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Experimental Investigations and Probabilistic Risk Assessment of Failure in Masonry Buildings with Load-Bearing Walls

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Abstract

The aim of this study is to determine the reliability level (the probability of failure-free operation) of a masonry building with load-bearing walls based on the conducted experimental investigations. The objective of the study is to compare the obtained reliability and failure risk values with the corresponding values calculated using the results of the structural certification. In 2017–2018, and subsequently in 2023–2024, a comprehensive structural certification of the multi-apartment residential building stock was carried out for the first time in the city of Almaty. A total of 1609 multi-story masonry buildings with heights of two to four stories were identified. Based on the certification results, quantitative estimates of the prior and posterior probabilities of failure and reliability for masonry buildings were obtained for the first time. The recurrence of earthquakes was taken into account. The novelty of the study lies in the experimental investigation of a three-story masonry building of series 308. The dynamic excitation was generated by an inertial vibration machine installed on the floor slab. As the inertial load increased, the resonant vibration period changed by a factor of three. This indicates that the building underwent significantly nonlinear deformation. The structure sustained substantial damage. Using statistical simulation methods based on the experimental data, the prior probabilities of failure for masonry buildings were calculated. In this case, the seismic action was modeled as a nonstationary random process with the deterministic envelope proposed by F. F. Aptikaev. Probabilistic estimates of the reliability of masonry buildings were obtained from the certification results both with and without taking into account the recurrence of earthquakes. The obtained estimates of reliability and failure probability can be used to develop practical recommendations aimed at reducing risk and expected losses in the event of possible earthquakes. It is recommended that masonry buildings with load-bearing brick walls either be structurally strengthened or be demolished.

Keywords: dynamics; certification; risk; brick building; losses; reliability; repeatability

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Academic Editor: Rita Bento

Received: 26 May 2026
Revised: 9 July 2026
Accepted: 15 July 2026
Published: 17 July 2026

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1. Introduction

1.1. Seismicity of the Territory and Literature Review

A significant part of the territory of Kazakhstan is located in seismically active areas. In the late 19th and early 20th centuries, the territory of Kazakhstan was twice subjected to some of the strongest earthquakes on record, with a magnitude of more than 8.

In 1996, a group of leading experts from the USA, Germany, Japan, Turkey and Kazakhstan, who participated in the international meeting "Seismic risk in the Republics of Central Asia" (Almaty, October 1996), having studied the state of existing buildings in Almaty, noted that in the event of a strong earthquake in Almaty, up to 75,000 people could die and more than 300,000 people would be injured.

This group of experts noted the high probability that a devastating earthquake will occur in the Central Asian countries over the next 20–25 years.

There are very serious reasons for such a forecast. Over the past 20 years, the elements have repeatedly reminded of themselves with strong earthquakes. Since 1970, five strong earthquakes have occurred in southern Kazakhstan alone, causing significant material damage to the republic's economy:

- Zhambyl earthquake—1970, M = 6.7;
- Zhalanash-Tyup earthquake—1978, M = 7;
- Baisorun earthquake—1990, M = 6.3;
- Suusamyr earthquake—1992, M = 7.3;
- Lugovaya station earthquake—2003, M = 5.3.

The most earthquake-prone regions of Kazakhstan include the territory where Almaty, the largest economic and cultural center of Kazakhstan, is located.

In 2024, two earthquakes occurred in Almaty, on 23 January and 4 March. Interestingly, the spectral characteristics of these earthquakes were very different. The earthquake of 23 January 2024 was located in China at a distance of 260 km south-east of Almaty, according to Kulbaev at al. [1]. The peculiarity of this earthquake was its low-frequency nature and significant duration of up to 50 s.

The earthquake of 4 March 2024 was registered by the stations of the engineering and seismometric service and the service of strong movements of the city of Almaty. Spectral analysis of the accelerograms showed that the prevailing period of ground oscillation was 0.2–0.3 s. The earthquake was centered in the Kemin tectonic faults, which are responsible for the most severe earthquakes in the Almaty city area. The earthquake intensity corresponded to a 6-point magnitude. Such earthquakes are most dangerous in terms of frequency characteristics for masonry buildings with similar oscillation periods.

It should be noted that two types of seismic stations are located within the territory of the city of Almaty. The first network comprises the stations of the Institute of Seismology, which record ground motion. This network is equipped with digital instrumental measurement systems of Swiss and Japanese manufacture. The total number of these stations exceeds 50.

