
Строительные материалы и изделия
Construction Materials and Products

journal homepage: https://bstu-journals.ru

ISSN 2618-7183

DOI: 10.58224/2618-7183-2025-8-2-10

# Capabilities of existing frame buildings with shear diaphragms to resist seismic effects of destructive earthquakes

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**Abstract.** In recent times, numerous powerful earthquakes have struck across the globe, with intensities exceeding standard design values by 1 … 2 points, resulting in widespread destruction of buildings and infrastructure. These events underscore the urgent need to revise current regulatory frameworks, particularly by increasing the prescribed seismic design loads. Consequently, it becomes essential to reassess the seismic performance of existing buildings that were originally designed according to outdated codes. This article explores the critical issue of evaluating and enhancing the earthquake resilience of such structures in light of evolving seismic realities.This article presents the results of a seismic resistance assessment for a nine-story reinforced concrete frame building with stiffening diaphragms, subjected to seismic loads exceeding the original design values. To evaluate the seismic performance of the existing structure, a numerical analysis was carried out using a static nonlinear (pushover) method. As a failure criterion, the maximum seismic load corresponding to the complete loss of the building's load-bearing capacity was selected. The seismic resistance was assessed by considering the maximum values of seismic impact from two horizontal components, applied independently along each principal direction of the building. According to the adopted methodology, the structural model of the building frame, subjected to vertical loads, was incrementally loaded with the horizontal component of seismic action using displacement-controlled nonlinear static analysis. The horizontal load was gradually increased until the structure reached its maximum seismic capacity. The building under study was originally designed in accordance with the outdated seismic code SNiP RK 2.03-30-2006, which specified a seismic load corresponding to a site acceleration of 0.125g. However, under current seismic design standards—SP RK 2.03-30-2017*—the same site is classified for a seismic acceleration of 0.2g. Therefore, the existing structure is now expected to resist a seismic load that is 1.6 times greater than the load considered in its original design (0.2g vs. 0.125g). The study revealed that complete loss of the building's load-bearing capacity occurs under a special load combination when the seismic load is applied in the direction of the Y-axis. It was determined that structural failure takes place at the thirteenth loading stage, corresponding to a horizontal seismic acceleration of 0.163g. This value is lower than the required acceleration of 0.2g as specified by the current seismic code SP RK 2.03-30-2017*. This finding indicates that the building is incapable of withstanding the seismic demands outlined in the updated standards, highlighting its insufficient seismic resistance under the revised design requirements. Based on the results of the conducted

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Received January 29, 2025; Received in revised form March 1, 2025; Accepted April 30, 2025

research, it was proposed to strengthen the structural system of the building, which was originally designed and constructed in accordance with the outdated standards SNiP RK 2.03-30-2006, in order to enhance its seismic resistance and ensure compliance with current seismic safety requirements.
**Keywords:** shear diaphragm, buildings of the old construction, seismic resistance assessment, life safety, loading stage, seismic load, old and new standards

**Please cite this article as:** Nurseitov Sh., Yerimbetov B., Duissenbekov B., Chalabayev B., Kolesnikov A., Dossaliyev K., Kunanbayeva Ya., Aubakirova F. Capabilities of existing frame buildings with shear diaphragms to resist seismic effects of destructive earthquakes. Construction Materials and Products. 2025. 8 (2). 10. DOI: 10.58224/2618-7183-2025-8-2-10

## 1. INTRODUCTION

In this article, destructive earthquakes are earthquakes whose intensity on the earth's surface exceeds the standard value adopted during design.

A powerful example of the damage caused by natural disasters is the earthquake that occurred in Turkey in February 2023. Fig. 1 displays the ruin of a contemporary building as a result of this event [1].

**Fig. 1.** Destruction of the elite house Rönesans Rezidans during the earthquake in Turkey. February 2023, Hatay province.

The 2023 seismic event in Turkey exposed the insufficient bearing capacity in both contemporary and older constructions, which were unable to withstand tremors surpassing the region's normative thresholds. This led to extensive damage and, in many cases, total destruction (Fig. 1) [1].

The authors of reference [2, 3] attribute the widespread damage of both older and newer buildings to an underestimated seismic hazard in Turkey. They emphasize that the lack of seismic capacity in the buildings was between 2 and 4 points, resulting in some structures facing stress levels up to 4 to 8 times above their intended limits.

It is not uncommon for earthquakes to exceed the seismic intensities assumed in design standards, often resulting in considerable destruction and high casualty rates [4]. The Spitak earthquake, which occurred on December 7, 1988, is one such case. As noted in studies [5, 6], the intensity exceeded the normative level by 2 points, leading to seismic forces four times greater than the design parameters.

Based on the above, it can be concluded that there is a growing trend toward increasing the estimated seismic hazard on the seismic zoning maps of various countries located in seismically active regions. In response to this trend, the requirements of outdated regulatory documents on earthquake-resistant construction are being revised, with increased values of seismic loads now considered in the design of buildings.

This situation underscores the need for a comprehensive reassessment of the seismic resistance of existing buildings that were originally designed according to outdated standards. Such reassessment should account for the extreme levels of seismic loading and incorporate the nonlinear behavior of building materials to ensure accurate evaluation of structural performance under seismic conditions.

This study introduces a novel approach for evaluating the seismic performance of buildings constructed under obsolete design codes, emphasizing the impact of elevated seismic demands and the nonlinear response characteristics of structural materials.

