting elements; shift by the size of the gap between the rivet and the hole; elastic-plastic work of the rivet rod. Clarifying ideas about the actual operation of riveted joints and improving the quality of factory rivets made it possible to increase the transition coefficients for rivets in drilled holes (rivets *B*) up to 0.9 (according to German research data, this ratio can reach 1.1

[20]), for rivets in punched holes up to 0.7 with a corresponding increase in allowable shear stress (Table 17).

As shown above, the allowable crumpling stress of a riveted joint $[\sigma]_{cm}$ depends on the distance between the rivet and the edge of the joined element. At the standard distance $a_1 = 2d$ $[\sigma]_{cm} = 2[\sigma]$, which was taken into account in the previous codes and new codes for rivets *C* (Table 17). The technical conditions of 1940 allowed an increase in stress $[\sigma]_{cm}$ to 2.5 [σ] for rivets *B* with a corresponding increase in the distance a_1 (Table 17). The allowable tension of the rivets remained unchanged for tension and changed significantly in the calculation for vibration load.

Table 17 - Evolution of allowable stresses of riveted joints (St3)

Codes, years	Type of rivets	Shear, kgf/cm^2		Crumpling, kgf/cm^2	
		Main loads	Main and accidental loads	Main loads	Main and accidental loads
1934	Rivets *B*	1100	1350	2800	3400
	Rivets *C*	950	1150	2400	2900
1940	Rivets *B*	1250	1500	до 3500	до 4200
	Rivets *C*	1000	1200	2400	2900

The improvement of welding with the use of electrodes with thick slag-forming coatings made it possible to bring the strength of welded joints closer to the strength of the base metal. Despite this, the allowable stresses of the deposited metal, which was considered less homogeneous, were still assumed to be lower than the allowable stresses of the base metal: on tension $[\sigma]_{св+} = 0.8\,[\sigma]$; for compression $[\sigma]_{св-} = 0.9\,[\sigma]$; on shear $[\tau]_{св} = 0.7\,[\sigma]$, i.e. its remained at the level of the 1934 codes (Table 14). As a result, the welded joints had larger reserve coefficients compared to the temporary strength than the base metal: for tension 3.4; on compression 3.0 (base metal 2.7). This was explained by the wide spread of the test results of the joints, which depended on the qualification of the welder and were very sensitive to the contingencies of the welding process. The increase in the reserve factor for tension accounted for the greater impact of these accidents (such as undercooking) in tension.

Taking into account the large amount of results of domestic and foreign research on the operation of welded joints under repeated loads, the recommendations regarding the vibration strength of these progressive joints were significantly expanded in the Technical Conditions of 1940.

At the end of the 1930s, a real scientific attack on the reserve factor took place, which was carried out by the classic of domestic metal structures, N. Streletsky and foreign specialists [17-19]. It is known that the main principle of engineering work and engineering calculation is the condition of indestructibility, according to which the greatest effort acting in the building (structure) during its service life must be less than or, in the extreme case, equal to the smallest possible maximum strength of the construction material during this time:

$$\max S^{fact.\lim}_{structure.} = k \cdot S_{design.force.} \leq \\ \leq c \cdot S^{material.}_{normative.} \min S^{fact.\lim}_{material} \quad (2)$$

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

N. Streletskyi was the first to note that the fulfillment of the specified inequality can be predicted only with a certain probability. In his small in scope, but extremely meaningful work "Fundamentals of statistical accounting of the reserve coefficient of structures strenght" [19], he substantiated the conclusion that following a statistical path, studying and comparing the facts of the operation of a homogeneous group of structures and material in structures, it is possible to establish the law of the appearance of these factors and extrapolate this law for the future, if there are sufficient grounds for this.

N. Streletsky first presented the reserve factor as the product of its constituent components:

$$k = k_1 k_2 ... k_n = \prod k_i \, . \quad (3)$$

The reserve factor structure in the form of a product of multipliers is called the canonical reserve factor structure. This structure is convenient because the number of multipliers in it can always be set depending on the course of the survey. This structure corresponds most closely to the practice of calculation, according to which specific cases of structure's work and material are traditionally evaluated by coefficients, which are included in the calculation as multipliers.