The second network comprises stations installed directly on buildings. The instruments are generally mounted on the roof and in the basement part of the building. Both digital and analog instruments are used. In the city of Almaty, such stations have been installed on 11 buildings.

Brick structures have historically been one of the oldest building materials widely used in the construction of buildings for various purposes. Masonry buildings are among the most widely used structural systems for construction in seismic regions and have been employed there for a very long time. They also have the most extensive documented history of structural performance during strong and catastrophic earthquakes.

This is evidenced by the consequences of major earthquakes in Tokyo (1923), Chile (1923), Ashgabat (1948), Tashkent (1966), Bucharest (1977), Armenia (1988), Neftegorsk (1995), Kobe (1995), Izmit (1999), Turkey (2024), and other seismic regions.

During earthquakes, they generally sustain the most severe damage. Bricks of various types are produced in large quantities to date and are very widely used in housing construction according to Polyakov and Safargaliev [2] and Safargaliev [3].

Brickwork has many advantages. It is an affordable raw material, with high compressive strength and excellent heat retention properties. Therefore, a significant portion of the global building stock consists of masonry structures. Masonry buildings are preferred because of their low construction cost, flexible layout, wide availability, and aesthetic design, as well as good thermal and acoustic insulation.

One of its significant drawbacks is vulnerability to seismic impacts. During an earthquake, brick masonry walls are subjected to a wide variety of loads, the most unfavorable of which are dynamic bending, shear, and tension. Unreinforced masonry performs well only under the action of vertical compressive forces with a small eccentricity. In manually laid brick masonry, it is often difficult to achieve the required strength in terms of the normal bond between the brick and the mortar. A brick masonry wall performs excellently in compression and considerably worse in tension, which develops under alternating seismic loading.

Residential buildings with load-bearing brick walls with a height of two to four floors in Almaty were built over a long period, but their construction was carried out most intensively from 1930 to 1968.

Currently, the construction of residential buildings with load-bearing brick walls with a height of more than two floors is prohibited in Almaty.

Masonry buildings were designed in compliance with the standards of different years and, for this reason, their ability to resist high-intensity seismic impacts is different.

The application of bricks in construction, including in seismic-resistant structures, necessitates extensive experimental investigations and the use of advanced computational analysis methods. There is a considerable body of scientific literature on this topic.

In the article of Ahmad et al. [4], brick masonry structures were studied by testing models on a vibration stand. A large number of experimental results were obtained regarding methods of reinforcing such structures. It was noted that accelerations of up to 0.5 g were created on the vibration stand, at which shear destruction of brick walls took place. Based on the experimental data, several ways of reinforcing brick walls using propylene tapes and vertical wire meshes are proposed.

The objective of the study by Kaya et al. [5] was to assess the seismic performance of an unreinforced hollow brick masonry (URM) building at a 1:2 scale, taking into account various damage states. Tests were conducted on a vibration table under seven loading conditions. To achieve different damage states, dynamic loads were applied incrementally. The natural frequencies of the undamaged building were identified for the first three modes of vibration. The damage patterns of the building were also determined. Numerical analysis yielded results that were approximately consistent with the experimental findings.

The analysis of the effects of strong earthquakes on buildings with brick walls is discussed in the article of Ahmad et al. [6]. It is noted that during strong earthquakes, shear cracks occur around the openings of the walls, and the roof detaches from the wall structures. Experimental studies were conducted. The tests were thoroughly analyzed to interpret and refine damage patterns, as well as local and global structural behavior. The results show that a closed brick structure demonstrates 45% greater seismic strength than a semi-enclosed stone structure.

A large number of buildings with load-bearing brick walls have been constructed in China. Wang et al. [7] reported the results of experimental studies on six brick wall specimens. The study addressed key issues regarding the influence of vertical loading on the load-bearing capacity of brick walls with varying infill depths. Vertical loads were applied through the weight of the upper prefabricated floor slab. The research determined the effective design height of brick walls for different boundary conditions and proposed refined formulas for calculating their load-bearing capacity. Out-of-plane deformation patterns of the walls were also examined. These findings are particularly useful for the design and construction of low-rise buildings with load-bearing brick walls.

Under seismic loading, self-supporting brick walls operate under diagonal compression. These issues are discussed in the study by Reynau et al. [8] for four types of brick masonry. Distortions were measured using linear variable differential transformers (LVDTs) installed perpendicular to the masonry surface. A particularly interesting aspect of the study was the investigation of crack pattern development using the digital image correlation (DIC) method. This approach allows tracking the transition from linear-elastic behavior to significantly nonlinear response up to the point of failure. The main outcomes provide valuable recommendations on the development of cracking in self-supporting brick walls, which is highly relevant for the seismic assessment of historical buildings.

