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RESEARCH ARTICLE

COMPARISON OF ANALYSIS RESULTS OF STEEL FRAME OF FRAME BUILDINGS IN ACCORDANC WITH SP RK EN 1998-1:2004/2012 AND SP RK 2.03-30-2017*

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Abstract. *In the Republic of Kazakhstan, a transition has been implemented to new structural design standards, SP RK EN, which are harmonized with the Eurocodes and include National Annexes. At present, in the field of seismic design in Kazakhstan, two regulatory documents coexist in parallel: SP RK 2.03-30-2017* "Construction in Seismic Regions" and SP RK EN 1998-1:2004/2012 "Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings," along with their respective Scientific and Technical Guidelines (STGs). According to Clause 1.4(5) of SP RK EN 1990:2002+A1:2005/2011 (the fundamental basis of the Eurocode system), alternative design rules differing from those specified in EN 1990 may be applied, provided that such alternative rules comply with the fundamental principles and ensure structural safety, serviceability, and durability at least equivalent to those required by the Eurocodes. This study compares the results of analyses of reinforced steel moment-resisting frames of frame-type buildings, obtained using the design provisions of SP RK 2.03-30-2017* and SP RK EN 1998-1, in order to verify the compliance of SP RK 2.03-30-2017* with Clause 1.4(5) of SP RK EN 1990. The objective of this study is to assess the compliance of the requirements of SP RK 2.03-30-2017* with the provisions of Clause 1.4(5) of SP RK EN 1990 through a comparative analysis of the results of three-dimensional structural analysis of steel moment-resisting frame buildings performed in accordance with the current national design code and the new Eurocode-based regulatory documents. The study is based on the numerical modeling of a three-dimensional steel frame using two design approaches, namely the current national code and the newly adopted Eurocode-based standards. A comparative evaluation of structural displacements, internal forces, stiffness characteristics, and the principal design parameters of the load-bearing frame elements was carried out. The obtained results make it possible to determine the degree of agreement between the structural response predicted by SP RK 2.03-30-2017* and SP RK EN 1998-1, as well as to evaluate the applicability of the national code as an alternative design approach for buildings located in seismic regions. The scientific novelty of the study lies in the comprehensive comparative assessment of the equivalence of the new regulatory framework for the three-dimensional analysis of steel frame buildings subjected to seismic loading. The findings provide a scientific basis for the further harmonization and updating of Kazakhstan's national seismic design standards in accordance with the Eurocodes and contribute to the improvement of engineering practices for earthquake-resistant structural design.*

Keywords: *alternative design rules, structural safety, frame building, reliability, durability, Eurocodes.*

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ҒЫЛЫМИ МАҚАЛА

ҚҰРЫЛЫС НОРМАЛАРЫНА ҚР ЕЖ EN 1998-1:2004/2012 ЖӘНЕ ҚР ЕЖ 2.03-30-2017* СӘЙКЕС ҚАҢҚАЛЫ ҒИМАРАТТАРДЫҢ БОЛАТ ҚАҢҚАЛАРЫ ҮШІН ЕСЕПТЕУ НӘТИЖЕЛЕРІН САЛЫСТЫРУ

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Аңдатпа. *Қазақстанда еурокодтарға ұқсас, Ұлттық қосымшалары бар ҚР ЕЖ EN жаңа құрылыстарды жобалау стандарттарына көшу жүзеге асырылды. Қазіргі уақытта Қазақстанда сейсмикалық жобалау саласында екі нормативтік құжат қатар жұмыс істейді. Бұл ҚР ЕЖ 2.03-30-2017* «Сейсмикалық аймақтағы құрылыс» және ҚР ЕЖ EN 1998-1:2004/2012 «Жер сілкінісіне төзімді құрылымдарды жобалау. 1-бөлім. Жалпы ережелер, сейсмикалық әрекеттер және ғимараттарға арналған ережелер» және оларға ҒТҚ (ғылыми және техникалық оқу құралдары) оларға. ҚР ЕЖ EN 1990:2002+A1:2005/2011 (Еврокодтардың негізгі құжаты) 1.4(5) тармағына сәйкес EN 1990 ережелерінен ерекшеленетін альтернативті жобалау ережелері, егер альтернативті ережелер негізгі қағидаттарға сәйкес келсе және құрылымдық қауіпсіздікті, жұмысқа қабілеттілікті және еурокодтарда қарастырылғандарға баламалы ұзақ мерзімділікті қамтамасыз етсе, қолданылуы мүмкін. Бұл жұмыста ҚР ЕЖ 2.03-30-2017* 1990 ҚР ЕЖ EN 1990 1.4(5) тармағына сәйкестігін тексеру үшін ҚР ЕЖ 2.03-30-2017* және ҚР ЕЖ EN 1998-1 жобалау стандарттары бойынша алынған қаңқалық ғимараттардың моменттік металл қаңқаларын есептеу нәтижелері салыстырылады. Зерттеудің мақсаты қолданыстағы ұлттық нормативтік құжаттар мен Еурокодтарға сәйкестендірілген жаңа нормативтік құжаттар бойынша орындалған қаңқалы ғимараттардың болат моменттік рамаларының кеңістіктік есептеу нәтижелерін салыстырмалы талдау негізінде ҚР ЕЖ 2.03-30-2017* талаптарының ҚР ЕЖ EN 1990 стандартының 1.4(5)-тармағына сәйкестігін бағалау. Зерттеу кеңістіктік болат қаңқаны қолданыстағы және жаңа нормативтік тәсілдер бойынша сандық модельдеуге негізделген. Зерттеу барысында қаңқаның есептік орын ауыстырулары, ішкі күштері, қаттылық сипаттамалары және көтергіш элементтердің негізгі есептік параметрлері салыстырмалы түрде талданды. Алынған нәтижелер ҚР ЕЖ 2.03-30-2017* және ҚР ЕЖ EN 1998-1 бойынша анықталған есептік көрсеткіштердің сәйкестік дәрежесін бағалауға, сондай-ақ ұлттық нормаларды сейсмикалық аудандарда жобалаудың баламалы ережелері ретінде қолдану мүмкіндігін айқындауға мүмкіндік береді. Жұмыстың ғылыми жаңалығы сейсмикалық аудандардағы болат қаңқалы ғимараттарды кеңістіктік есептеу үшін жаңа нормативтік базаның баламалылығын кешенді салыстырмалы бағалауда жатыр. Зерттеу нәтижелері ұлттық нормативтік құжаттарды Еурокодтар талаптарымен одан әрі үйлестіру мен жаңғыртуға ғылыми негіз қалыптастырып, сейсмотұрақты жобалау тәжірибесін жетілдіруге ықпал етеді.*

Түйін сөздер: *альтернативті жобалау ережелері, құрылымдық қауіпсіздік, рамалық құрылыс, сенімділік, ұзақ мерзімділік, Еурокодтар.*

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НАУЧНАЯ СТАТЬЯ

СРАВНЕНИЕ РЕЗУЛЬТАТОВ РАСЧЕТА МЕТАЛЛИЧЕСКИХ РАМ КАРКАСНЫХ ЗДАНИЙ, ВЫПОЛНЕННЫХ ПО НОРМАМ СП РК EN 1998-1:2004/2012 И СП РК 2.03-30-2017*

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Аннотация. *В Казахстане осуществлен переход на новые строительные нормы проектирования СП РК EN, идентичных Еврокодам, с Национальными приложениями. На данный момент в Казахстане в области сейсмического проектирования параллельно сосуществуют два нормативных документа. Это СП РК 2.03-30-2017* «Строительство в сейсмических зонах» и СП РК EN 1998-1:2004/2012 «Проектирование сейсмостойких конструкций. Часть 1. Общие правила, сейсмические воздействия и правила для зданий» и НТП (научно-технические пособия) к ним. Согласно 1.4(5) СП РК EN 1990:2002+A1:2005/2011(основному основополагающему документу Еврокодов) альтернативные правила проектирования, отличающиеся от правил EN 1990, допускается применять, если альтернативные правила соответствуют основным принципам и обеспечивают конструктивную безопасность, эксплуатационную пригодность и долговечность, как минимум, равнозначные, предусмотренным в Еврокодах. В данной работе сравниваются результаты расчетов моментных металлических рам каркасных зданий, полученным по нормам проектирования СП РК 2.03-30-2017* и СП РК EN 1998-1 для проверки соответствия СП РК 2.03-30-2017* пункту. 1.4(5) СП РК EN 1990. Цель исследования заключается в оценке соответствия требований СП РК 2.03-30-2017* пункта 1.4(5) СП РК EN 1990 на основе сравнительного анализа результатов пространственного расчета моментных стальных рам каркасных зданий, выполненного по действующим и новым нормативным документам идентичных Еврокодам. Исследование основано на численном моделировании пространственного стального каркаса с использованием двух (действующий и новой) нормативных подходов. Выполнено сопоставление расчетных перемещений, внутренних усилий, жесткостных характеристик и основных параметров несущих элементов каркаса. Полученные результаты позволят установить степень соответствия расчетных показателей, определяемых по СП РК 2.03-30-2017* и СП РК EN 1998-1, и оценить возможность применения национальных норм в качестве альтернативных правил проектирования в сейсмических зонах. Научная новизна работы заключается в комплексной сравнительной оценке эквивалентности новой нормативной базы для пространственного расчета стальных каркасных зданий в сейсмических зонах, что создает научную основу для дальнейшей актуализации национальных норм с требованиями Еврокодов и совершенствования практики сейсмостойкого проектирования.*

Ключевые слова: *альтернативные правила проектирования, конструктивная безопасность, каркасное здание, надежность, долговечность, Еврокоды.*

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1 INTRODUCTION

As you know, the 49th step of the "National Plan – 100 concrete steps to implement five institutional reforms" provides for the introduction of a system of Eurocodes to replace outdated building codes applied since the Soviet period.

The implementation of European standards is conditioned by the tasks of comprehensively improving the quality of buildings and structures under construction, manufactured materials and structures, increasing their competitiveness in domestic and foreign markets while optimizing the consumption of labor, material, energy and financial resources.

In order to integrate internationally and increase the competitiveness of domestic specialists, as well as introduce innovations and advanced technologies, work began on reforming the regulatory framework of the construction industry in 2010 on behalf of the Government of the Republic of Kazakhstan. In the Republic of Kazakhstan, the codes of rules of the SP RK EN, identical to the Eurocodes, have been harmonized with the development of National Annexes and Regulatory and Technical Manuals, which have been in force since July 2015. The improvement of the regulatory framework of the construction industry in Kazakhstan provided for moving away from the prescriptive method of building rationing to the progressive parametric method. The parametric method of rationing, in contrast to the strict prescriptive one on which the Soviet SNiPs are based, provides freedom in choosing design solutions, building materials and technologies, without limiting designers and builders. As part of the reform of the regulatory framework of the construction industry of the Republic of Kazakhstan, it was assumed to switch to the European standards of the SP RK EN 1990-1999 identical to the Eurocodes.

Some regions of Kazakhstan are in high seismic activity, for which ensuring the stability and seismic safety of buildings and structures is one of the key tasks facing science. Frame structures with reinforced concrete diaphragms are widely used in the construction of civil and industrial buildings. In 2017, based on the developed sets of maps, new standards for earthquake-resistant construction and design of the SP of the Republic of Kazakhstan were developed. 2.03-30-2017* "Construction in seismic zones", interacting with the existing standards of the SNiP at that time [1]. This document significantly facilitated the transition to Eurocode standards, as it contained many additional parameters from them. Codes of rules of the SP of the Republic of Kazakhstan 2.03-30-2017* and the SP RK EN 1998-1 have fundamentally different scientific and methodological foundations. Design of buildings and structures in accordance with the provisions of the SP of the Republic of Kazakhstan 2.03-30-2017* It is carried out in compliance with the condition of equal strength of all elements of the structural system involved in the perception of seismic loads. When designing buildings and structures in accordance with the provisions of the SP RK EN 1998-1 and its normative and technical manuals, the rules of the method of additive design are followed, which provides for the planning of damage zones of structural systems during seismic impacts. After the complete transition to the European Standards of the fundamental documents on loads, combinatorics of load effects, calculations of metal structural elements, it became necessary to verify the relevance of the document of the SP of the Republic of Kazakhstan 2.03-30-2017* [2,3]. In addition, in the SP of the Republic of Kazakhstan 2.03-30-2017* There are many references in the text to the coefficients of the already canceled snips, which were missing from the Eurocodes. In this regard, it is an urgent task to carry out comparative calculations of frame buildings with metal frames according to the standards of the SP RK EN 1998-1:2004/2012 and the SP RK 2.03-30-2017*. Such an analysis makes it possible to identify the relevance in the calculated parameters, assess the degree of regulatory approaches and determine the scope of application of the SP of the Republic of Kazakhstan. 2.03-30-2017*.

2 MATERIALS AND METHODS

In this work, the method of comparative-system analysis according to the current design standards of the SP of the Republic of Kazakhstan is used. 2.03-30-2017* and the new regulatory framework of the SP RK EN 1998-1 for calculations of momentary metal frames of frame buildings [4].

Unlike similar studies for reinforced concrete structures, in this work special attention is paid to the specifics of the work of steel frames, including increased sensitivity to loss of stability of the

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elements, the peculiarities of the formation of plastic joints, as well as the effects of stiffness and dissipative capacity of nodal joints.

The empirical basis of the study is the finite element method, in which the movements and rotations of the nodes of the computational scheme are assumed to be the main unknowns. The calculations were performed using models that implement physical and geometric nonlinearity, which makes it possible to take into account second–order effects (P-Δ) and the general loss of stability of elements characteristic of steel structures. The calculation models are based on the approaches used by KazNIISA JSC specialists in accordance with the requirements of current regulations [5].

The research methodology includes:

- Spectral method of calculation for seismic impacts;
- Probabilistic approach to seismic hazard assessment;
- consideration of behavior coefficients and plasticity classes regulated by the SP RK EN 1998-1;
- analysis of the dissipative capacity of structures, taking into account the requirements for the strength of the elements (capacity design).

The research materials were the current regulatory documents of the SP of the Republic of Kazakhstan 2.03-30-2017* and SP RK EN 1998-1:2004/2012 and NTP RK 08-01.1-2012, NTP RK 08-01.2-2021, NTP RK 08-01.5-2013 [6].

The comparative analysis was performed according to the following parameters and criteria:

- values of calculated seismic loads (base acceleration, reliability coefficients);
- the shape and parameters of the reaction spectra (periods, gain coefficients, characteristic spectral regions);
- values of internal forces in the frame elements (bending moments, transverse forces, longitudinal forces);
- Horizontal movements and floor - to - floor misalignments;
- requirements for limit conditions (load-bearing capacity and operational suitability);
- requirements for structural parts and assemblies (rigidity, strength, stability);
- indicators of earthquake resistance and reliability of structures;

The results of comparative calculations are aimed at evaluating, improving and updating existing regulatory documents with the requirements of the new regulatory framework for earthquake-resistant construction [7].

3 RESULTS AND DISCUSSION

Input calculated data.

Data from seismic impacts on the building.