In countries located in seismically active zones, construction codes and standards are constantly refined based on insights gained from analyzing the effects of significant earthquakes, including techniques for assessing and establishing standard seismic intensity levels [7, 8].

Insights gained from major earthquakes emphasize the need for continuous improvement in calculation methods for special load combinations, accounting for seismic forces, to enhance the earthquake resistance of buildings [9, 10] and structures [11-12]. Moreover, reliably estimating the intensity of probable high-impact earthquakes continues to be a key challenge [13, 14].

Certain elements of this assessment method for evaluating the load-bearing capacity of buildings and structures under elevated seismic impacts [15, 16], as outlined in regulatory guidelines guidelines SNiP RK 2.03-30-2006 and SP RK 2.03-30-2017*, were explored in our previous studies [17, 18], which focused on specific design solutions for buildings [19, 20].

In the Republic of Kazakhstan, up until 1998, the design and construction of buildings and structures in seismically active areas followed the guidelines set by regulatory documents such as SNiP II-A.12-62, SNiP II-A.12-69*, and SNiP II-7-81*.

Over the years, the regulatory framework for the design and construction of buildings and structures in seismic zones within the Republic of Kazakhstan has seen considerable evolution. Since 1998, several regulatory documents have guided this process, including SNiP RK B.1.2-4-98, SNiP RK 2.03-04-2001, SNiP RK 2.03-30-2006, and SP RK 2.03-30-2017*. In 2020, new regulations were implemented that align with the Eurocodes (SP RK EN 1990:2002+A1:2005/2011, SP RK EN 1998-1:2004/2012, SP RK EN 1998-1:2004/2012, NP to SP RK EN 1998-1:2004/2012, NTP RK 08-01.1-2012 (to JV RK EN 1998-1:2004/2012), NTP RK 08-01.2-2012). When evaluating earthquake intensity in compliance with regulatory standards (SNiP RK V.1.2-4-98, SP RK EN 1990:2002+A1:2005/2011, SP RK EN 1998-1:2004/2012, SP RK EN 1998-1:2004/2012, NP to SP RK EN 1998-1:2004/2012, NTP RK 08-01.1-2012 (to JV RK EN 1998-1:2004/2012), NTP RK 08-01.2-2012), general seismic zoning maps specific to Kazakhstan were employed. Presently, the relevant regulatory documents (SP RK 2.03-30-2017*, SP RK EN 1990:2002+A1:2005/2011, SP RK EN 1998-1:2004/2012, SP RK EN 1998-1:2004/2012, NP to SP RK EN 1998-1:2004/2012, NTP RK 08-01.1-2012 (to JV RK EN 1998-1:2004/2012) integrate these seismic zoning maps (GSZ RK), which represent seismic hazard levels in terms of points and accelerations.

In the Republic of Kazakhstan, seismic hazard evaluations based on the GSZ RK maps have been carried out utilizing a new methodological approach, following the principles outlined in Eurocode 8 "Design of Earthquake-Resistant Structures".

A distinctive feature of the GSZ RK map is that they provide quantitative parameters of ground vibrations, namely the values of peak accelerations required to carry out engineering calculations when designing buildings and structures. For example, when planning the construction of buildings and structures in Shymkent, Republic of Kazakhstan, which is situated on soils classified as second and third categories in terms of seismic properties, based on earlier regulatory documents] SNiP RK 2.03-30-2006 that utilized the GSZ map, the peak acceleration value of the foundation was determined, which characterizes the seismic hazard of a seven-point construction site, was 0.125g and

0.2g for soils of the second and third categories, respectively. According to subsequent regulatory documents, for example SP RK 2.03.30-2017*, based on GSZ maps, the value of the peak acceleration of the foundation, characterizing the seismic hazard of the construction site, is 0.2g and 0.253g, respectively, for soils of the second and third categories according to seismic properties. That is, in subsequent regulatory documents for the city of Shymkent, the values of peak accelerations in seismic zoning maps were increased with a corresponding increase in the design intensity of earthquakes for construction sites with different soil conditions.

A comparison of the requirements of earlier standards SNiP RK 2.03-30-2006 with those of later standards SP RK 2.03.30-2017* indicates that the seismic loads on frame buildings have increased. For instance, the seismic loads on a frame building designed according to standards SP RK 2.03.30-2017* for a construction site with type II soils and a seismicity of 7 points have risen by 1.6 times in comparison to the seismic calculations performed in accordance with the standards SNiP RK 2.03-30-2006.

This scenario indicates that existing buildings, which were designed based on earlier standards SNiP RK 2.03-30-2006, require a reassessment of their seismic resistance to address the heightened seismic loads outlined in standards SP RK 2.03.30-2017*. Consequently, it is essential to re-evaluate the seismic resistance of various building types from different construction periods, ensuring that the potential rise in seismic impact values aligns with the requirements of more recent standards, such as SP RK 2.03.30-2017*.

This situation is highlighted by the requirements of standards SP RK 2.03.30-2017*, which state that for buildings constructed under older standards, a new assessment of seismic safety must be conducted for structures located in areas where seismicity has increased following updates to the maps of seismic zoning.

This study explores the structural behavior of an old multi-story frame building to assess its load-bearing ability, accounting for prior loading conditions and the nonlinear properties of the materials, which become more apparent under loads that surpass design standards.