In the study by Galvez et al. [9], the discrete element method was used to investigate the seismic collapse mechanism of brick masonry walls as a function of boundary conditions, wall length, diaphragm stiffness, and the presence of openings in the wall. The study examined the process from the formation of initial cracks to the complete collapse of the building structures. In the numerical modeling, a Rayleigh damping model was employed, accounting for two important factors: mass-proportional damping and stiffness-proportional damping. The seismic loading was of impulsive type, with a predominant period of 0.3 s (average period), which is typical of frequently occurring earthquakes. A realistic representation of the collapse behavior of brick walls was obtained. For validation, previously conducted experimental studies and analyses of the consequences of strong earthquakes were used. It was found that modeling boundary conditions is particularly challenging when comparing numerical analysis results with observed seismic performance of brick walls.

Chaimae et al. [10] proposed an alternative numerical modeling approach for brick masonry walls. The ABAQUS software package, implementing a variant of the finite element method, was used. Three types of brick masonry were analyzed: the conventional layout and two newly developed wall bonding patterns. In certain scenarios, the strengthening of brick walls using polymer strips was examined. The presence of openings of varying sizes in the walls was also considered. Load–displacement relationships were determined. The study clearly demonstrated that the type of brick bonding significantly affects the wall failure mechanism and that polymer films influence wall stiffness. The numerical results were in satisfactory agreement with experimental findings reported by other researchers.

In another article [11], the results of the analysis of the behavior of brick curtain walls of frame buildings under dynamic and seismic influences are presented. Schemes of destruction of such walls are given. An interesting software tool has been developed to predict the behavior of brick walls under seismic conditions. The tool can be used in the reconstruction and modernization of buildings, strengthening of critical joints, determining the installation locations of damping elements that ensure energy dissipation during seismic action. The proposed software tool can be further improved, allowing users to add new datasets over time, taking into account the latest advances in modeling and analysis.

Numerous studies have focused on methods for reinforcing brick masonry walls.

The monograph of Abakanov et al. [12] describes the main ways to increase the earthquake resistance of buildings with brick walls:

- Rebar nets laid in horizontal masonry seams;
- Creation of composite structural systems by strengthening walls with vertical reinforcing meshes embedded in a shotcrete layer of not less than class B7.5;
- Creation of an integrated structure by including monolithic horizontal and vertical reinforced concrete elements in the masonry;
- Installation of an internal reinforced concrete layer in the masonry (three-layer masonry).

In their comprehensive study, Wu et al. [13] proposed strengthening unreinforced brick masonry walls using reinforced cement mortar (RCM). Over 30 tests of low-strength brick walls under quasi-static conditions have been performed. A four-story brick building in the city of Beijing was also considered as an object of research. It has been established that this method of reinforcing brick walls increases the seismic resistance of structures, reduces the likelihood of destruction during seismic impacts and increases the level of resistance to external dynamic influences.

A study of the operation of low-rise masonry buildings was conducted in the article of Liu et al. [14]. It is noted that masonry buildings have good thermal and sound insulation, durability and fire resistance, and are also relatively cheap. Therefore, buildings of this type are the main ones for construction in rural areas of China. A test was carried out on a vibrating stand of a brick wall, and schemes for the destruction of brick walls reinforced with polypropylene mesh were obtained.

In the article of Maheri et al. [15], using the ABAQUS software package, a theoretical analysis of the destruction of brick walls using nonlinear models was performed. It is revealed that the parameters of a brick wall are essential for the analysis of the strength and deformability of brick walls—the ratio of the sides of the walls, the size of the opening in the wall and the compressive strength of the brick wall.

Another paper [16] examines the vulnerability of stone buildings with non-reinforced walls. Many such buildings were destroyed in the earthquake on 24 August 2016 in central Italy. The forms of destruction of such walls have been studied experimentally and numerically. It is noted that a significant vertical component of the seismic impact leads to a form of destruction of the stone wall in the form of subsidence.

Zhan et al. [17] proposed the reinforcement of brick masonry walls by applying a surface layer of ultra-high-performance engineered cementitious composite (UHP-ECC) in combination with a wire mesh. Four reinforced walls and two unreinforced walls were experimentally subjected to cyclic loading. The results demonstrated that this reinforcement technique substantially improves energy dissipation and increases wall stiffness.