N. Streletsky rightly showed that each of the coefficients characterizing any particular feature of construction work depends on a large number of reasons and circumstances that may occur during construction service, and therefore can best be described using a statistical method.

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

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

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

$$G = 1 - \omega_1 \cdot \omega_2 \,, \tag{4}$$

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

Therefore, this value G was called by N. Streletsky as a guarantee of the non-destructiveness of the structure. He emphasizes that the value of the guarantee of non-destructiveness is a conditional value associated with the fulfillment of the condition of non-destructiveness (2).

Back in 1938, N. Streletsky was the first to determine the numerical values of the indestructibility guarantee. Steel trusses under a cold reinforced concrete roof for the Moscow region were considered. Statistical data on loads from snow and wind for 35 years (1885...1930) were taken into account. The obtained values of the indestructibility guarantee were close to unity: $G = 1 - 5.5 \cdot 10^{-8}$ and $G = 1 - 8.5 \cdot 10^{-8}$. Therefore, steel trusses calculated according to the codes of 1934 had very high values of the guarantee of indestructibility.

These considerations were substantiated by N. Streletsky and were taken into account by the Narkombud of the USSR in the midst of the Second World War in 1942. As a result, the allowable stresses for steel structures were increased by 200 kgf/cm^2 and were accepted for structures made of steel Oc equal to 1400 kgf/cm^2 (by 15%!) and for structures made of steel St3 equal to 1600 kgf/cm^2 (by 12.5%!) while maintaining without changing the mechanical characteristics of steels (normalized minimum yield strength for steel Oc 1900 kgf/cm^2 and for steel St3 2200 kgf/cm^2). It was a real feat of N. Streletsky and the triumph of scientific and technical thought, when "at the tip of a pen" such a significant increase in the design strength of steel was achieved especially necessary during the war. These changes reduced the reserve factor from 1.58 to 1.36 (under the main loads). Despite the rather small value of this factor, which was a record, such an increase in stress was apparently possible, as the corresponding analysis showed. The value of the indestructibility guarantees with a reserve factor of 1.36 was, according to various estimates, $G = 1\text{-}6 \cdot 10^{-7}$; $1\text{-}3 \cdot 10^{-6}$; $1\text{-}5 \cdot 10^{-6}$. All the values of G remained quite close to unity, they are less than unity by only parts per million for light-type metal structures.

By 1930-40, it became increasingly obvious that the method of allowable stresses, which was based on the principle of calculation based on the working state, had exhausted itself and required replacement, which happened in 1955 with the transition to the method of limit states. One of the apologists of the allowable stress method, S. Bernstein, whose classic book "Essays on the History of Construction Mechanics" [2] was mentioned above, characterized the transition period from the allowable stress method (which he called the "working state principle") to the limit states method: "We believe that the reconstruction of the doctrine of the strength of materials, which is coming now, should not be understood as a rejection of the principle of the working state and a complete transition to the principle of the limit state. It is more correct to consider the next new - third - era in this science as a synthesis of both directions, with each of them given its deserved place in strength calculations. It seems to us that a correct understanding of the dialectical duality of the new era in the strength of materials can contribute to its successful development and prevent premature erroneous judgments."

Conclusions

A sequential review of the calculation methods of building structures was carried out, starting with the classical studies of the 17th and 18th centuries until the middle of the 20th century, when the allowable stress method was dominant. The Urgent position, which regulated construction activities from the middle of the 19th century to the beginning of the 20th century, are considered in detail. A comparison of the domestic method of allowable stresses with foreign and modern design codes is carried out. The general conclusion is substantiated that the method of allowable stresses, despite all its shortcomings, over the 200-year period of application still ensured the necessary reliability and safety of construction objects around the world. In the background of the method, a large and valuable baggage of scientific results was acquired, which were later laid as the basis of a new method of limit states.

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