Estimated accelerations at the construction site in fractions of g: 0.42 g:

Type of soil according to seismic properties: IA;

Horizontal coefficient of behavior: 4.0;

Vertical behavior coefficient: 1.5;

The coefficient of the lower limit of the spectrum; 0.2;

The structural scheme of the object - the building being calculated is a framed metal frame with hard disks of floors and coatings in the form of a reinforced concrete monolithic slab. The 5-storey building measures 24x12 m in plan along the axes, 3 m high, and the pitch of the transverse and longitudinal frames along the axis is 6 m.

The rigidity of the frame in the transverse direction is ensured by the rigid pinching of the main columns of the frame in the foundation.

The spatial immutability of the frame elements is ensured by a system of longitudinal and transverse metal beams and monolithic reinforced concrete slabs 200 mm thick made of class B25 concrete.

Columns made of 260x14 composite box along the outer contour and 360x14 composite box along the inner contour. Beams from I-beams 25Б1 and 35Б2 STO ASChM 20-93 [8].

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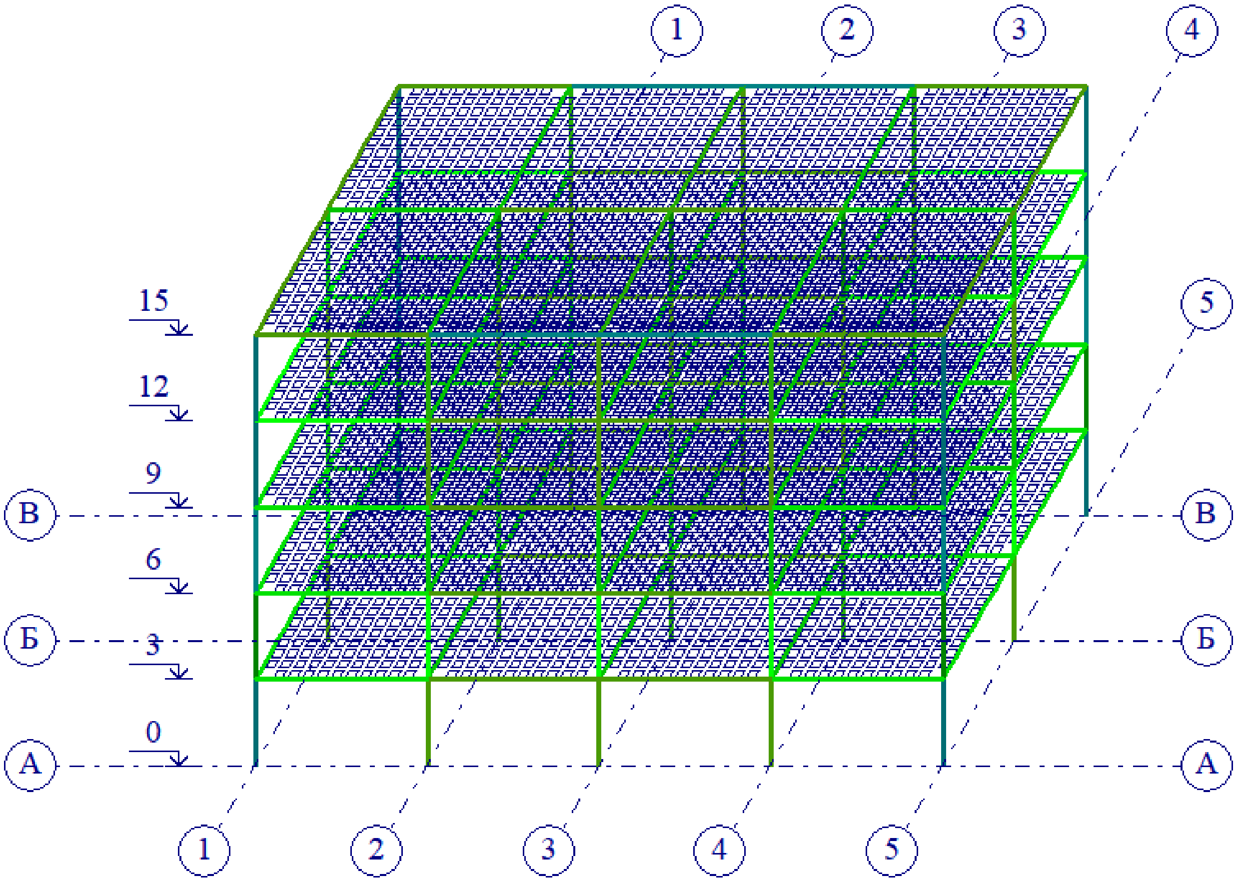

Figure 1 - Design scheme of the building. [Authors' material]

3.1. General characteristics of the calculation scheme

The calculation was performed in a spatial setting with elastic stiffness characteristics of materials for the action of basic and special combinations of loads. The calculation scheme was created and all calculations were performed using the LIRA-CAD 2022 R2.2 software package.

The calculation is based on the finite element method using the main unknown displacements and rotations of the nodes of the calculation scheme. In this regard, the idealization of the design is made in a form adapted to the use of this method, namely: the system is represented as a set of standard-type bodies (rods, plates, shells, etc.), called end elements and attached to the nodes of the design scheme [9].

The calculation scheme includes structures whose operation is essential and decisive for this system; all other structures that have little effect on the scheme of the system and its operation are excluded from the calculation scheme.

For a more correct inclusion of metal beams in the work when transmitting the calculated effects of impacts from loads, the bending stiffness of reinforced concrete slabs has been reduced without changing their shear stiffness. The angular end elements of the floor slabs are also calculated to exclude the transfer of residual bending moments from them for a more correct influence of the frame beams on the columns.

In this way, the direct transmission of bending moments from the floor slabs to the frame columns is minimized.

When creating the finite element model, the finite element "Type 10 - universal spatial core FE", "Type 44 - universal quadrangular FE" was used.

The loads were collected according to SP RK EN 1991-1-1:2002/2011 EN 1991-1-1:2002 for all calculation options.

The following downloads were used in the calculation:

1- Constant 1 (net weight of structures);

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2- Constant 2 (weight of the floor structure);
3- Constant 3 (weight of the coating structure);
4- Constant 4 (weight of the exterior fence structure);
5- Constant 5 (weight of the parapet);
6- Temporary 1 (weight of internal partitions);
7- Temporary 2 (useful);
8- Temporary 3(snow);
9- Temporary 3(snow emergency);
10- Seismic in the X direction;
11- Seismic in the Y direction;
12- Seismic in the X direction with an eccentricity;
13- Seismic in the Y direction with an eccentricity;

The bearing capacity of the cross sections in terms of strength of metal structures was determined in accordance with SP RK EN 1993-1-1:2005/2011.

The stability of the metal frames was determined by applying the design lengths to the column elements in accordance with clause 5.2.2(8) of SP RK EN 1993-1-1:2005/2011.

The estimated column lengths were calculated using the ESPRI 2020 program. The design combinations of loads (RSN) are performed in accordance with SP RK EN 1990:1002+A1:2005/2011 and NA to SP RK EN 1990:2002+A1:2005/2011 [10].

The calculated combinations of loads (RSN) were performed according to Table "B" using the formulas (6.10a) and (6.10b) with amendments to the NA dated 12/30/2021 for all calculation options [11].

3.2. The results of a comparison of the calculated data obtained for ntp rk 08-01.5-2013, performed in two versions of the acceptance of the yield strength of steel ***fy*** *and the coefficient of reserve strength* ***γov****.*

The calculation was performed in two variants of the acceptance of the yield strength of steel f_y and the coefficient of reserve strength γ_{ov}:

1. In accordance with paragraph 2.2.2.1 [6.2(3)] a) we accept for dissipative zones (beams) fy = 245 MPa and for non-dissipative zones (columns) fy = 345 MPa at γ_{ov} = 1.25.

2. In accordance with2.2.2.1 [6.2(3)] b) we accept for dissipative zones (beams) fy = 345 MPa and for non-dissipative zones (columns) fy = 345 Mpa at γ_{ov} = 1.0.

In SP RK EN 1998-1:2004/2012 and NTP RK 08-01.5-2013, there is also a third method for assigning the yield strength of steel fy and the strength reserve coefficient γ_{ov} – this is an alternative to c), where the actual yield strength of steel fy in each dissipative zone is determined by measurement results, and the strength reserve coefficient γ_{ov} is calculated for each dissipative zone.

The third condition is possible in a limited number of situations, for example, when evaluating existing buildings and is not included in the scope of this work [12].

A general mosaic of the test results for the percentage of use of the assigned ULS (load-bearing capacity) sections of the building, calculated according to the standards of the NTP RK 08-01.5-2013 with a yield strength of steel for columns fy = 345 MPa, with a yield strength of steel for beams fy = 245 MPa, and a strength reserve coefficient γ_{ov} = 1.25 are shown in Figure 2.

A mosaic of the building's ULS (load-bearing capacity) results calculated according to the standards of the NTP RK 08-01.5-2013 with a yield strength of steel for columns fy = 345 MPa, with a yield strength of steel for beams fy = 345 MPa, and a strength reserve coefficient γ_{ov} = 1.0 are shown in Figure 3.

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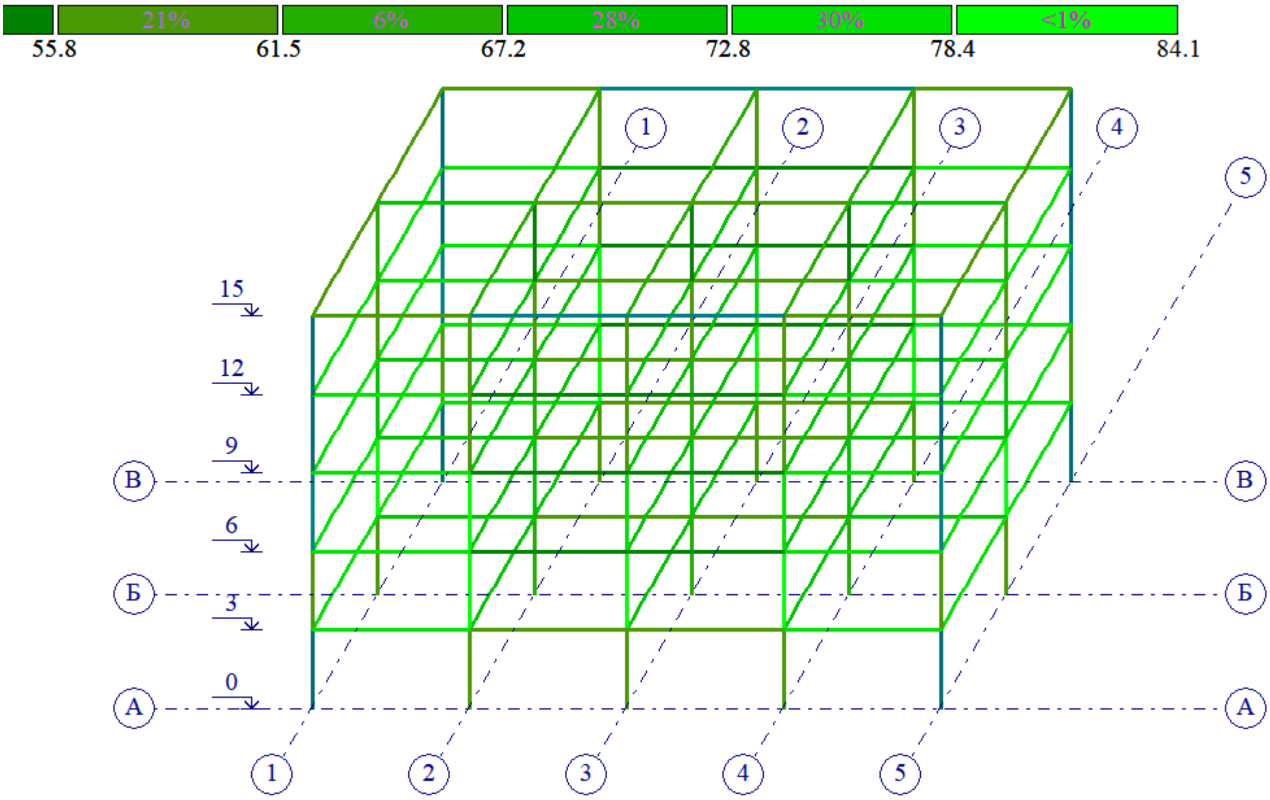

Figure 2 - General mosaic of verification results. [Authors' material]

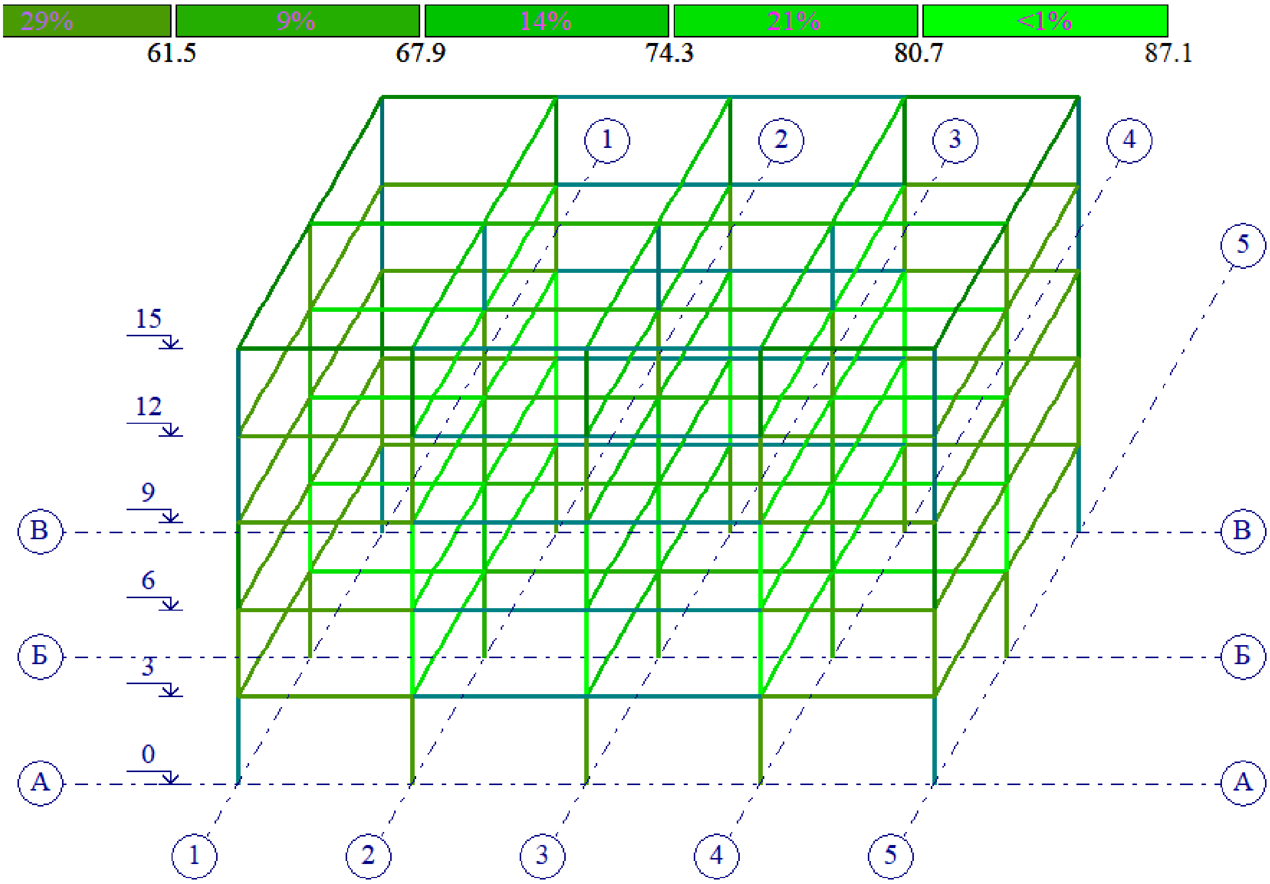

Figure 3 - General mosaic of verification results. [Authors' material]