1.2. Object Statement and Relevance

In 2017–2018 and 2023–2024, a comprehensive survey of the multi-apartment residential building stock was conducted in Almaty, as reported by Aldakhov [18], Tuleev et al. [19], and Shokbarov [20]. As a result of this survey, electronic records (passports) of the examined buildings were compiled.

The electronic building passports include the following information: administrative district; address; building name and function; year of construction; standard design series (if available) according to which the inspected building was constructed; building plan configuration; space-planning and structural solutions; presence (or absence) of major overhaul; presence (or absence) of extensions; availability of gas supply and elevators; soil categories according to seismic properties; seismicity of the construction region and of the specific construction site; location within a zone of possible surface manifestation of a tectonic fault; presence of seismic joints; implementation of seismic protection measures;

a brief conclusion on the technical condition of structural elements; availability of evacuation routes and assembly areas in case of earthquakes; assessment of seismic vulnerability; photographs of façades and roofs taken using rotary-wing unmanned aerial vehicles (UAVs); and proposed measures for re-planning and reconstruction. The electronic passports of each building were entered into the electronic map of Almaty.

The reliability and risk assessments of such buildings were carried out below using the results of certification.

One of the objectives of the study is to compare the a priori and a posteriori probability of failure of masonry buildings based on the results of previous studies and the features of seismic impact, as well as the results of certification.

The existing development in Almaty is very typical for most urbanized areas of Central Asia.

In the construction of Almaty, there are masonry buildings of various structural types, number of floors and earthquake safety. Approximately 60% of the population of Almaty lives and works in buildings constructed according to standard designs developed in the period from the late 1950s to the late 1980s. Consequently, the safety of the majority of the population of Almaty and the level of material losses during severe earthquakes depend most on the effectiveness of the design solutions of these buildings and the physical wear of their structures during operation.

Therefore, the findings of reliability assessments for brick masonry buildings with load-bearing walls are highly relevant for major cities in Central Asia exposed to comparable seismic conditions. For instance, Bishkek, the capital of the Kyrgyz Republic, is located in a seismic zone of intensity 9 and contains a substantial portion of residential buildings with load-bearing brick walls. Consequently, the results of this study can be applied in the construction industry of the region.

The most objective information about the effectiveness of structural solutions for buildings and structures erected in seismic areas can be obtained based on an analysis of their conditions after strong earthquakes. However, earthquakes are quite rare events, and in these conditions, the results of expert assessments based on special experimental studies and studies of the effects of strong earthquakes are of crucial importance for the theory and practice of earthquake-resistant construction.

The main purpose of this work was to assess the earthquake resistance of residential buildings erected 40–50 years ago in accordance with standard projects of the 308 series at sites with a seismicity of 9 points.

The assessment of the earthquake resistance of these buildings was carried out through a comprehensive analysis:

- compliance of building design solutions with the requirements of modern standards;
- materials of a detailed survey of some representative houses;
- the results of experimental studies.

The initial data for the work were:

- typical projects of existing residential buildings of the 308 series;
- the provisions of the building regulations in seismic areas that were in force during the design period of the buildings under consideration and are currently in force;
- the results of experimental studies of a natural object.

A three-story representative house, built according to the standard design of the 308 series, was adopted for experimental research. The representative house was located at 26 Satpayev St., Almaty.

The main objective of the experimental research is to obtain objective data on the actual earthquake resistance of residential buildings constructed according to standard designs of the 308 series.

The main aim of this study is to assess the reliability (probability of failure-free performance) and failure risk of a brick masonry building, based on experimental data on its behavior under dynamic loading and a mathematical model developed using a probabilistic seismic hazard framework. Furthermore, the obtained reliability and failure metrics are compared with the corresponding values derived from the results of structural certification.

The completed research will contribute to:

- creation of a database that allows forecasting the condition of existing buildings after possible strong earthquakes;
- improving the effectiveness and validity of decisions made in the earthquake preparation process;
- reducing the time required for design work during the reconstruction of existing buildings.

2. Materials and Methods

2.1. Multi-Family 2-Story Residential Buildings with Load-Bearing Brick Walls

Multi-apartment 2-story residential buildings with load-bearing brick walls were designed based on the static theory of earthquake resistance and were built in the period from 1930 to 1957.

The foundations of buildings and basement walls are precast reinforced concrete foundation blocks 500 mm thick and made of monolithic concrete. The depth of the foundations is 1.3 m (2.8 m if there is a basement).

The aboveground part of the buildings is designed with load-bearing brick walls. The outer walls are 510 mm thick; the inner walls are 380 mm thick.