## 2. METHODS AND MATERIALS

To analyze the load-bearing capacity of the structural elements of the building, a numerical study was conducted on the performance of a multi-story a reinforced concrete frame structure with stiffening diaphragms, constructed in accordance with the requirements SNiP RK 2.03-30-2006, under a special combination of loads, accounting for seismic impact beyond the design limit. The study aims to evaluate how such systems perform when subjected to earthquake forces exceeding the intended design intensity.

As part of the numerical simulation, a nine-story reinforced concrete frame building with rigidity diaphragms is analyzed. It was developed and constructed according to the guidelines in SNiP RK 2.03-30-2006 for regions with a seismic intensity rating of 7, taking into account the combined action of special loads.

To assess the seismic resistance of the building being studied, the maximum values of seismic impact from two horizontal components were considered separately for each principal direction of the building. In other words, to evaluate the seismic resistance of a building designed according to the requirements of earlier standards, two separate scenarios of seismic loading were analyzed. The first scenario considers the case where the maximum seismic impact acts in the longitudinal direction of the building along the X-axis, with no seismic impact applied along the Y-axis. In the second scenario, the case of maximum seismic impact acting in the transverse direction of the building (along the Y-axis), with no seismic action along the X-axis, is considered. The stress-strain state of both individual structural elements and the building as a whole is analyzed under a special load combination that includes seismic effects. This analysis accounts for the elastic-plastic properties of the structural materials, utilizing nonlinear stress-strain relationships for concrete and reinforcement. The calculations are performed using the LIRA-FEM software package – a multifunctional computational system for the analysis, research, and design of structures for various applications, based on the finite element method.

To analyze the stress-strain state of the building while accounting for the nonlinear properties of structural materials (concrete and reinforcement), the structural model of the building frame incorporates physically nonlinear universal spatial beam finite elements (type 210) for columns, and physically nonlinear universal rectangular shell finite elements (type 241) for vertical stiffening diaphragms and floor slabs. Finite element type 210 is employed to perform analyses of various rod systems, incorporating the effects of material physical nonlinearity. Finite element type 241 is used to evaluate the stress–strain state of planar structural elements composed of physically nonlinear materials. The selected finite element types enable comprehensive analysis of structural stresses and deformations, capturing the nonlinear behavior of materials up to the point of complete element failure.

The connections between the frame elements in the model are assumed to be rigid, reflecting the behavior of joints in monolithic structures during deformation. Additionally, the connections of the vertical load-bearing elements to the foundation are also considered rigid at the level of the top edge of the foundation. In this study, the stress–strain behavior of structural elements was analyzed under special load combinations, including seismic actions exceeding the original design levels. The effects of soil–structure interaction were not considered, as the nonlinear analysis was conducted using the same boundary conditions that were applied during the initial design of the building. This approach, in our view, ensures a valid basis for comparing the obtained results. Prior to performing the static nonlinear analysis, the computational model of the structure—developed in the LIRA-FEM software—underwent verification. This included checking the boundary conditions, applied loads, geometric configuration, and the types of finite elements used to accurately capture the nonlinear response of the structure.

In line with our suggested approach, the structural frame model under gravitational loading experiences a steadily increasing horizontal seismic force, with horizontal displacement being tracked throughout the process. For the nonlinear analysis, the seismic load is represented by inertial forces obtained from a prior linear dynamic analysis, corresponding to the mode shape with the largest modal mass. These inertial forces are then transformed into a separate static load case to enable the subsequent static nonlinear (pushover) analysis.

The horizontal load is gradually increased until a defined limit is reached, which serves as the structural failure parameter. In this numerical study, the value of the seismic load corresponding to the complete loss of the building's load-bearing capacity is taken as the critical failure threshold.

In this study, the building was subjected to horizontal loading up to a level corresponding to a base acceleration of 0.2g, which, according to the requirements of SP RK 2.03-30-2017*, corresponds to seismic intensity of 7 points. Each loading stage corresponded to an incremental base acceleration of 0.0125g. Notably, the eighth loading stage, corresponding to a base acceleration of 0.125g, matched the seismic intensity of 7 points as defined by the earlier standard SNiP RK 2.03-30-2006.

Based on this nonlinear analysis, structural failure of the building occurred under the action of the special load combination at the thirteenth stage of horizontal loading. This stage corresponds to a base acceleration of 0.1625g, which is lower than the 0.2g base acceleration defined in SP RK 2.03-30-2017* as representative of a 7-point seismic event.

The reinforcement of the analysis of the building's load-bearing structures was carried out in accordance with the design, which was completed following the requirements of the standards in force at that time SNiP RK 2.03-30-2006.

Using the calculation results, the level of seismic load impact on the previously designed building was determined, and the calculated reinforcement for the supporting structures was compared to the actual reinforcement values specified in the project.

For the analysis, a spatial frame configuration was used to represent the building. The corresponding model, developed in the LIRA FEM platform, is shown in Fig. 2.