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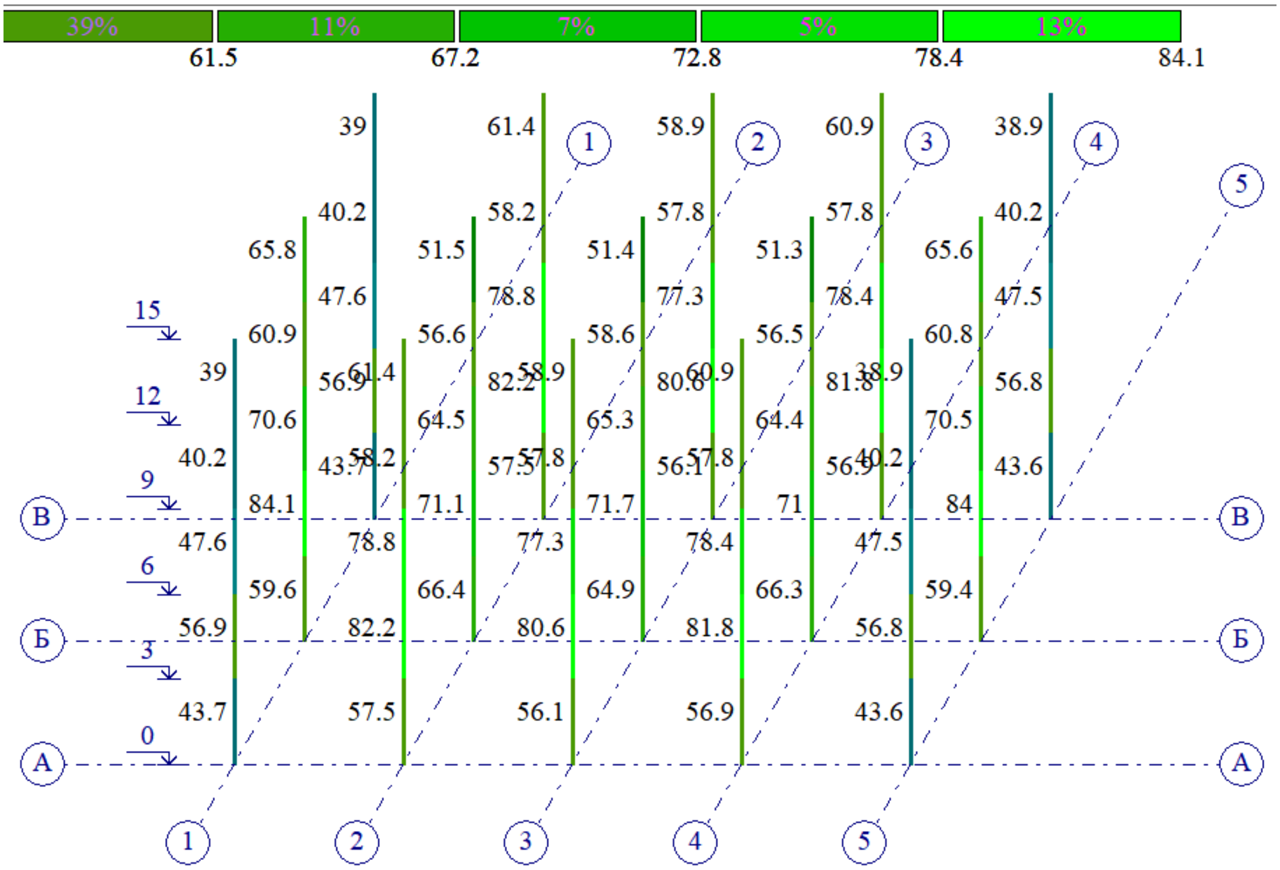

Figure 4 - General mosaic of verification results. [Authors' material]

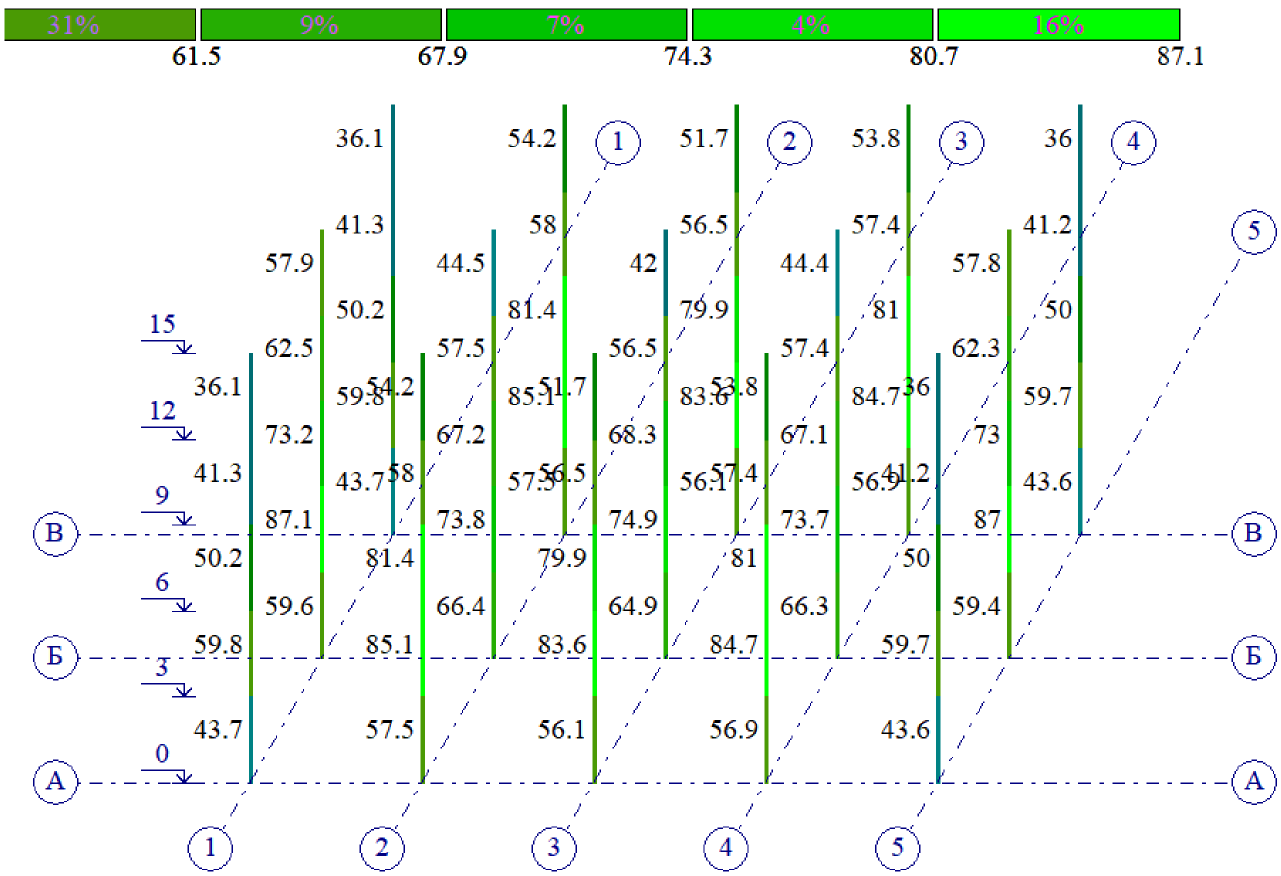

Figure 5 - General mosaic of verification results. [Authors' material]

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Mosaics of the test results for the percentage of use of the assigned column sections according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the NTP RK 08-01.5-2013 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, and a strength reserve coefficient γ_{ov} = 1.25 are shown in Figure 4.

The results of the ULS (load-bearing capacity) test of the building, calculated according to the standards of NTP RK 08-01.5-2013 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, and a strength reserve coefficient γ_{ov} = 1.0, are shown in Figure 5 [13].

Analysis of the calculation results obtained for two variants, although the difference is insignificant for columns, preference should still be given to option 1 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, and a strength reserve coefficient γ_{ov} = 1.25, shown in Fig.3.

Moreover, the use of a reduced yield strength of steel for beams creates fewer problems when designing beam-column assemblies. The comparative results of the two calculations are shown in Table 1.

Table 1 - Comparative results of two calculations according to NTP RK 08-01.5-2013, in Figures 4, 5. [Authors' material]

№	Floor number	Naming of axes	Result according to NTP RK 08-01.5-2013 in %, option 1	Result according to NTP RK 08-01.5-2013 in %, option 2	Difference of results
1	1	«1, Б»	59.6%	59.6%	0%.
2	2	«1, Б»	84.1%	87.1%	3.444%.
3	3	«1, Б»	70.6%	73.2%	3.552%.
4	4	«1, Б»	60.9%	62.5%	2.56%.
5	5	«1, Б»	65.8%	57.9%	12.006%.
6	1	«3, Б»	64.9%	64.9%	0%.
7	2	«3, Б»	71.7%	74.9%	4.272%.
8	3	«3, Б»	65.3%	68.3%	4.392%.
9	4	«3, Б»	58.6%	56.5%	3.584%.
10	5	«3, Б»	51.4%	42%	18.288%.

3.3. *The results of comparing the calculated data obtained for the sp of the republic of kazakhstan 2.03-30-2017* , executed in two versions of the acceptance of the work conditions coefficient* γ_τ.

When calculating the SP of the Republic of Kazakhstan 2.03-30-2017* p. 8.2 requires that when calculating structures for strength and stability, in addition to the coefficients of working conditions adopted in accordance with other regulatory documents, additional coefficients of working conditions should be introduced, determined according to Table 8.1.

This coefficient of working conditions γ_τ = 1.3 for elements made of C235, C245, C255 steels and γ_τ = 1.2 for elements made of other steels [14].

The use of these coefficients has been going on since the time when SNiP II was used in calculations-7-81*.

There it had the designation $m_{кр}$ and had the same parameters. SNiP II-7-81* as well as SP RK 2.03-30-2017* The SNiP II documents were assumed to be used in the calculation of metal structures-23-81* Steel Structures and SP of the Republic of Kazakhstan 5.04-23-2002 "Steel structures and design standards".

In these documents, the coefficients of the working condition were multiplied by the yield strength R_y.

In section 2, the Regulatory references of the SP of the Republic of Kazakhstan 2.03-30-2017* The note says that if the reference document is replaced (changed), then when using this standard, the replaced (changed) document should be guided. SNiP II-23-81* It was replaced by the document SP RK EN 1993-1-1:2005/2011 "Design of steel structures. Part 1-1. General rules and regulations for buildings."

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In SP RK EN 1993-1-1:2005/2011 there is no concept of coefficients of working conditions, as SP RK 5.04-23-2002.

Since this coefficient is used in calculation formulas for strength and stability calculations in the SP of the Republic of Kazakhstan 5.04-23-2002 If the yield strength of steel increases, then in SP RK EN 1993-1-1:2005/2011 it is possible to introduce it as an additional coefficient of the inverse value to the particular safety coefficients γ_{MO}, γ_{M1}, γ_{M2} [15].

A general mosaic of the test results based on the percentage of use of the assigned ULS (load-bearing capacity) sections of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 Mpa with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (0.833, 0.833, 1.04) and with a yield strength of The steels for beams f_y = 245 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (0.769, 0.769, 0.961) are shown in Figure 6.

Mosaic of the results of checking the columns according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (0.769, 0.769, 0.961) adjusted to the strength reserve coefficient γ_{ov} = 1.25 are shown in Figure 7.

Checks on the percentage of use of the assigned column sections according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (0.833, 0.833, 1.04) are shown in Figure 9.

There is no difference for the columns compared to the results obtained for calculating the columns according to the two options, and it is impossible to give preference to any option. This may be relevant when calculating beams, since an increased value of the yield strength of steel f_y can reduce the metal consumption of beams. The comparative results of the two calculations are shown in Table 2 [16].

Table 2 - Comparative results of two calculations for the SP of the Republic of Kazakhstan 2.03-30-2017*, in Figures 8, 9. [Authors' material]

№	Floor number	Naming of axes	Result of the SP of the Republic of Kazakhstan 2.03-30-2017* in %, Option 1	Result for the SP of the Republic of Kazakhstan 2.03-30-2017* in %, option 2	Difference of results
1	1	«1, Б»	50.8%	50.8%	0%.
2	2	«1, Б»	50.2%	50.2%	0%.
3	3	«1, Б»	41.5%	41.5%	0%.
4	4	«1, Б»	32.5%	32.5%	0%.
5	5	«1, Б»	30.3%	30.3%	0%.
6	1	«3, Б»	55.4%	55.4%	0%.
7	2	«3, Б»	43.8%	43.8%	0%.
8	3	«3, Б»	40%	40%	0%.
9	4	«3, Б»	36%	36%	0%.
10	5	«3, Б»	30.5%	30.5%	0%.