The floors were solved in two versions: using prefabricated or monolithic reinforced concrete slabs and using wooden beams with a panel flooring.

2.2. Multi-Apartment 3- and 4-Story Residential Buildings with Load-Bearing Brick Walls

The category of multi-family 3- and 4-story residential buildings with load-bearing brick walls includes houses designed according to standard series 261, 275, 308.

The 261 series was developed in 1955. The buildings in this series have three floors with a basement. The foundations are made of rubble concrete, and the basement walls are made of precast concrete blocks. The floors were solved in two versions: using precast reinforced concrete slabs and using wooden beams with a panel flooring.

The buildings of the 261 series are characterized by:

- low strength of brickwork;
- insufficient rigidity of wooden floors;
- some of the internal load-bearing walls were interrupted by the width of the building;
- the narrow walls were not reinforced;
- the limit value of the ratio of floor height to wall thickness is underestimated;
- exceeding the maximum permissible height;
- significant physical deterioration of structures.

The 275 series was developed in 1957. The buildings in this series have four floors with a basement. The foundations are made of rubble concrete, and the basement walls are made of precast concrete blocks. The floors were made of prefabricated multi-cavity reinforced concrete slabs with reinforced concrete bindings.

The buildings of the 275 series are characterized by:

- low strength of brickwork;
- the narrow walls were not reinforced;
- exceeding the maximum permissible height;
- significant physical deterioration of structures.

The 308 series was developed in 1958. The buildings in this series have three or four floors with a basement (Figures 1 and 2). The foundations are made of rubble concrete, and the basement walls are made of precast concrete blocks. The floors were made of prefabricated multi-cavity reinforced concrete slabs with reinforced concrete bindings.



Figure 1. Four-story building with load-bearing brick walls.



Figure 2. A three-story building with load-bearing brick walls.

During the certification, 663 multi-family 3- and 4-story residential buildings with load-bearing brick walls of the 308 series were examined.

The buildings of the 308 series are characterized by:

- low strength of brickwork;
- the narrow walls were not reinforced;
- exceeding the maximum permissible height;
- significant physical deterioration of structures.

According to the spatial planning and structural solutions, the buildings of the standard series 275 and 308 are solved in the same way.

The primary distinction between brick masonry buildings of series 261 and those of series 275 and 308 lies in the use of wooden floor slabs between stories in series 261. Furthermore, series 261 buildings were exclusively constructed as three-story structures.

A comparative analysis of the comprehensive detailed inspections carried out and the calculation results allows the conclusion that multi-apartment three- and four-story residential buildings with load-bearing brick walls were constructed without seismic protection measures. Their space-planning and structural solutions do not comply with current seismic design codes and are considered seismically hazardous.

Based on the results of the structural certification, only 15 of all brick masonry buildings are recognized as seismic-resistant. Consequently, 99.07% of brick masonry buildings are classified as non-seismic-resistant. These are 1594 houses out of the total number from Table 1. The results were obtained using previously conducted experimental studies, analyses of the consequences of strong earthquakes, accumulated experience from detailed inspections of residential buildings, and data from the engineering-seismology service of KazNIISA JSC.

Table 1. Distribution of masonry buildings with load-bearing walls by series.

№	A Series of Brick Buildings	Number of Buildings
1	2-story buildings	912
2	series 261	30
3	series 275	2
4	series 308	665
5	total	1609

A passport has been drawn up for each of the 1609 buildings, which indicates the soil conditions. The buildings are located on soils of category I and II in terms of seismic properties, with a predominance of category I.

2.3. Experimental Data on Explosive Impact Tests

In 1966 and 1967, dynamic tests of urban development structures in Almaty were conducted through a series of controlled explosions required for the construction of a debris-protection dam in the Malaya Almatinka River gorge (in the Medeo area) according to Zhunusov [21] (Figure 3).

The first right-bank explosion from two series of charges with a total explosive weight of 5294 tons was carried out on 21 October 1966. The second, a left-bank explosion, also from two series of charges with a total explosive weight of about 3900 tons, was carried out on 14 April 1967.

The shaking of the ground in the city was estimated at 3–5 points in the first explosion, and 2–3 points in the second. In the area close to the epicenter of the explosion, with a radius of about two kilometers, the impact corresponded to an earthquake of 7–8 points.



Figure 3. Experimental test site in Medeo, in the center of a 4-story brick building.

Under the leadership of academician T. Zh. Zhunusov, the registration of vibrations of buildings of all types typical for the development of the city of Almaty was organized.