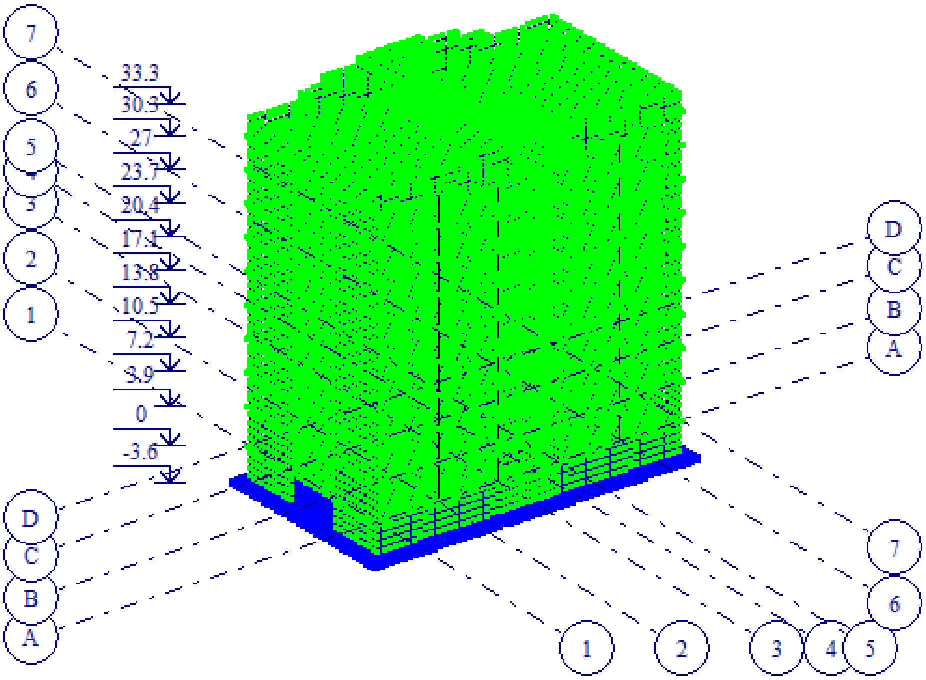



**Fig. 2.** The computational model of the research object.

Fig. 3 shows the general structural scheme of the load-bearing elements in the analyzed object.

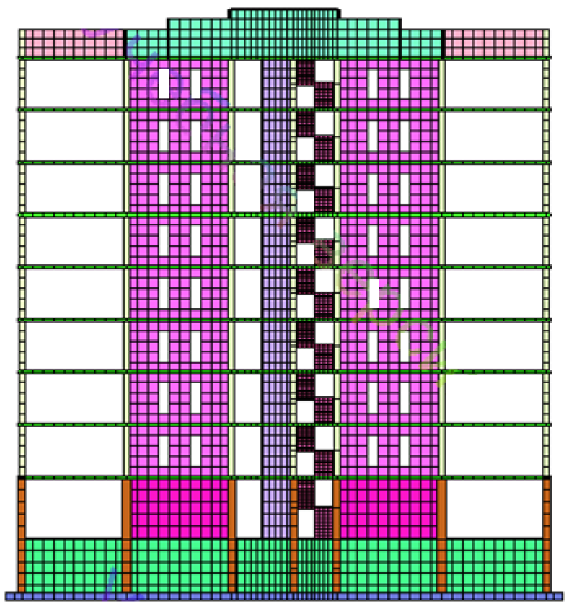


**Fig. 3.** Layout of the load-bearing structures of the research object.

The initial data of the frame-type building with stiffening (shear) diaphragms are adopted in accordance with the building design. The following characteristics of the load-bearing and non-load-bearing structures are adopted in the design.

The building is a 9-story frame-and-brace structure, with a reinforced concrete frame and stiffening diaphragms. Its outer dimensions in plan are 33.0 by 18.6 meters. The basement height is 3.6 meters, the first floor height is 3.9 meters, and the height of the upper floors is 3.3 meters each. The basement columns have a cross-section of 50x50 cm, while the first-floor columns share the same dimensions. Columns on the upper floors are 40x40 cm in cross-section. The basement walls are 40 cm thick and made of reinforced concrete. The stiffening diaphragms on the basement and first floors are also 40 cm thick, while those from the second to ninth floors are 30 cm thick. The stairwell walls are 24 cm thick, and the elevator shaft walls are 15 cm thick. The material used for the frame elements of the frame-braced scheme is reinforced concrete: concrete class C20/25, reinforcement class A-III (S400), wall material: foam concrete blocks with a bulk density of 1200 kg/m$^3$. Floor and roof slabs are monolithic, 200 mm thick.

For a more detailed analysis of the stress-strain state of the building frame elements, considering the nonlinear material properties, the adopted reinforcement details for the structural elements are presented below.

Columns are positioned along the full height of the building at the intersections of grid axes A, B, C, and D with axes 1, 2, 3, 5, 6, and 7. Based on the calculation results, the most heavily loaded columns are located at the intersections of axes D and 2, D and 3, D and 5, and D and 6. For these critical columns, the following reinforcement has been adopted:

- within the basement and first floors, the columns with cross-sectional dimensions of 50×50 cm are reinforced according to the following scheme: 4Ø28 + 4Ø25.
- within the second floor, these columns, with cross-sectional dimensions of 40×40 cm, are reinforced according to the scheme: 4Ø28 + 8Ø25.
- within the fourth and fifth floors, columns with cross-sectional dimensions of 40x40 cm are reinforced according to the scheme: 4ø28 + 4ø16;
- within the sixth to the ninth floors, the columns with cross-sectional dimensions of 40×40 cm are reinforced according to the scheme: 4Ø28 + 8Ø16.

The remaining columns, excluding the four critical rows located at the intersections of grid axes D and 2, D and 3, D and 5, and D and 6, are reinforced according to the following scheme:

- within the basement and first floors, the columns with cross-sectional dimensions of 50×50 cm are reinforced according to the scheme: 4Ø28 + 4Ø22.
- within the second to the fifth floors, the columns with cross-sectional dimensions of 40×40 cm are reinforced according to the scheme: 4Ø25 + 4Ø16.
- within the sixth to the ninth floors, the columns with cross-sectional dimensions of 40×40 cm are reinforced according to 4Ø28 + 8Ø16.