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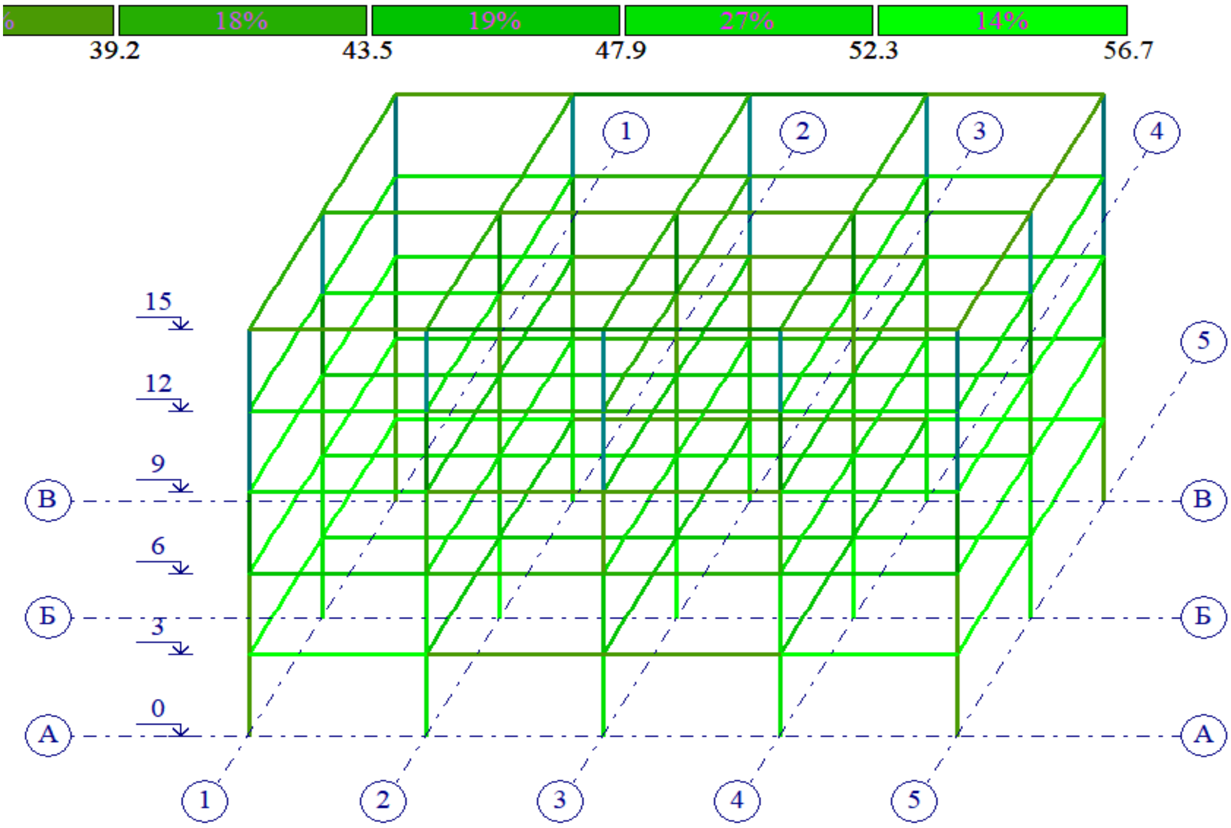

Figure 6 - General mosaic of verification results. [Authors' material]

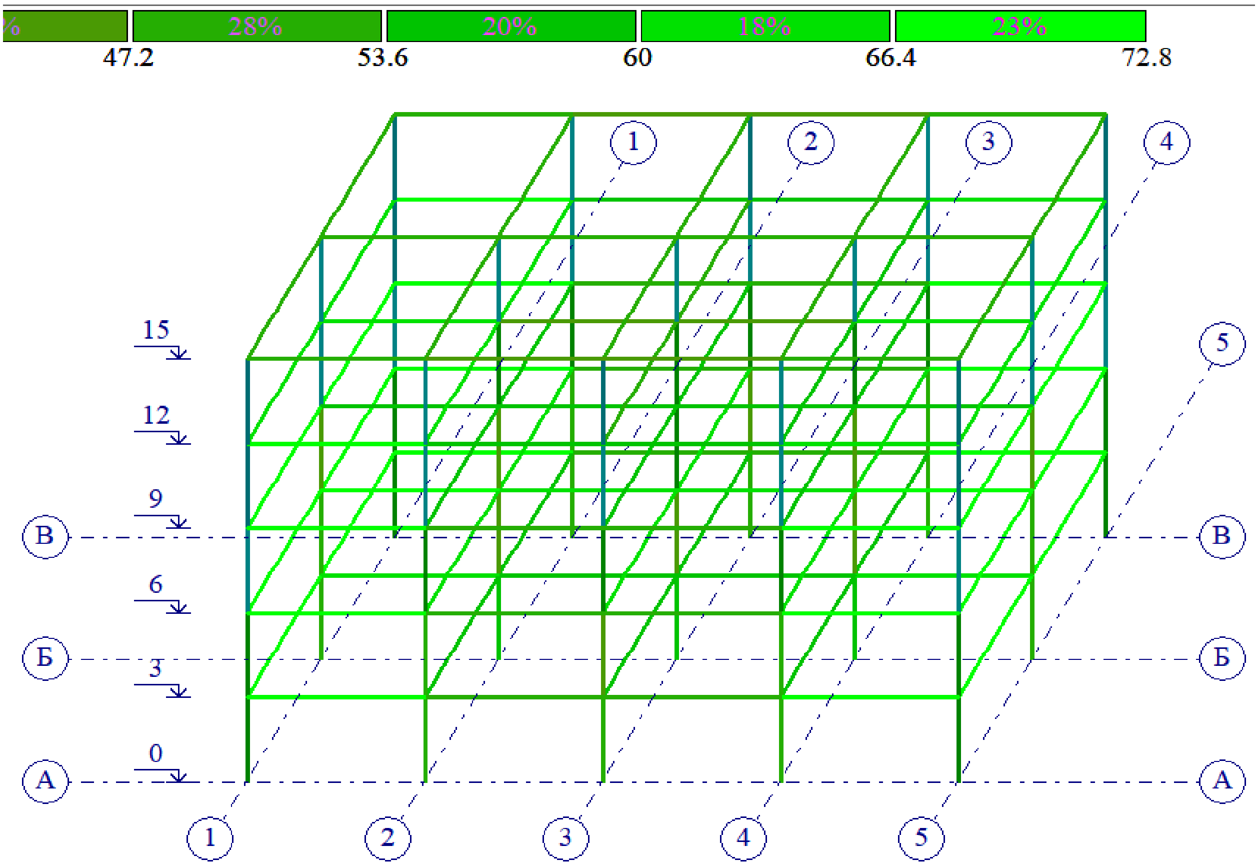

Figure 7 - General mosaic of verification results. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

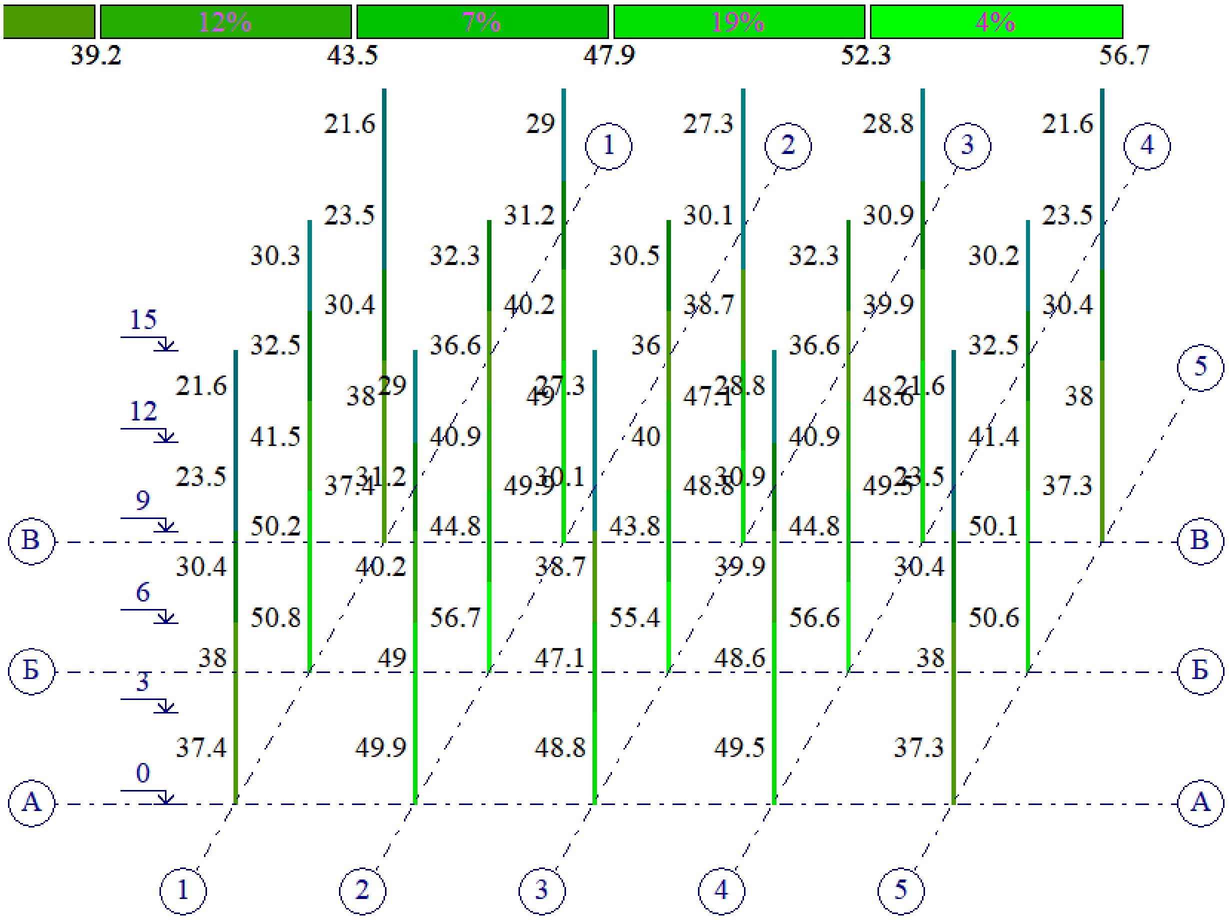

Figure 8 - General mosaic of verification results. [Authors' material]

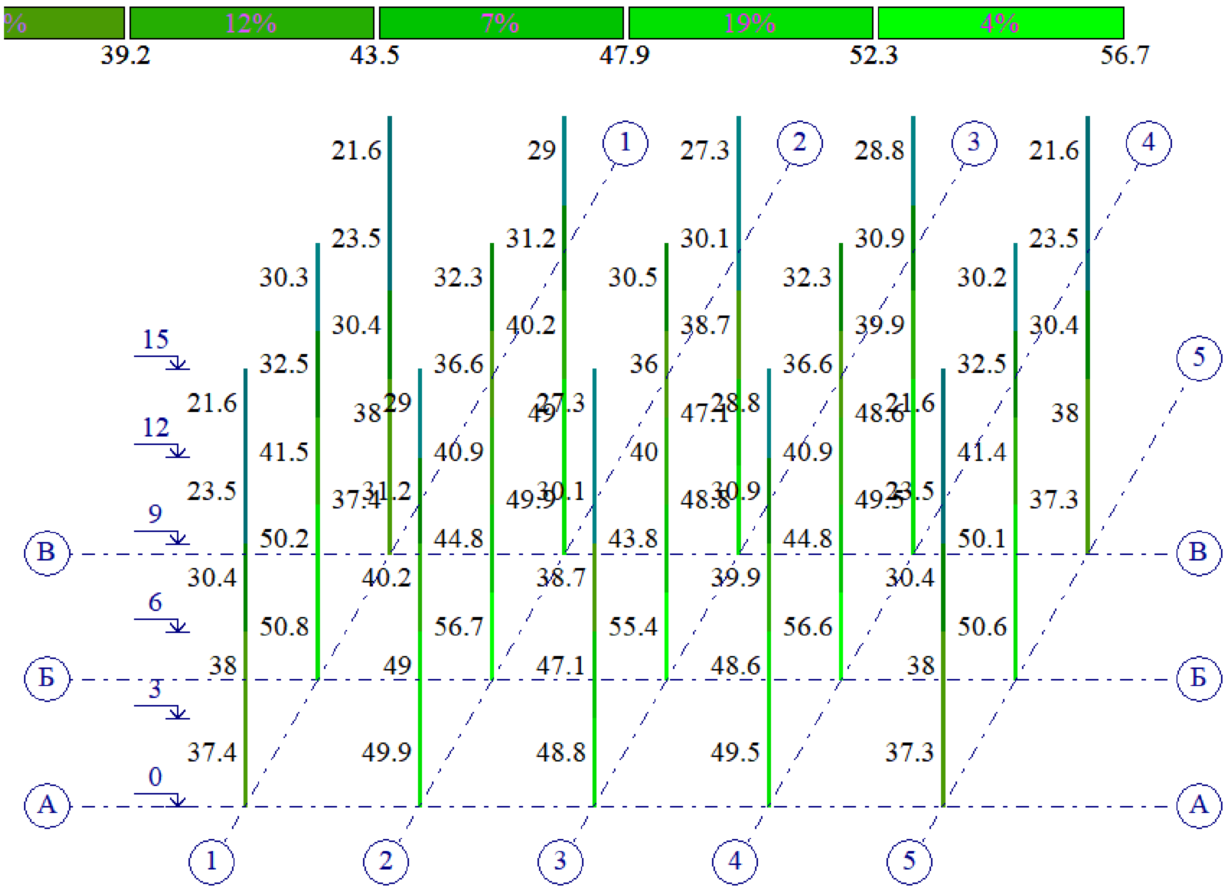

Figure 9 - General mosaic of verification results. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

3.4. *Comparisons of the calculation results of metal moment frame frames made according to NTP RK 08-01.5-2013 and according to SP RK 2.03-30-2017* (using the coefficient of working conditions $\gamma_\tau = 1.3$)*

Next, we check the difference in the bearing capacity of the most loaded columns in the calculations according to NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* .

Mosaics of the test results for the percentage of use of the assigned column sections according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the NTP RK 08-01.5-2013 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, and a strength reserve coefficient γ_{ov} = 1.25 shown in Figure 10.

The results of checking the ULS (load-bearing capacity) of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (0.833, 0.833, 1.04) shown in Figure 11.

Checks on the percentage of use of the assigned column sections according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the NTP RK 08-01.5-2013 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, and a strength reserve coefficient γ_{ov} = 1.0 shown in Figure 12.

The results of the ULS (load-bearing capacity) inspection of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (0.833, 0.833, 1.04) shown in Figure 13.

The comparative results of the two calculations are shown in Table 3.

As can be seen from the comparative results, the difference in the bearing capacity of the columns is quite large [17]. Even if we take only the most loaded columns of the 1st and 2nd floors, the difference reaches its maximum on the 2nd floor and amounts to 40.309% for the outermost columns and 38.912% for the inner columns.

Table 3 - Comparative calculation results for NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* , in Figures 10, 11. [Authors' material]

№	Floor number	Naming of axes	Result according to NTP RK 08-01.5-2013, in %	Result according to SP RK 2.03-30-2017* , in %	Difference of results
1	1	«1, Б»	59.6%	50.8%	14.765%.
2	2	«1, Б»	84.1%	50.2%	40.309%.
3	3	«1, Б»	70.6%	41.5%	41.218%.
4	4	«1, Б»	60.9%	33.5%	44.992%.
5	5	«1, Б»	65.8%	30.3%	53.951%.
6	1	«3, Б»	64.9%	55.4%	14.638%.
7	2	«3, Б»	71.7%	43.8%	38.912%
8	3	«3, Б»	65.3%	40%	38.744%.
9	4	«3, Б»	58.6%	36%	38.567%.
10	5	«3, Б»	51.4%	30.5%	41.634%.