During the first explosion, 23 buildings of existing buildings in the city were examined, during the second, the behavior of six experimental buildings was studied—full-scale fragments of modern earthquake-resistant buildings specially built at a distance of 800 m from the epicenter of the explosion.

During the first explosion, vibrations of two four-story (series 275 and 308) and one three-story buildings were instrumentally recorded. It was found that for two four-story buildings, periods of oscillation in the transverse direction of 0.20–0.22 s were obtained. In the longitudinal direction, the oscillation period increased to 0.22–0.35 s. The logarithmic decrement for both buildings was 0.51–0.52 in the transverse direction, and 0.38–0.41 in the longitudinal direction.

For a three-story brick building with a basement, the oscillation period did not change significantly compared to a four-story building and amounted to 0.20 s. In this case, the logarithmic decrement of the oscillation was 0.47.

During the second explosion, the vibrations of a 4-story brick building of the 308 series, erected at a special landfill near the epicenter of the explosion, were instrumentally measured. At the building's foundation level, an acceleration of 600 cm/s^2 was recorded, while at the level of the 4th-floor slab, the acceleration reached 900 cm/s^2. The building fluctuated with an oscillation period of 0.3–0.5 s; therefore, the oscillation period under high-intensity exposure changed by at least 2 times. The logarithmic decrement was 0.38–0.52.

Thus, there is experimental data on the behavior of masonry buildings with load-bearing walls of those series that were built in Almaty. These experimental data are considered highly reliable and may be employed to validate the results of vibration tests conducted on the three-story brick masonry building of series 308 (Section 3: Result: Dynamic Parameters of the Object and Features of Deformation).

2.4. The Object of Experimental Research

Building designs of series 308 were developed by the "KAZGOR" institute and are intended for the construction of three- and four-story residential buildings on sites with a seismicity of 9 points.

The development of projects for buildings of the 308 series was carried out in accordance with the provisions of CH-8-57 "Norms and rules of construction in seismic areas" and "Construction in seismic areas. Design standards".

The construction of residential buildings with load-bearing brick walls of the 308 series in Almaty began in 1958 and lasted until 1967–1968.

The general view of the four-story building of the 308 series is shown in Figure 4.



Figure 4. A four-stores version of the 308 series brick building.

The building compartments of the 308 series are rectangular in plan.

The buildings of the 308 series were designed with or without a basement. The height of the residential floors (from floor to floor) is 2.8 m; the basement is 2.0 to 2.8 m.

Longitudinally, the rigidity and strength of the building is provided by three walls—one inner and two outer. In the transverse direction, the rigidity and strength of the building are provided by the internal and external transverse walls. All longitudinal and transverse walls run through the entire length of the building.

The pitch of the longitudinal walls (in axes) is 6 m; the pitch of the transverse walls is 8.4, 7.8, 5.8 and 3.0 m.

The foundations are solved in the form of cross-monolithic tapes.

The basement walls are prefabricated made of concrete blocks.

The walls above the 0.00 mark are brick; the outer walls are 51 cm thick, and the inner walls are 38 cm thick. The corners and intersections of the walls, as well as the inner walls, are reinforced with horizontal rebar grids with a diameter of 6 mm of class A-I, laid in height in increments from 30 to 60 cm.

The doorways of the entrances to the stairwells have a reinforced concrete frame.

The minimum width of the piers, as a rule, is:

- in three-story buildings—1.30 m;
- in four-story buildings—1.45 m.

The design category of masonry for resistance to seismic effects is the first category (i.e., the temporary axial tensile strength of masonry at unbonded joints must be at least 1.8 kg/cm^2).

The floors are precast reinforced concrete made of 220 mm thick multi-cavity panels connected to antiseismic belts by means of interlocking hinges. The depth of the floor slabs on the walls is 120 mm.

The roof is pitched from wavy asbestos cement sheets on wooden rafters and a crate.

2.5. Analysis of the Compliance of the Structural Solutions of the 308 Series Building with the Requirements of Modern Standards

2.5.1. Analysis of Spatial Planning and Structural Solutions of Buildings of the 308 Series

The technical solutions of the 308 series buildings comply with the following requirements of the current earthquake-resistant construction standards:

- The dimensions of the compartments of buildings of the 308 series do not exceed the dimensions specified in Table 7.1 of the SNiP RK 2.03-30-2006;
- The buildings have a regular shape in the plan that meets the requirements of paragraph 3.4 of the SNiP RK 2.03-30-2006; there are no height differences within the compartments;
- In cases defined in clause 7.2 of the SNiP RK 2.03-30-2006, buildings are divided by antiseismic seams into compartments; antiseismic seams between individual compartments are made by installing paired walls over the entire height of buildings, which meets the requirements of clauses 7.6.1 and 7.6.2 of the SNiP RK 2.03-30-2006;
- The location and construction of stairwells meet the requirements of paragraphs 7.11 and 7.12 of the SNiP RK 2.03-30-2006;
- The spatial system of buildings of the 308 series meets the requirements of paragraphs 7.8, 7.23.a, 7.24 of the SNiP RK 2.03-30-2006.