For the purpose of seismic vulnerability reassessment, a building originally designed under SNIP RK 2.03-30-2006 was analyzed using an amplified seismic load. The applied peak acceleration exceeded the reference value defined in SNiP RK 2.03-30-2006 by a factor of 1.6.

In the course of the evaluation of the resistance of buildings designed in accordance with the requirements of old standards, in the event of an earthquake exceeding the design value, the extent of damage to the building's load-bearing structures was revealed.

## 3. RESULTS AND DISCUSSION

As already noted, we have studied the operation of the above-described 9-story building, designed in accordance with the requirements SNiP RK 2.03-30-2006 for the action of a special combination of loads, using a numerical experiment. The horizontal component of a seismic impact with an intensity of 7 points, corresponding to the acceleration of the construction site equal to 0.125g, which corresponds to the provisions of the old standards, is taken as a special load.

It is known that in the seismic construction standards currently in force in the Republic of Kazakhstan for sites with an intensity of 7 points, their acceleration of the base is set at 0.2g. This circumstance increases the seismic load on the old building by 0.2g:0.125g=1.6 times. Consequently, the numerical experiment must ascertain whether the old building possesses a safety margin in the event of a more intense earthquake. In other words, it is crucial to assess the strength and stability of the old building when subjected to seismic loads that are 1.6 times greater than the design value.

To assess the strength of the building, the potential for plastic deformations in the building's reinforced concrete frame, under a specific combination of loads—including seismic loads as per SP RK 2.03.30-2017* – was considered, along with the behavior of the materials in accordance with the actual deformation diagram. This approach for assessing the building's load-bearing capacity is detailed in earlier studies [14, 15]. The behavior of concrete in the load-bearing elements was modeled using an exponential "stress-strain" relationship based on the actual strength and deformation characteristics of concrete class C20/25 for both compression and tension. Similarly, the performance of the longitudinal reinforcement in the reinforced concrete structures was modeled using a symmetric "stress-strain" relationship, reflecting the real strength and deformation characteristics of reinforcement class S400 under both compression and tension.

The frame was subjected to loading by gradually increasing the horizontal loads, which simulate the horizontal components of seismic forces. This increment in horizontal load allowed for an analysis of the frame's performance beyond its design load capacity and enabled the assessment of its ability to absorb excess seismic energy when the earthquake intensity surpasses the calculated threshold.

The horizontal component of the seismic load on the frame of the old building was applied in stages. In this case, the calculated value of the seismic load corresponded to the requirements SP RK 2.03.30-2017* for the construction site under consideration and was divided into sixteen loading stages. Based on the fixed values of the horizontal displacement of the building, a load-displacement graph was constructed for each step of changing the loads. The displacement of the building in the direction of the *X* and *Y* axes was recorded for characteristic nodes at the level of the building roof.

The stress-strain analysis of the frame elements, considering the combined effects of special load combinations and seismic action, revealed that an increase in horizontal loads caused partial failure in the most stressed areas of the load-bearing structure. Fig. 4 presents a graph showing how the building's displacement varies with load magnitude as seismic loading increases along the X-axis, up to the level defined in the current standard SP RK 2.03-30-2017*.

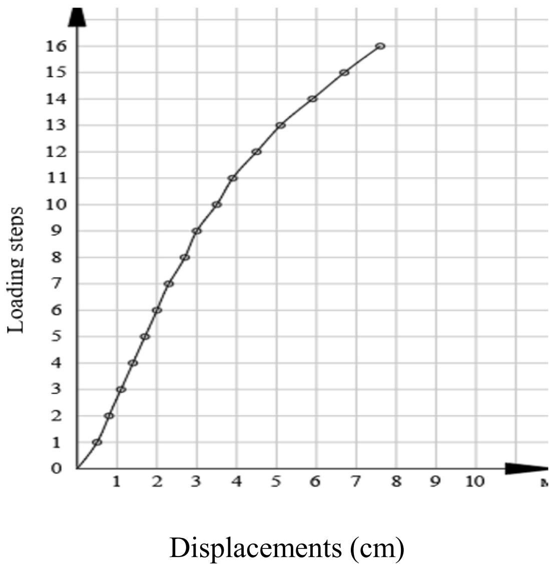



**Fig. 4.** Chart illustrating how the building's displacement varies with seismic load intensity applied along the X-axis.

Based on the simulation results, the highest displacement in the X direction due to seismic loading with a 7-point intensity was 2.8 cm. According to SNiP RK 2.03-30-2006, the permissible displacement for a 9-storey RC frame building is set at 9.1 cm.

This provision of the SNiP RK 2.03-30-2006 standard is intended to limit the horizontal deformation of buildings in order to prevent damage to non-structural elements such as infill walls, partitions, and glazing systems. In other words, the maximum allowable displacement of 9.1 cm, as specified in the standard, is aimed at maintaining the functional integrity of buildings designed under earlier codes and does not correspond to the failure of the primary load-bearing structures.

According to the findings of the current numerical study, seismic effects in the X direction, meeting the criteria of standard SP RK 2.03.30-2017*, cause damage to certain critical, high-stress regions of the supporting structure. In Figure 5, the damaged structural elements are shown in red.