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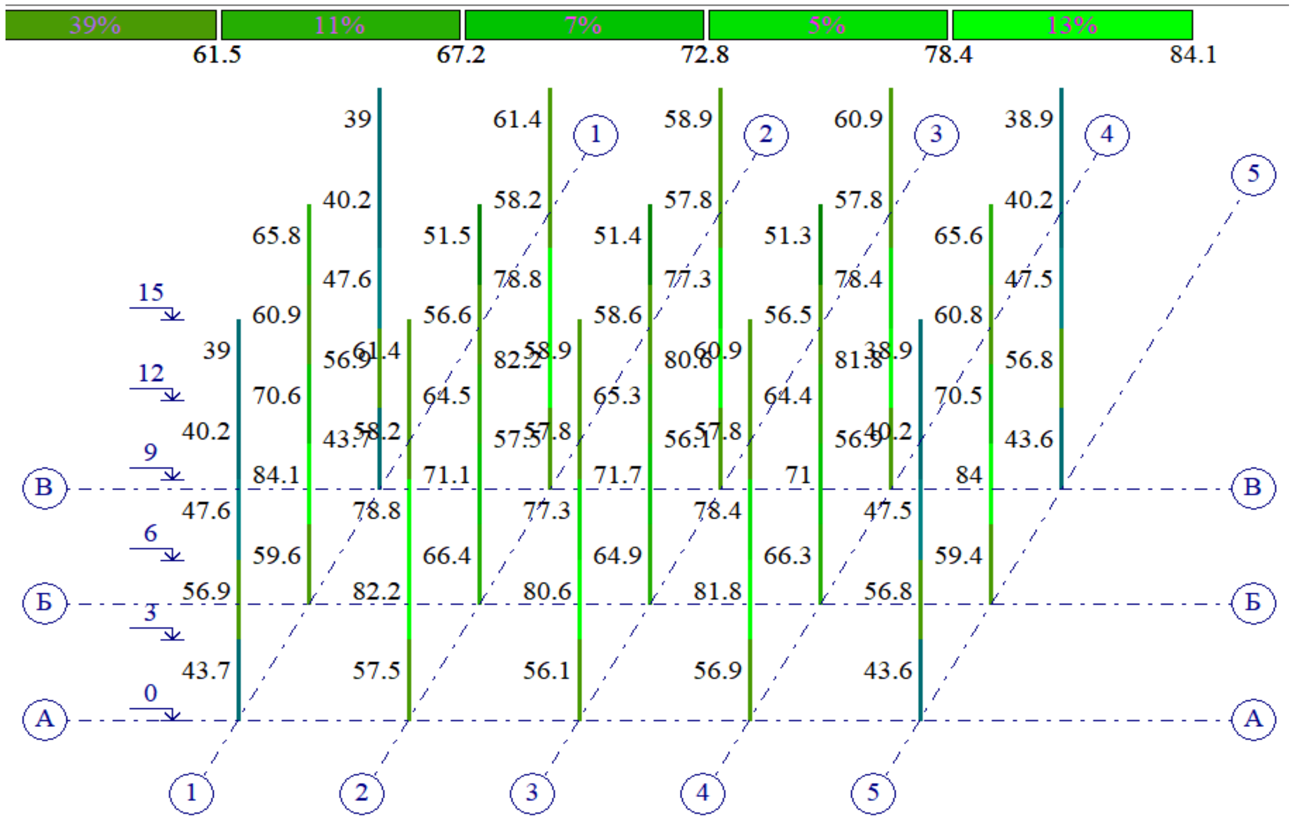

Figure 10 - Mosaic according to NTP RK 08-01.5-2013. [Authors' material]

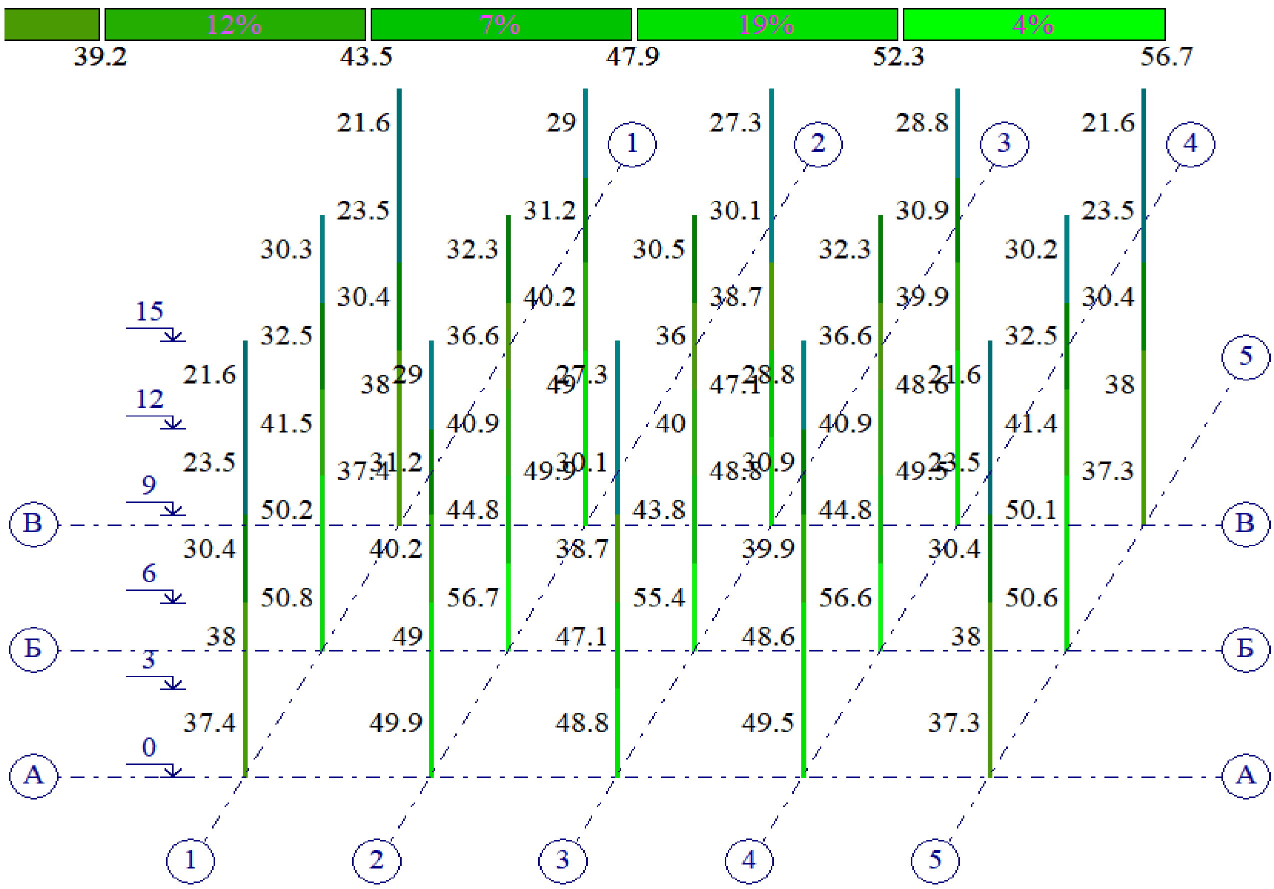

Figure 11 - Mosaic on the SP of the Republic of Kazakhstan 2.03-30-2017*. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

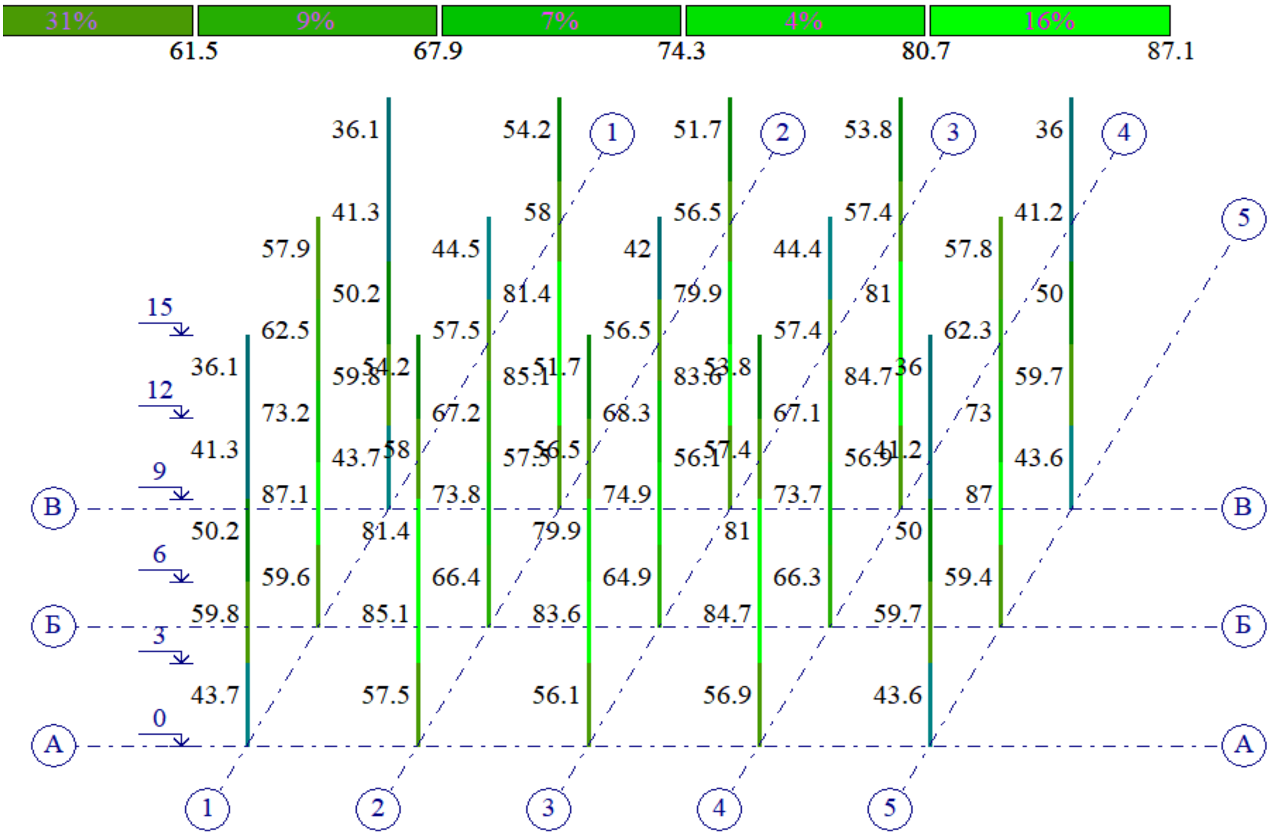

Figure 12 - Mosaic according to NTP RK 08-01.5-2013. [Authors' material]

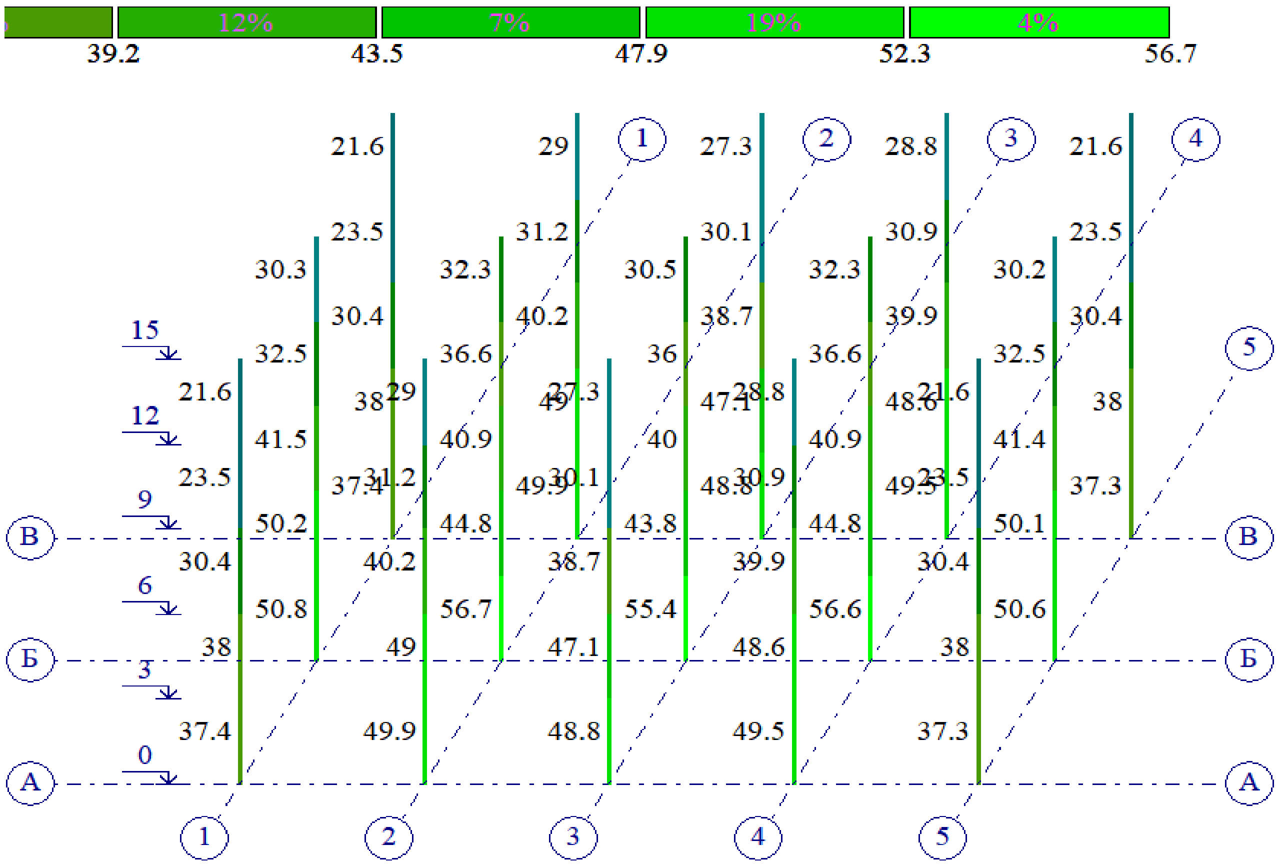

Figure 13 - Mosaic on the SP of the Republic of Kazakhstan 2.03-30-2017*. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

As can be seen from the comparative results, the difference in the bearing capacity of the columns is quite large [18]. Even if we take only the most loaded columns of the 1st and 2nd floors, the difference reaches its maximum on the 2nd floor and amounts to 46.728% for the outermost columns and 41.522% for the inner columns. The comparative results of the two calculations are shown in Table 4.

Table 4 - Comparative calculation results for NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* , in Figures 12, 13. [Authors' material]

№	Floor number	Naming of axes	Result according to NTP RK 08-01.5-2013, in %	Result according to SP RK 2.03-30-2017* , in %	Difference of results
1	1	«1, Б»	59.6%	46.9%	14.765%.
2	2	«1, Б»	87.1%	46.4%	42.365%.
3	3	«1, Б»	73.2%	38.3%	43.306%.
4	4	«1, Б»	62.5%	30%	48%.
5	5	«1, Б»	57.9%	29.9%	47.668%.
6	1	«3, Б»	64.9%	52.1%	14.638%.
7	2	«3, Б»	74.9%	43.8%	41.522%.
8	3	«3, Б»	68.3%	40%	41.435%.
9	4	«3, Б»	56.5%	36%	36.283%.
10	5	«3, Б»	42%	30.5%	28.571%.