In the constructions of buildings of the 308 series, there are the following inconsistencies with the requirements of current regulations:

- The height of buildings does not meet the requirements of Table 7.2 of the SNiP RK 2.03-30-2006; according to Table 7.2, the height of buildings with brick walls erected in areas with a seismicity of 9 points should not exceed two floors;
- The distance between the axes of transverse walls having only horizontal reinforcement does not meet the requirements of Table 7.4 of the SNiP RK 2.03-30-2006; the distance between such walls should not exceed 6 m;
- The walls of buildings with a width of 1.30 m and 1.45 m do not meet the requirements of Table 7.7 of the SNiP RK 2.03-30-2006; the width of the piers must be at least 1.55 m;
- In the levels of floors and coverings, instead of antiseismic belts, bindings made of monolithic reinforced concrete are provided, which contradicts the requirements of clause 7.97 of the SNiP RK 2.03-30-2006;
- Prefabricated bar lintels are provided above the window and door openings, which contradicts the requirements of clause 7.113 of the SNiP RK 2.03-30-2006.

As an additional remark, it should be noted that tests of brick wall masonry, conducted in different years by the staff of KazNIISA in accordance with the requirements of GOST 24992-81, showed that the temporary axial tensile strength of masonry at unbonded joints is within the following range:

- 1.24 to 1.43 kg/cm^2 for individual objects;
- 0.30 to 0.60 kg/cm^2 for most objects.

That is, the actual value of the temporary resistance of masonry walls to axial stretching along loose seams (normal adhesion) in many cases is 3 to 6 times less than that required by the project and 2 to 4 times less than that required according to clause 7.95 of the SNiP RK 2.03-30-2006.

2.5.2. Analysis of Design Provisions in Effect at the Time of the Series 308 Building Designs and Those Currently in Force

According to the results of numerous experimental studies and calculations, the initial pitch period of natural vibrations of three- and four-story buildings of the 308 series is 0.15–0.25 s. Table 2 shows data characterizing the values of the calculated seismic loads determined for buildings with the specified values of periods with the same seismicity of the construction site:

- in column 2—according to the provisions of SN-8-57;
- in column 3—according to the provisions of the SNiP RK 2.03-30-2006 (for soils of categories I and II in terms of seismic properties).

Table 2. Estimated seismic loads according to two regulatory documents.

Indicator	CH-8-57	SNiP RK 2.03-30-2006
1	2	3
The relative magnitude of the estimated seismic load, in %	100	167

It follows from Table 2 that the values of the horizontal calculated seismic loads, determined in accordance with the provisions of the SNiP RK 2.03-30-2006, 67% more than the values of the horizontal calculated seismic loads determined in accordance with the provisions of CH-8-57.

According to the provisions of the SNiP RK 2.03-30-2006 (clause 5.16), when calculating buildings with a length or width of more than 30 m, it is necessary to take into account the horizontal torque relative to the vertical axis of the building passing through its centers of rigidity at floor levels.

In CH-8-57, this requirement was absent and calculations of buildings with a length of more than 30 m (for example, consisting of two or more block sections) for additional torques were not performed.

According to the provisions of the SNiP RK 2.03-30-2006 (clause 5.5.e), the calculation of stone buildings should be based on the simultaneous action of horizontally and vertically directed seismic forces.

There was no such requirement in CH-8-57.

According to the norms of CH-8-57, when calculating structures for strength and stability, additional coefficients of the working conditions of the brickwork were taken into account, increasing its design resistance by 1.2 to 2.0 times (depending on the norms and stress-strain state of the calculated structure).

In the SNiP of the Republic of Kazakhstan 2.03-30-2006, there are no such coefficients or they are equal to 1.0.