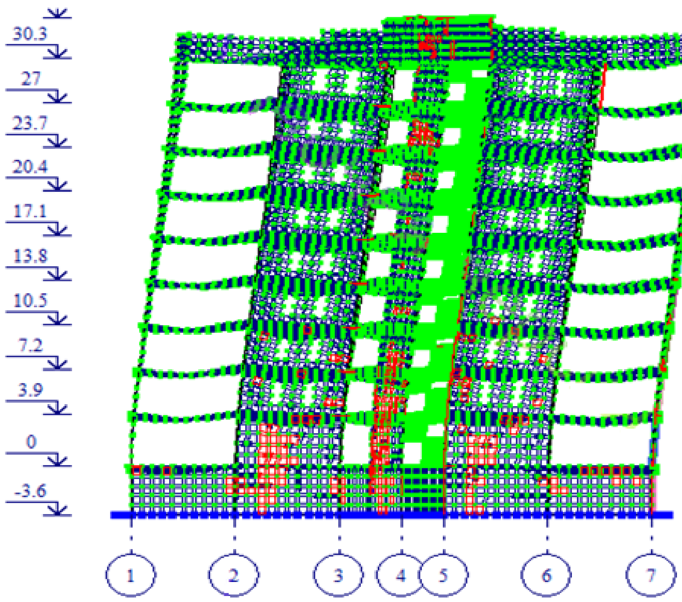



**Fig. 5.** Overall view of the stress-strain state of the building frame under a special combination of loads, including seismic impact along the X-axis, with a magnitude that complies with the requirements SP RK 2.03.30-2017*.

Overall, the results of the nonlinear analysis showed that the old building was able to withstand the special combination of loads, including the seismic load applied along the X-axis, with a magnitude that met the standard requirements SP RK 2.03.30-2017*.

Taking into account the nonlinear (elastoplastic) behavior of the structure's materials, the analysis of the building-planned for a location with a seismic rating of 7 in accordance with SNiP RK 2.03-30-2006-demonstrated a maximum displacement of 11 cm along the Y-axis under horizontal seismic loading of the same level.

In the case under consideration, the greatest displacement of the building, examined taking into account its elastic-plastic behavior of the structural materials, exceeded the maximum permissible value of the building displacement provided for by the standards. As the horizontal load acting along the Y-axis continued to increase, successive local failures occurred in the most stressed areas of the supporting structures, ultimately resulting in the building's collapse at the thirteenth loading stage under horizontal seismic load. In this case, the displacement value at the thirteenth loading stage in the direction of the Y-axis increased due to the failure of the most heavily stressed regions of the load-bearing structures and was more than 160 cm, which significantly exceeded the maximum permissible displacement value. The value of the seismic load at the loading stage corresponding to the building's collapse is proportional to the site acceleration of 0.163g, which is approximately 82% of the calculated acceleration value for the construction site, as stipulated by the requirements in SP RK 2.03.30-2017*. This circumstance indicates that the nine-story reinforced concrete frame building with shear diaphragms, designed according to the old standards SNiP RK 2.03-30-2006, cannot withstand an earthquake with a force corresponding to the requirements of subsequent standards SP RK 2.03.30-2017*. Consequently, in the case under consideration, in order to ensure seismic resistance of the building designed according to the old standards in the event of a possible earthquake with an intensity corresponding to the new current standards, it is essential to reinforce the load-bearing structures.

The table 1 presents the results of the stress-strain analysis of the building’s flat vertical load-bearing elements under special load combinations. The analysis accounts for seismic action along the X-axis, in accordance with the design parameters specified in SNIP RK 2.03-30-2006, and incorporates the nonlinear behavior of construction materials.

**Table 1.** The compressive stress levels observed in the cross-sections of building’s flat vertical load-bearing elements under special load combinations, including the impact of seismic action along the X-axis.

| Relevant vertical levels of the building, (m) | Compressive stress level in vertical elements parallel to the X-axis, (N/cm²) | Relevant vertical levels of the building, (m) | Compressive stress level in vertical elements parallel to the Y-axis, (N/cm²) |
|---|---|---|---|
| 3.9 ÷ 4.26 | 563 ÷ 597 | -1.8 | 365 |
| -3.6 ÷ 9.84 | 397 ÷ 505 | -3.6 ÷ 12.5 | 132÷265 |
| -3.6 ÷ 15.0 | 265 ÷ 393 | | |
| -3.6 ÷ 20.4 | 132 ÷ 265 | | |

The Table 2 presents the results of the stress-strain analysis of the building’s flat vertical load-bearing elements under special load combinations. The analysis accounts for seismic action along the Y-axis, in accordance with the design parameters specified in SNIP RK 2.03-30-2006, and incorporates the nonlinear behavior of construction materials.