3.5. Comparisons of the calculation results of metal moment frame frames made according to NTP RK 08-01.5-2013 and according to SP RK 2.03-30-2017 (without applying the coefficient of working conditions $\gamma_\tau = 1.3$).*

Let's see what a comparison of calculations according to the norms of NTP RK 08-01.5-2013 and SP RK will show. 2.03-30-2017* without applying the coefficient $\gamma_\tau = 1.3$, although this contradicts the instructions of the SP of the Republic of Kazakhstan 2.03-30-2017*.

A general mosaic of the test results based on the percentage of use of the assigned ULS (load-bearing capacity) sections of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) are shown in Figure 14.

Mosaic of the test results based on the percentage of use of the assigned ULS (load-bearing capacity) sections of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) are shown in Figure 15.

The non-passage of the beams in terms of bearing capacity by 9% can be ignored, since it is less than 10%.

It also does not affect the effect of gravitational and seismic loads on the columns.

Building ULS (load-bearing capacity) checks, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 Mpa with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) and with a yield strength of steel for beams f_y = 245 MPa, the safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) are shown in Figure 16.

Results on the ULS (load-bearing capacity) of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) shown in Figure 17.

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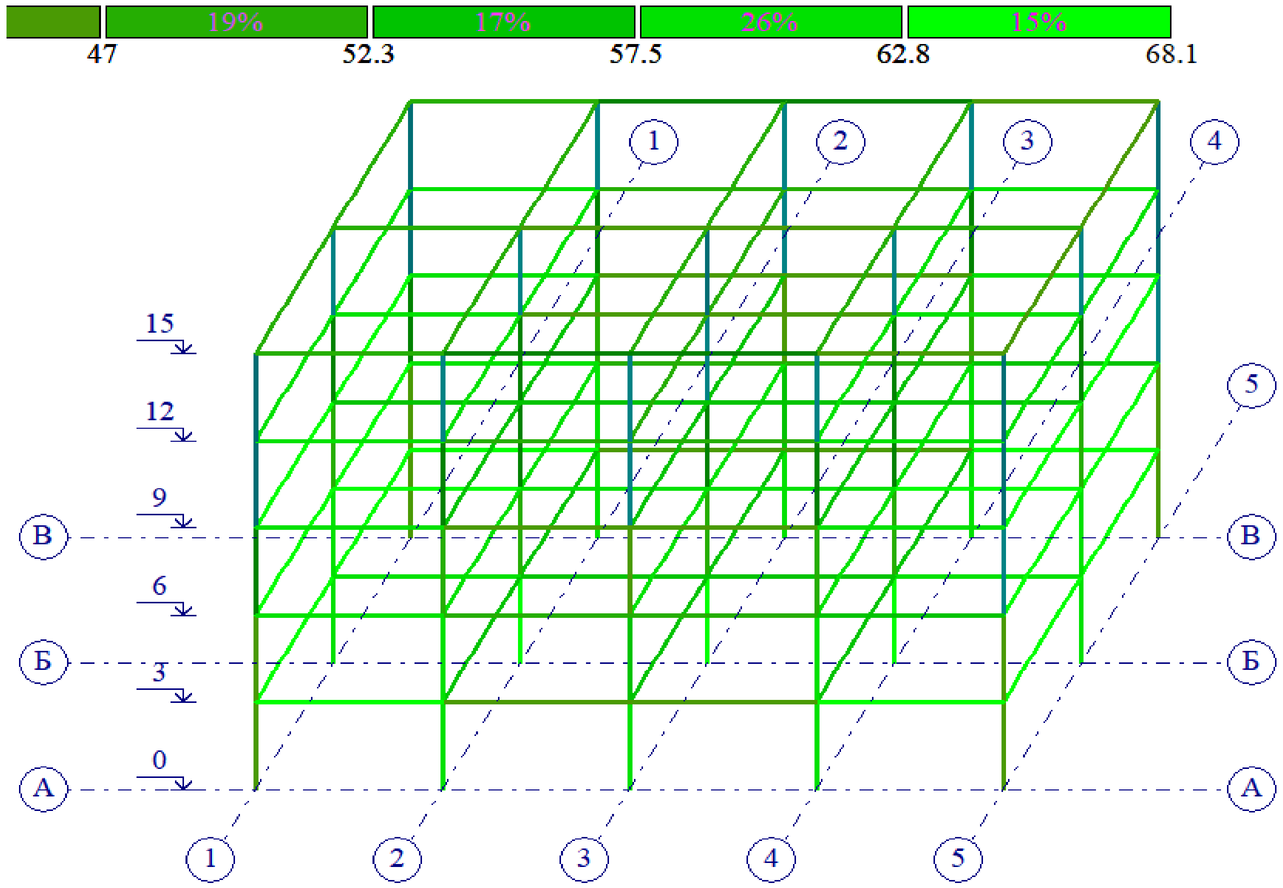

Figure 14 - General mosaic of verification results. [Authors' material]

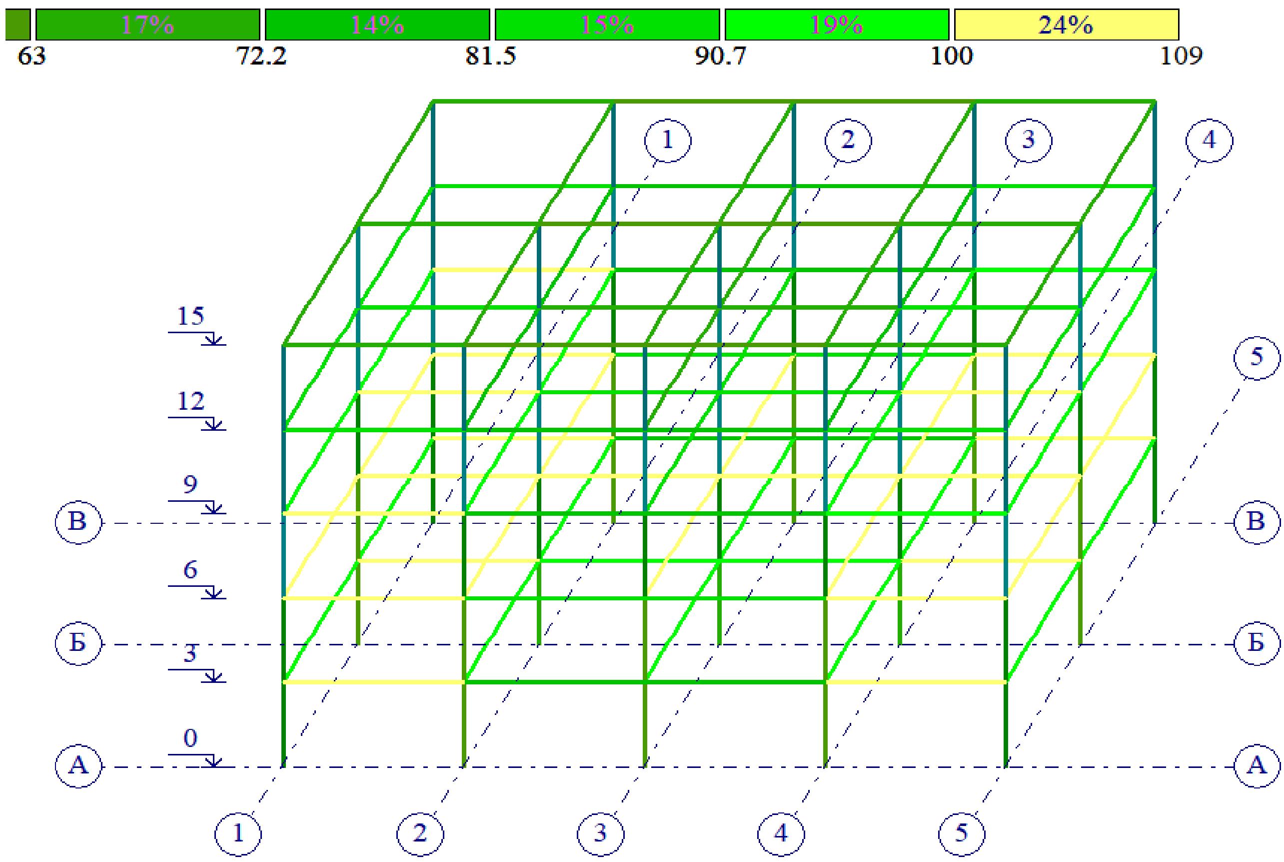

Figure 15 - General mosaic of verification results. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

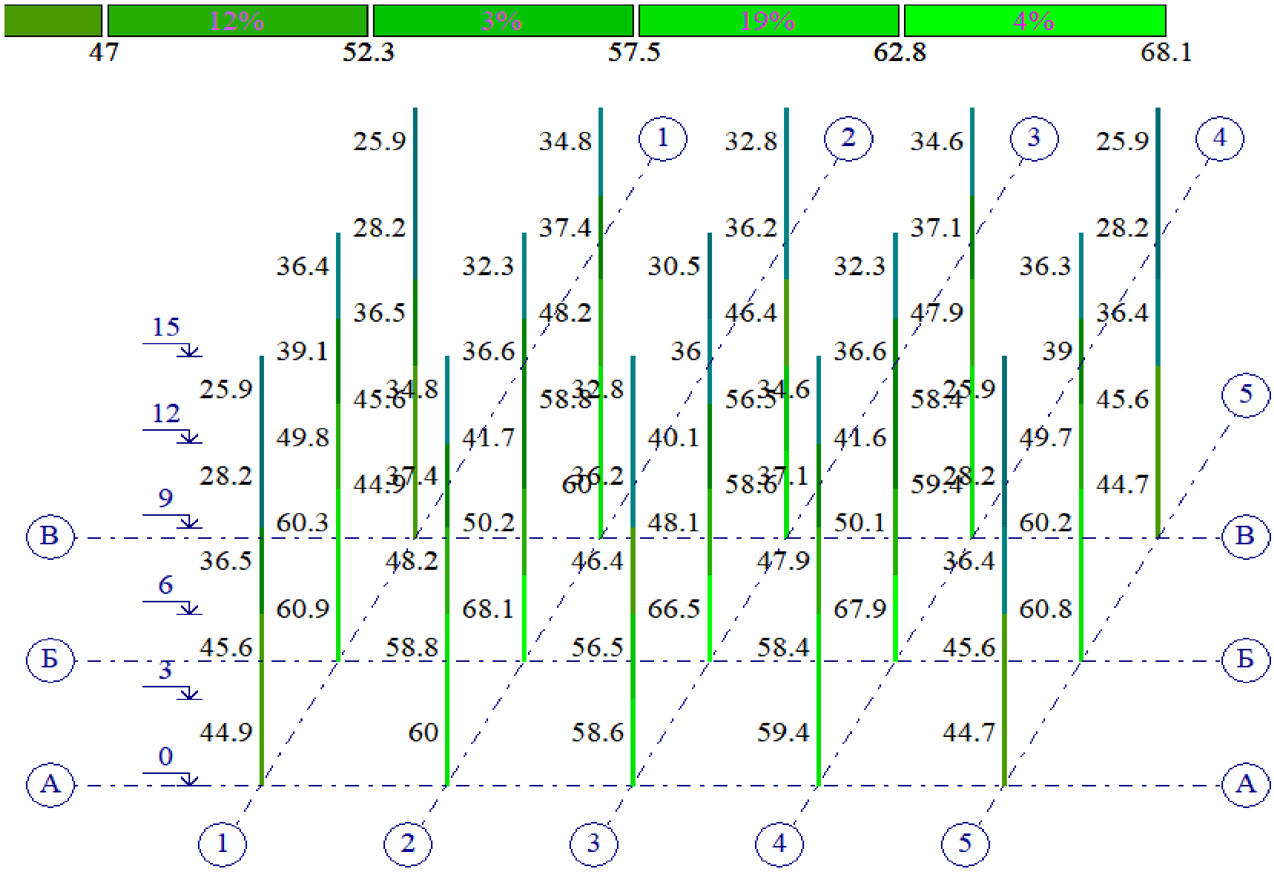

Figure 16 - Mosaic of verification results. [Authors' material]

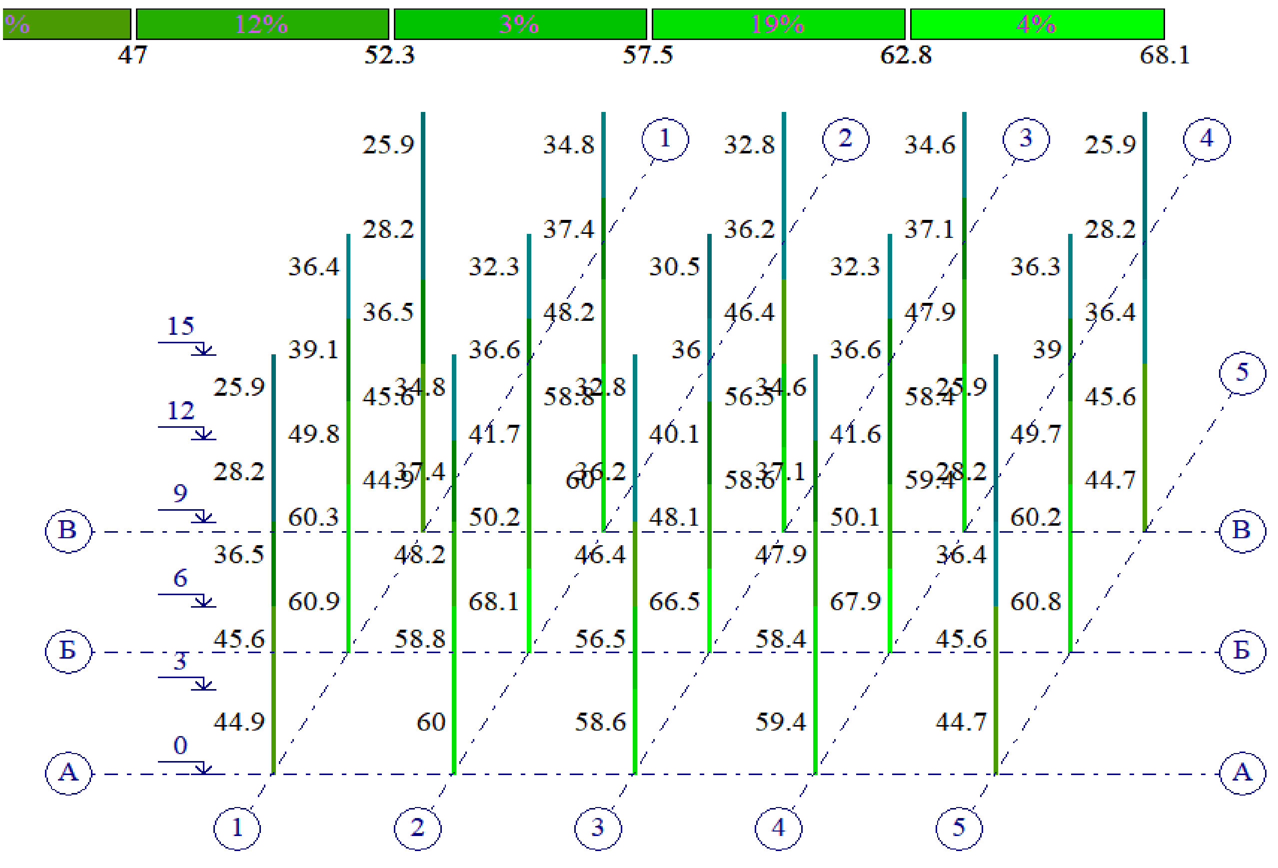

Figure 17 - Mosaic of verification results. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

There is no difference for the columns from the comparison of the results obtained for calculating the columns according to the two options, and it is impossible to give preference to any option.

This may be relevant when calculating beams, since an increased value of the yield strength of steel f_y can reduce the metal consumption of beams. The comparative results of the two calculations are shown in Table 5.

Table 5 - Comparative results of calculations of variants 1 and 2 for the SP of the Republic of Kazakhstan 2.03-30-2017*, in Figures 16, 17. [Authors' material]

№	Floor number	Naming of axes	Result of the SP of the Republic of Kazakhstan 2.03-30-2017* in %, Option 1	Result for the SP of the Republic of Kazakhstan 2.03-30-2017* in %, option 2	Difference of results
1	1	«1, Б»	60.9%	60.9%	0%.
2	2	«1, Б»	60.3%	60.3%	0%.
3	3	«1, Б»	49.8%	49.8%	0%.
4	4	«1, Б»	39.1%	39.1%	0%.
5	5	«1, Б»	36.4%	36.4%	0%.
6	1	«3, Б»	66.5%	66.5%	0%.
7	2	«3, Б»	48.1%	48.1%	0%.
8	3	«3, Б»	40.1%	40.1%	0%.
9	4	«3, Б»	36%	36%	0%.
10	5	«3, Б»	30.5%	30.5%	0%.

Building ULS (load-bearing capacity) checks calculated according to the standards of NTP RK 08-01.5-2013 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, and a strength reserve coefficient γ_{ov} = 1.25 shown in Fig.18.

The results of columns for ULS (load-bearing capacity) buildings designed according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 245 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) shown in Fig.19.

As can be seen from the comparative results, the difference in the bearing capacity of the columns is quite large.

Even if we take only the most loaded columns of the 1st and 2nd floors, the difference reaches its maximum on the 2nd floor and amounts to 30.769% for the outermost columns and 35.781% for the inner columns.%.

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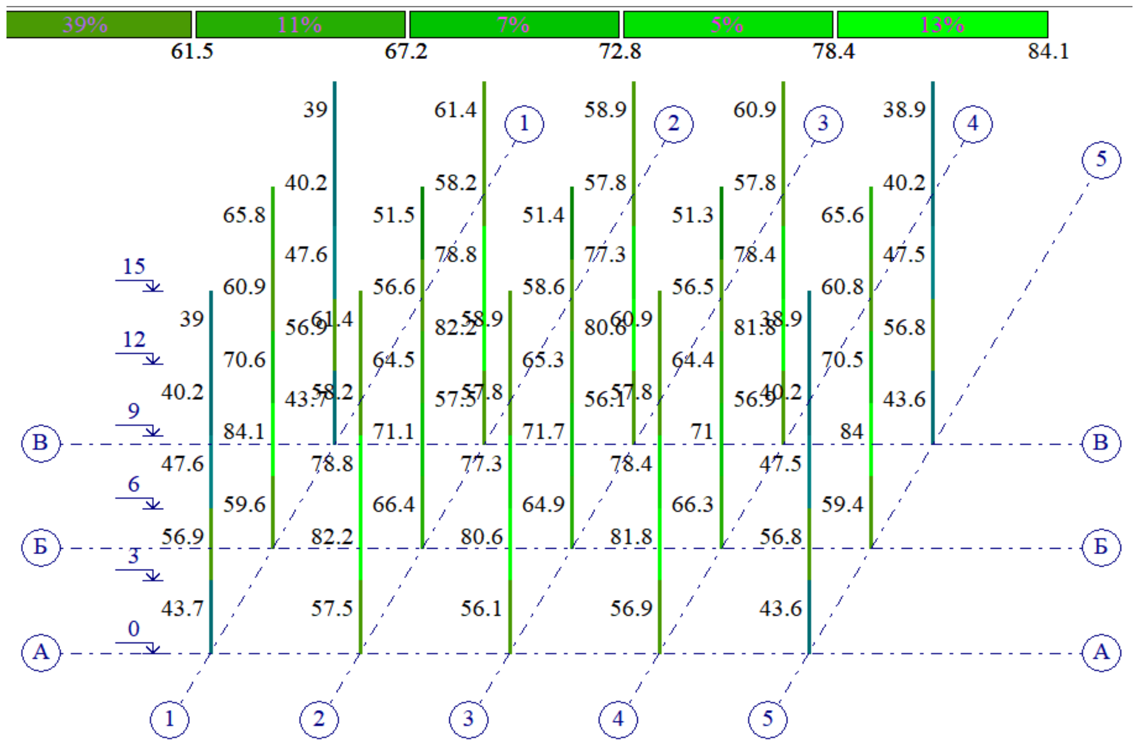

Figure 18 - Mosaic according to NTP RK 08-01.5-2013. [Authors' material]

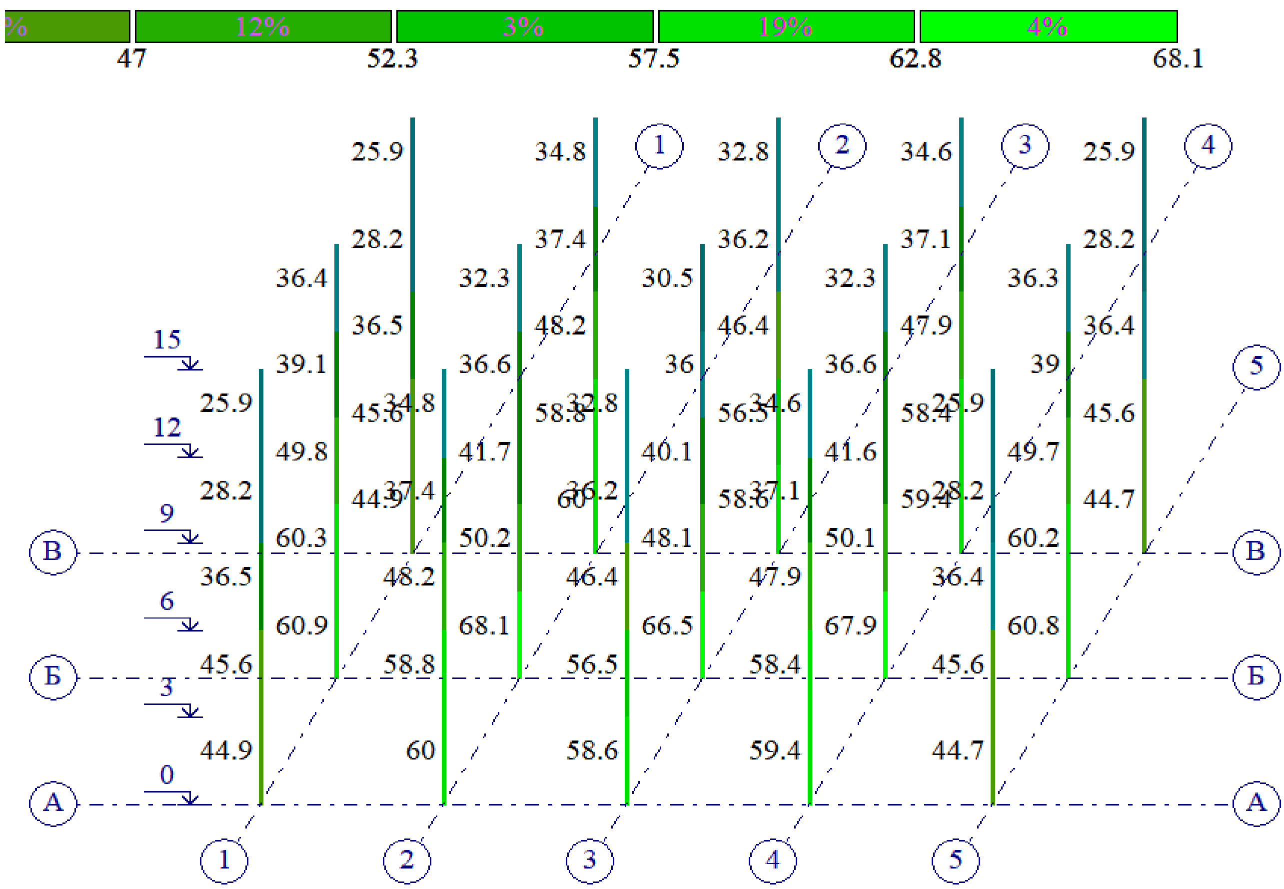

Figure 19 - Mosaic on the SP of the Republic of Kazakhstan 2.03-30-2017*. [Authors' material]

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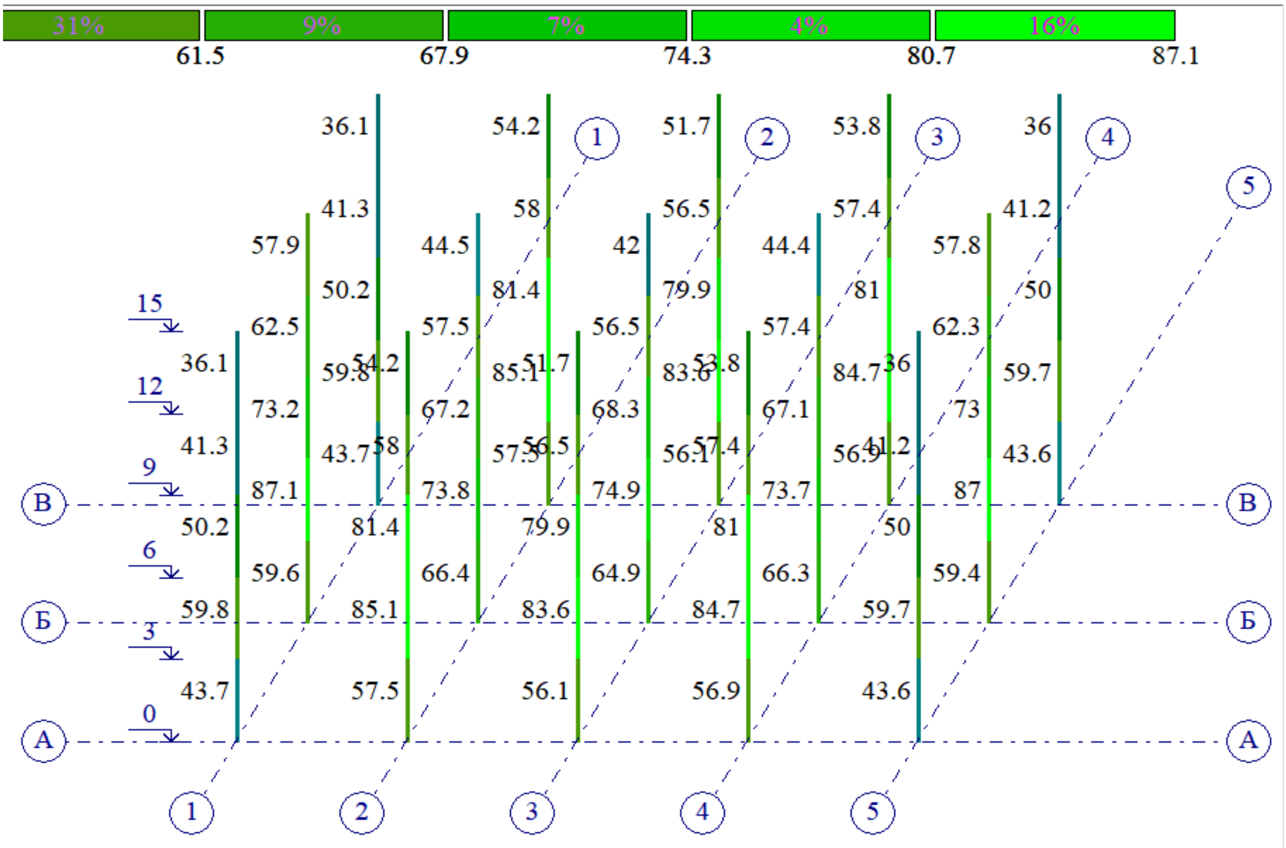

Figure 20 - Mosaic according to NTP RK 08-01.5-2013. [Authors' material]

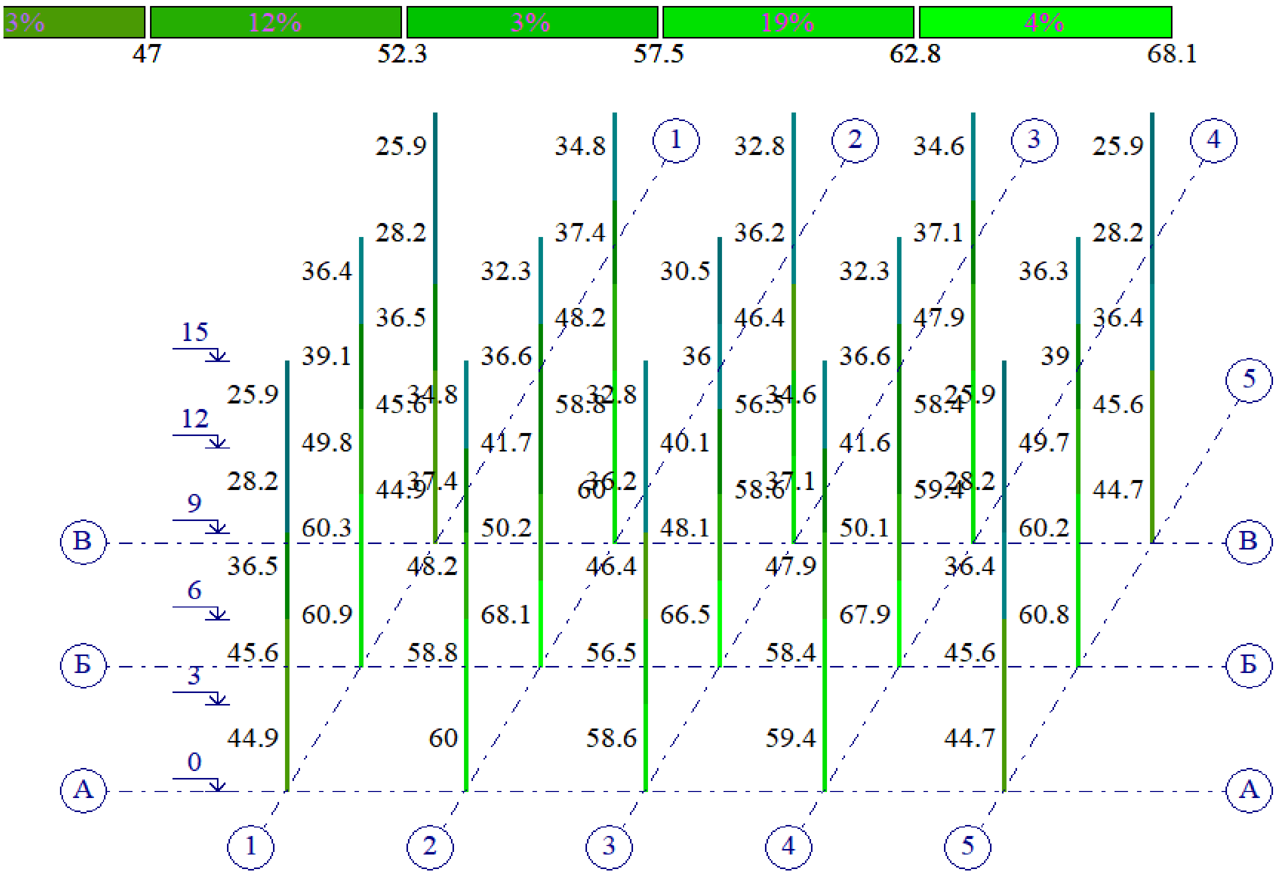

Figure 21 - Mosaic on the SP of the Republic of Kazakhstan 2.03-30-2017*. [Authors' material]

https://doi.org/10.51488/1680-080X/2026.2-04

The percentage of use of the assigned column sections according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the NTP RK 08-01.5-2013 with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, and a strength reserve coefficient γ_{ov} = 1.25 shown in Fig.20.