**Table 2.** The compressive stress levels observed in the cross-sections of building’s flat vertical load-bearing elements under special load combinations, including the impact of seismic action along the Y-axis.

| Relevant vertical levels of the building, (m) | Compressive stress level in vertical elements parallel to the Y-axis, (N/cm²) | Relevant vertical levels of the building, (m) | Compressive stress level in vertical elements parallel to the X-axis, (N/cm²) |
|---|---|---|---|
| -1.8 ÷ 4.26 | 575 ÷ 624 | 4.5 | 453 |
| -3.6 ÷ 8.56 | 417 ÷ 546 | -3.6 ÷ 8.4 | 285 ÷ 359 |
| -3.6 ÷ 10.5 | 278 ÷ 412 | -3.6 ÷ 12.0 | 140÷277 |
| -3.6 ÷ 14.0 | 139 ÷ 278 | | |

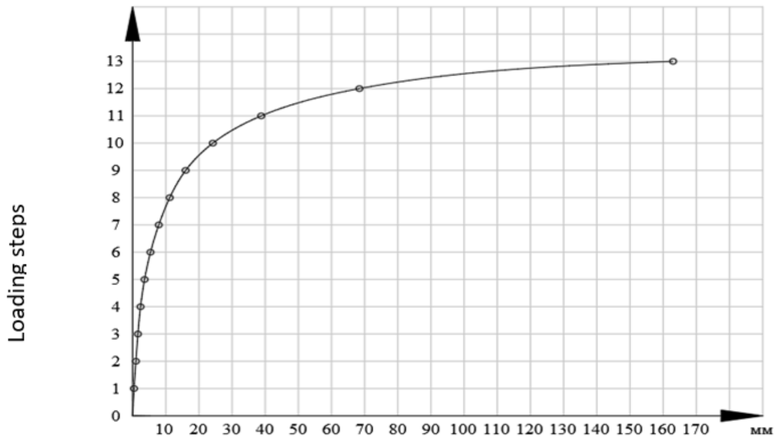
The results of the stress-strain analysis of the building frame elements indicate that, under a seismic load corresponding to the design level prescribed by SNiP RK 2.03-30-2006, the normalized compressive forces in the most critically stressed sections of the stiffening diaphragms oriented along the Y-axis were, on average, 30-35% higher than those observed in the corresponding sections of the diaphragms oriented along the X-axis.

A comparative analysis of the seismic load components along the X- and Y-axes demonstrated that the seismic load acting in the Y-direction is approximately 8% greater than that in the X-direction.

The increase in seismic load beyond the design level specified by the outdated standards resulted in the failure of the most heavily loaded stiffening diaphragms oriented along the Y-axis. This loss of diaphragm capacity was the primary factor leading to the overall loss of the building's load-bearing capacity under the action of the special load combination, which included seismic loading in the Y-direction. The stiffening diaphragms oriented along the X-axis, owing to their relatively low stress-strain state at the point when the seismic load exceeded the design level specified in the outdated standards, demonstrated sufficient structural capacity to resist seismic loading up to the design level prescribed by the updated code requirements.

Based on the analysis of the results from the numerical experiment carried out on a nine-story reinforced concrete frame structure with stiffening diaphragms, designed according to outdated standards, it can be concluded that assessing the seismic resistance of buildings built under these old standards requires an evaluation of their performance under increased seismic loads that align with the requirements of more recent standards. This assessment should consider the elastic-plastic behavior of the structural materials. Decisions regarding their seismic resistance should be based on such evaluations.

The findings of a numerical study regarding the stress-strain behavior of frame elements under the combined action of a special combination of loads, including seismic impact along the Y-axis and considering the elastic-plastic behavior of the structural materials, are presented below. The a graph titled "horizontal load – horizontal displacement," which demonstrates the increase in seismic load to a level that complies with the standards SP RK 2.03.30-2017*, is created considering the elastic-plastic behavior of the structural materials and is displayed in Fig. 6.




Displacements (mm)

**Fig. 6.** Graph representing the dependence of building displacement on the seismic force applied in the Y direction.

The special combination of loads, factoring in the seismic impact along the Y-axis, resulted in the collapse of the building frame at the thirteenth loading stage with a horizontal load. Fig. 7 depicts the stress-strain condition of the main load-bearing elements of the building (columns, shear diaphragms) before the structure fails.

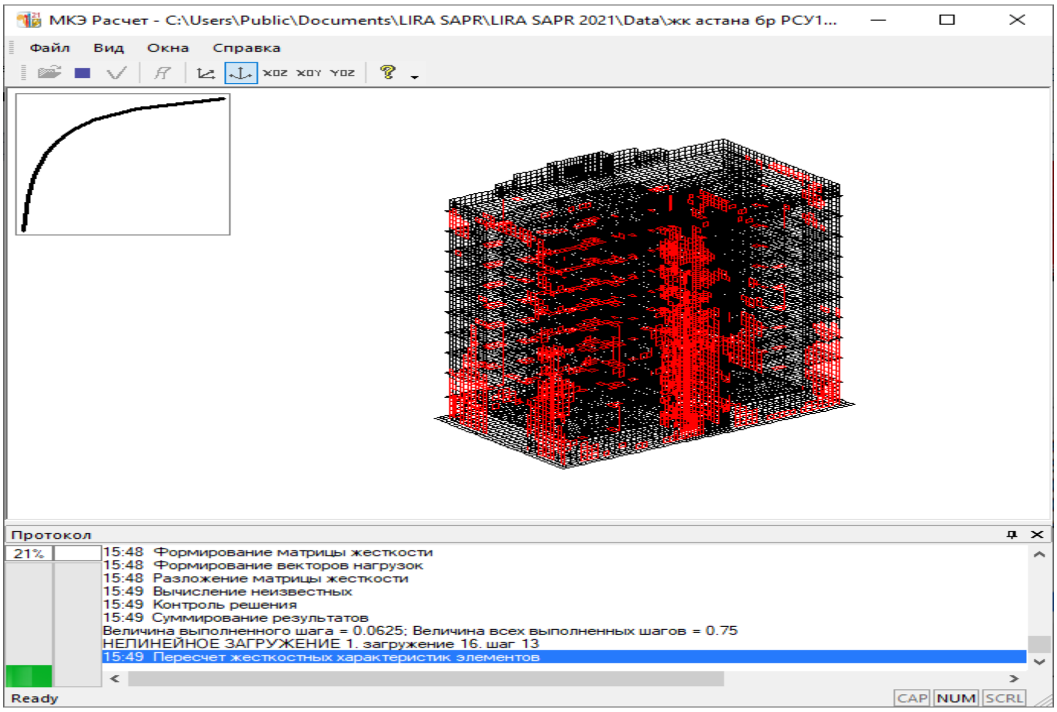



**Fig. 7.** The condition of stresses and strains in the structure of the research object just before failure.

The damaged elements of the building's load-bearing structures are marked in red.