The results of the verification of the percentage of use of the assigned column sections according to the ULS (load-bearing capacity) of the building, calculated according to the standards of the SP of the Republic of Kazakhstan 2.03-30-2017* with a yield strength of steel for columns f_y = 345 MPa, with a yield strength of steel for beams f_y = 345 MPa, with safety coefficients γ_{MO}, γ_{M1}, γ_{M2} (1.0, 1.0, 1.25) shown in Fig.21.

The comparative results of the two calculations are shown in Table 6.

Table 6 - Comparative results of column calculations according to NTP RK 08-01.5-2013 and SP RK 2.03-30-2017*, in Figures 20, 21. [Authors' material]

№	Floor number	Naming of axes	Result according to NTP RK 08-01.5-2013, in %	Result according to SP RK 2.03-30-2017*, in %	Difference of results
1	1	«1, Б»	59.6%	60.9%	2.135%.
2	2	«1, Б»	87.1%	60.3%	30.769%.
3	3	«1, Б»	73.2%	49.8%	31.967%.
4	4	«1, Б»	62.5%	39.1%	37.44%.
5	5	«1, Б»	57.9%	36.4%	37.133%.
6	1	«3, Б»	64.9%	66.5%	2.406%.
7	2	«3, Б»	74.9%	48.1%	35.781%.
8	3	«3, Б»	68.3%	40.1%	41.288%.
9	4	«3, Б»	56.5%	36%	36.283%.
10	5	«3, Б»	42%	30.5%	13.333%.

3.6. Analyses of comparative calculations of the metal frame

When calculating the SP of the Republic of Kazakhstan 2.03-30-2017* the conditions of equal strength of all elements of the structural system involved in the perception of seismic loads were observed. This technique was inherited from previous SNiP documents as in the assignment of external loads to the structure (impact effects) This is also the case when calculating the cross-sections of the elements of this structure.

With this technique, seismic impacts were considered to be very short-term effects, as a result of which the mcr operating condition coefficient was additionally applied to the calculated resistance of the metal, increasing the calculated resistance of the metal. In the SP of the Republic of Kazakhstan 2.03-30-2017* it has been replaced by the coefficient γ_τ.

As a result, the resulting cross-sections of the calculated structures decreased. Savings in metal consumption were achieved, but at the same time the reliability of the structures decreased.

When designing buildings and structures in accordance with the provisions of the SP RK EN 1998-1 and its normative and technical manuals, the rules of the method of additive design are followed, which provides for the planning of damage zones of structural systems during seismic impacts [19].

Several methods are used to create plastic joints in beams (the strong column-weak beam concept) in the SP RK EN 1998-1 and regulatory and technical manuals.

1. Application of formula (4.29) according to SP RK EN 1998-1 or formula (4.1) according to NTP RK 08-01.5-2013 ($\Sigma MRc \geq 1.3\ \Sigma MRb$). When applying these formulas, the sum of the calculated values of the moments of resistance of the columns in the zones of nodal joints should exceed the sum of the calculated values of the moments of resistance of the beams in the zones of nodal joints by 1.3 times. This increases the rigidity of the columns in the areas of nodal joints and makes it possible to form plastic joints in the joints of the beams with the columns, i.e. in the beams, not in the columns.

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2. The use of class S235 steels (calm and semi-calm) in beam elements and class S325 steels in column elements with a strength reserve coefficient of 1.25. With this use of steels, it is assumed that during an earthquake, plastic deformation of dissipative zones will occur earlier than in other zones in which deformations remain in the elastic range (according to 6.2 (2)R SP RK EN 1998-1 and 2.2.2.1 NTP RK 08-01.5-2013).

3. Application of formulas (6.6) for SP RK EN 1998-1 and (4.6) for NTP RK 08-01.5-2013

$N_{Ed} = N_{Ed,G} + 1,1$ □□□$_{ov}$ □□□□□□$N_{Ed,E}$ (1)

$M_{Ed} = M_{Ed,G} + 1,1$ □□□$_{ov}$ □□□□□□$M_{Ed,E}$ (2)

$V_{Ed} = V_{Ed,G} + 1,1$ □□□$_{ov}$ □□□□□□$V_{Ed,E}$, (3)

it makes it possible to increase the effects of seismic impacts on the columns ($N_{Ed,E}$, $M_{Ed,E}$ и $V_{Ed,E}$) additionally by the product of the coefficients 1,1 □□□$_{ov}$ □□□, this contributes to an increase in the cross-section of the columns and a more favorable formation of plastic joints in the beams, rather than in the columns.

Also, the coefficient Ω ($\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$) is a "safety device" in case a large beam cross-section is not justified.

From the obtained comparative calculations of column cross-sections according to ULS (load-bearing capacity), calculated according to the standards of NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* using steel C245 for beams and C345 for columns with a strength reserve coefficient γ_{ov} = 1.25 (Table.4) it can be seen that the difference in the results in terms of the percentage of use of the ULS section (1PS) for columns of the 1st and 2nd floors ranges from 38.912% to 40.309%.

Results for the SP of the Republic of Kazakhstan 2.03-30-2017* lower than according to NTP RK 08-01.5-2013, which means that when calculating the SP RK 2.03-30-2017* sections smaller than according to NTP RK 08-01.5-2013 can be accepted.

This difference in the calculation results indicates that the reliability of the accepted cross-sections calculated according to the SP RK 2.03-30-2017* lower than calculated according to NTP RK 08-01.5-2013.

The results obtained require additional assessment for compliance with the provisions of paragraph 1.4(5) of the SP RK EN 1990:2002+A1:2005/2011, which states that alternative calculation methods can be used provided that a level of safety, serviceability and durability comparable to the requirements of the Eurocodes is ensured.

When used in the calculations of column cross-sections according to ULS (load-bearing capacity), calculated according to the standards of NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* using C345 steel for beams and columns with a strength reserve coefficient γ_{ov} = 1.0 (Table.5) it can be seen that the difference in the results in terms of the percentage of use of the ULS section (1PS) for columns of the 1st and 2nd floors ranges from 41.522% to 46.728%.

The general trend in the difference in the calculation results is observed in both tables.

When comparing the calculation results for NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* without applying the coefficient γ_τ = 1.3 (Table. 7 and Table.8) although the difference is smaller, it ranges from 28.3% to 35.781%.

This difference is in the calculations of column cross-sections according to ULS (load-bearing capacity), calculated according to the standards of NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* due to the fact that in the SP of the Republic of Kazakhstan 2.03-30-2017* there is no increase in column moments by 1.3 times, as in NTP RK 08-01.5-2013, and there is no increment of the effects of seismic impacts by the product of the coefficients 1,1 □□□$_{ov}$ □□□□

Although there is a difference in the spectra of calculated reactions of seismic impacts according to the standards of NTP RK 08-01.5-2013 and SP RK 2.03-30-2017* there is a very

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small difference in the results obtained when designing in compliance with the conditions of equal strength of all elements of the structural system involved in the perception of seismic loads (SP RK 2.03-30-2017*) and compliance The rules of the method of additive design (NTP RK 08-01.5-2013) are quite large [20].

4 CONCLUSIONS

1. Analysis of the results of comparative calculations of frame buildings with metal moment frames, made according to the requirements of the SP of the Republic of Kazakhstan 2.03-30-2017* and SP RK EN 1998-1:2004/2012, shows that with the overall goal of ensuring seismic resistance, these regulatory documents are based on various methodological approaches to determining calculated seismic impacts and assessing the stress-strain state of structures.

2. Under the same effects on metal frame buildings with momentary frames, calculation according to the norms of the SP of the Republic of Kazakhstan 2.03-30-2017* This leads to the adoption of smaller column cross-sections than in the calculation according to NTP RK 08-01.5-2013, which reduces the reliability of the building structure and contradicts clause 1.4(5) of the SP RK EN 1990:2002 A1:2005/2011 (the main founding document of the Eurocodes).

3. The results of the comparative analysis indicate the expediency of further updating the provisions of the SP of the Republic of Kazakhstan 2.03-30-2017* "Construction in seismic zones" in order to ensure a comparable level of reliability with the requirements of the NTP RK 08-01.5-2013, as well as harmonization with the principles and calculation approaches of the regulatory framework of the SP RK EN, identical to the Eurocodes.

4. Taking into account the identified differences, it seems reasonable to clarify the scope and individual calculation provisions of the SP of the Republic of Kazakhstan 2.03-30-2017*, aimed at improving the consistency of calculation results and ensuring the required level of structural safety.

REFERENCES

1. **SP RK 2.03-30-2017*.** (2025). Construction in seismic zones. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
2. **SP RK EN 1998-1:2004/2012.** (2016). Design of earthquake-resistant structures. Part 1. General rules, seismic actions and rules for buildings. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
3. **National Annexes to SP RK EN 1998-1:2004/2012.** (2017). Design of earthquake-resistant structures. Part 1. General rules, seismic actions and rules for buildings. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
4. **NTP RK 08-01.1-2012.** (2017). Design of earthquake-resistant buildings and structures. Part 1. General provisions. Seismic actions. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
5. **NTP RK 08-01.2-2021.** (2021). Design of earthquake-resistant buildings. Part 2. Design of civil buildings. General requirements. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
6. **NTP RK 08-01.5-2013.** (2017). Design of earthquake-resistant buildings. Part 5. Design of earthquake-resistant steel structures. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
7. **SNiP II-23-81*.** (1981). Steel structures. State Committee of the Council of Ministers of the USSR for Construction Affairs, Moscow.

https://doi.org/10.51488/1680-080X/2026.2-04

8. **SP RK 5.04-23-2002.** (2002). Steel structures. Design standards. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
9. **SP RK EN 1993-1-1:2005/2011.** (2016). Design of steel structures. Part 1-1. General rules and rules for buildings. Committee for Construction and Housing and Communal Services of the Ministry of Industry and Construction of the Republic of Kazakhstan, Astana.
10. **Fardis, M. N.** (2009). Seismic Design, Assessment and Retrofitting of Concrete Buildings. Springer. https://doi.org/10.1007/978-1-4020-9842-0
11. **European Convention for Constructional Steelwork** (ECCS). (2016). Design of Steel Structures for Buildings in Seismic Areas. ECCS Eurocode Design Manuals.
12. **Zhanabayeva, A., Moon, S.-W., Ocheme, J. I., Shokbarov, E., Khomyakov, V., Kim, J., & Satyana, A.** (2023). Comparison analysis of seismic design codes for shallow foundations according to the Kazakhstani and European approaches. Sustainability, 15(1), 615. https://doi.org/10.3390/su15010615
13. **Muhammad, S., Dichuan, Z., Moon, S.-W., Shokbarov, Y., Sarkulova, D., & Kim, J.** (2023). Macro-seismic assessment for residential buildings constructed during the Soviet Union era in Almaty, Kazakhstan. Buildings, 13(4), 1053. https://doi.org/10.3390/buildings13041053
14. **Begaliev, U., & Abdybaliev, M.** (2022). The approaches to define seismic hazard and demand for building design in the Kyrgyz Republic. In Proceedings of the 3rd European Conference on Earthquake Engineering and Seismology. Bucharest, Romania. https://3ecees.ro/3ecees-publications/
15. **Abdykalykov, A., Boronbaev, E., Begaliev, U., Holmatov, K., & Zhyrgalbaeva, N.** (2021). Building wall corner structures, its microclimate and seismic resistance. E3S Web of Conferences. https://doi.org/10.1051/e3sconf/202126304051
16. **Tuleyev, A. N., Abakanov, T. D., Lapin, V. A., Begaliev, U. T., Seitkazinov, O. D., & Dzholdasova, K. K.** (2024). Effect of the number of storeys on seismic resistance of high-rise monolithic buildings in Almaty. Bulletin of the Kazakh Leading Academy of Architecture and Civil Engineering, 4(94), 183–192. https://doi.org/10.51488/1680-080X/2024.4-14
17. **Tuleyev, A. N., Abakanov, T. D., Begaliev, U. T., & Lapin, V. A.** (2025). Seismic response of a 22-storey monolithic building based on instrumental acceleration records. Bulletin of the Kazakh Leading Academy of Architecture and Civil Engineering, 4(98), 241–255. https://doi.org/10.51488/1680-080X/2025.4-15
18. **Mels, R., & Shults, R.** (2026). Multicomponent seismic assessment of a 16-story building in Almaty using Eurocode 8. Bulletin of the Kazakh Leading Academy of Architecture and Civil Engineering, 1(99), 233–248. https://doi.org/10.51488/1680-080X/2026.1-16
19. **Surovtseva, N. V., Abakanov, T., & Nigmetov, G. M.** (2026). Experimental studies of the seismic vulnerability of a building model strengthened on soils. Bulletin of the Kazakh Leading Academy of Architecture and Civil Engineering, 1(99), 311–325. https://doi.org/10.51488/1680-080X/2026.1-21
20. **Shadkam, A. S., Bessimbayev, E. T., Begaliev, U. T., & Niyetbay, S. E.** (2024). The comparison of deformation calculation results of structures under seismic impact using Mohr–Coulomb and Hardening Soil Small (HSS) soil models. Bulletin of the Kazakh Leading Academy of Architecture and Civil Engineering, 4(94), 193–202. https://doi.org/10.51488/1680-080X/2024.4-16

https://doi.org/10.51488/1680-080X/2026.2-04

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CONFLICT OF INTEREST

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