In the columns and diaphragms of reinforced concrete frame buildings designed according to outdated standards, an increase in the seismic load magnitude acting along the Y-axis beyond the design value leads to a progressive failure of the most heavily loaded areas of the load-bearing structures. Simultaneously, the results of a numerical study on the building's performance indicated a building constructed based on outdated standards may collapse when the seismic load surpasses the design value, well before it reaches the seismic load specified by the new applicable standards.

When the horizontal load exceeds the design level corresponding to the seismic action defined by SNiP RK 2.03-30-2006, a progressive failure of the stiffening diaphragms oriented along the Y-axis is observed. The initial failure occurred in sections at the first-floor level, followed by partial failure at the basement level, and subsequently in the upper floors. The observed sequence of structural failure began with the vertical stiffening diaphragms, followed by the failure of the columns.

This finding indicates that the building under study does not possess sufficient capacity to withstand seismic loads as specified by the current standard SP RK 2.03-30-2017*.

Based on the above findings, the building in question—designed and constructed in accordance with the outdated standard SNiP RK 2.03-30-2006-requires structural strengthening to enhance its seismic resistance in accordance with current code requirements.

In cases where buildings designed and constructed under outdated seismic standards exhibit insufficient seismic resistance, it becomes necessary to strengthen the most highly stressed sections of the vertical load-bearing elements. The goal is to enhance the structural capacity to meet the seismic demand specified by current regulatory requirements. The selected strengthening techniques must be supported by detailed calculations and must comply with modern code provisions, particularly with respect to limiting the normalized compressive force ratio ($N_{Ed}/f_{cd}{\cdot}l_w{\cdot}b_w$) in the strengthened sections.

Thus, based on a numerical experiment in studying the operation of a 9-story frame designed for loads of a special combination according to SP RK 2.03.30-2017*, the necessity for a mandatory evaluation of the strength of the frame of an old building for a possible increase in the seismic load that may occur during the operation of the building has been established. If the load-bearing capacity of an old building is insufficient to withstand a possible increase in seismic load, measures should be taken to strengthen the load-bearing structures of the old building derived from an analysis of the building's stress-strain state.

The analysis of the results from a numerical study on a building with stiffening diaphragms, subjected to a special combination of loads while considering seismic impacts aligned with outdated standards, indicated that during potential earthquakes with seismic intensities that meet the criteria of newer standards, the building's supporting structures could sustain significant damage, potentially leading to complete destruction. This study of a multi-story frame building with stiffening diaphragms revealed that to ensure the seismic resistance of an old building as per the standards in SP RK 2.03.30-2017*, reinforcing the structure is essential.

In order to determine the reserve strength of buildings built in accordance with outdated regulations—should the site experience earthquake intensities exceeding those specified in modern codes—the following sequence of calculations is suggested.

- Verifying the characteristics of load-bearing components used in the initial design of an existing structure requires reanalyzing the same building with the help of structural modeling software. For example, a structural analysis can be carried out using tools like LIRA FEM. During this process, the calculated reinforcement of the structural elements should be evaluated against that of the original design. This step is also important for identifying the level of seismic load defined by previous standards.
- The analysis should account for seismic loads applied horizontally to the building's mass in two perpendicular directions, corresponding to the X and Y axes.
- Develop a computational model of the existing building in LIRA-SAPR software, incorporating the nonlinear performance of structural elements for analysis.
- For the building in question, designed and built in accordance with the outdated standards, determine the level of seismic impact on the building in accordance with the requirements of the new current standards.
- Conduct a calculation of the structure, taking into account the physical nonlinearity of the materials used in the supporting structures designed under outdated standards. This should consider a specific combination of loads, including seismic forces that align with the criteria of current earthquake-resistant construction standards, to assess the stress-strain condition of the supporting structures.
- Drawing on the analysis of the stress-strain condition of the supporting structures in the old building under a special combination of loads, including the increased seismic impact, determine whether the bearing capacity reserve of the old building is adequate or insufficient.
- Then carry out the calculation following the previously mentioned sequence to determine the bearing capacity reserve of the old building with reinforced supporting structures.

## 5. CONCLUSIONS

1. This work presents a method to detect the lack of seismic resilience in buildings designed following older codes, subjected to heightened seismic forces, by taking into account the nonlinear response of construction materials.
2. If seismic resistance deficiencies are found in buildings designed and constructed according to older codes, the most critically stressed areas of the vertical load-bearing structures must be reinforced to improve seismic capacity in line with the requirements of updated regulations. The reinforcement approaches selected should be validated through calculations that comply with the new standards' constraints on the normalized compressive stress ($N_{Ed} / f_{cd} \cdot l_w \cdot b_w$) in the strengthened sections.

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