As an example, it should be noted that the verification calculations of a four-story building for the basic and special load combinations, taking into account seismic effects, performed by specialists of KazNIISA in 1985 (in accordance with the requirements of SNiP II-7-81), showed that the load-bearing capacity of category II masonry under principal tensile stresses was below the required level:

- in many sections of the longitudinal exterior walls by 1.1–2.5 times, in some up to 3 times;

- for some internal longitudinal walls—1.2 times;
- in many cross-wall panels (usually exterior walls) 1.6–2.5 times, in some cases up to 4–5 times.

From a joint review of the above data, it can be seen that the estimated load-bearing capacity of the 308 series of building structures, determined based on the requirements of previously existing standards, is significantly lower than the design strength required by current standards.

2.5.3. Earthquake Resistance Assessments of 308 Series Buildings Contained in Regulatory and Instructional Documents

Below are the conclusions regarding the seismic resistance of series 308 buildings, contained in previously developed regulatory and instructional documents that were in effect in the territory of the Kazakh SSR.

Conclusions on the seismic resistance of series 308 buildings, contained in the "Guide for Inspection and Assessment of Seismic Resistance of Existing Buildings" (KazpromstroyNIIproekt, 1986):

- … "the computational and analytical assessment shows that the building in question is earthquake-prone …".
- … "the building does not meet the structural requirements and strength conditions accepted in current regulations and needs to be strengthened."

Conclusions on the seismic resistance of series 308 buildings, contained in the "Methodological Recommendations for the Registration of Existing Buildings in Almaty and Other Settlements Located in Seismically Hazardous Areas of the Kazakh SSR" (KazpromstroyNIIproekt, 1989).

The buildings of the 308 series belong to category 10 (out of 13 possible) in terms of earthquake resistance, "… do not meet the requirements of modern standards and are earthquake-prone."

Conclusions on regulatory documents:

1. Standard designs of three- and four-story buildings with load-bearing brick walls of the 308 series do not meet the requirements of the standards governing construction in seismic areas of the Republic of Kazakhstan.
2. Analysis of the results of calculations, tests and the effects of earthquakes, as well as verification of the actual strength of the brickwork walls indicate that buildings of the 308 series should be considered earthquake-prone.
3. Considering that the low level of earthquake resistance of buildings of the 308 series poses a direct threat to the safety of people living in them, it is necessary to:
    - conduct a detailed survey of 1–2 representative houses and carry out an experimental and theoretical assessment of the degree of their actual earthquake resistance;
    - based on the results of the survey and experimental and theoretical research, develop a project to strengthen the structures of buildings.

The above data clearly show that all regulatory and instructional documents developed by different specialists in different periods of time have a very negative assessment of the earthquake resistance of buildings of the 308 series.

2.6. Characteristics of the Test Object

A three-story residential building located at 26 Satpayev St., Almaty, was adopted for experimental research.

The test facility is based on a standard design of the 308 series. The estimated seismicity of the building, according to the design data, is 9 points.

The tested building has a ground floor and three residential floors. The shape of the building is simple in plan and height.

The height of the basement and residential floors is 2.8 m. The dimensions of the building in the extreme axes are 53 × 12 m.

The general view of the test object is shown in Figure 5.



Figure 5. Experimental object.

2.7. The Method of Creating Dynamic Loads and Recording Equipment

In the experimental studies performed, dynamic loads on the object under study were created using an inertial vibration machine of type B-3.

The set of power equipment for vibration testing of the experimental facility included:

- 200 kW DC motor;
- five two-shaft vibrators with horizontal axes of rotation of the balance levers;
- finishing weights are unbalances hung on the levers of vibrators;
- a control panel that allows you to smoothly adjust the engine shaft speed.

During the tests, the motor and vibrators were rigidly fixed to a horizontal steel frame located in the floor level above the 3rd floor of the building. The steel frame, in turn, was rigidly fixed to this floor and sealed.

The layout of the vibration machine and the arrangement of accelerometers on the building surface are shown in Figure 6. Sensors were installed in the basement and on each floor of the building. The red paint on the building cover shows the location of the vibrating machine.

Figure 6 shows that the resultant of the forces generated by the vibrating machine was directed in the longitudinal direction of the building.

The general view of the vibrating machine on the coating is shown in Figure 6.

Vibrations are recorded using the digital recording system RCM-16. Measurements are carried out with a sampling frequency of 156.25 Hz (sampling step 0.0064 s) and frequency filters in the range of 0.5 Hz—25 Hz. ADX accelerometers, designed for measuring linear accelerations, are employed.

Metrological support of the tests is ensured. To this end, the measuring instruments undergo mechanical verification on a dedicated calibration vibration stand. Instrument certificates are issued based on the verification results. These procedures guarantee the accuracy and reliability of the measurements.

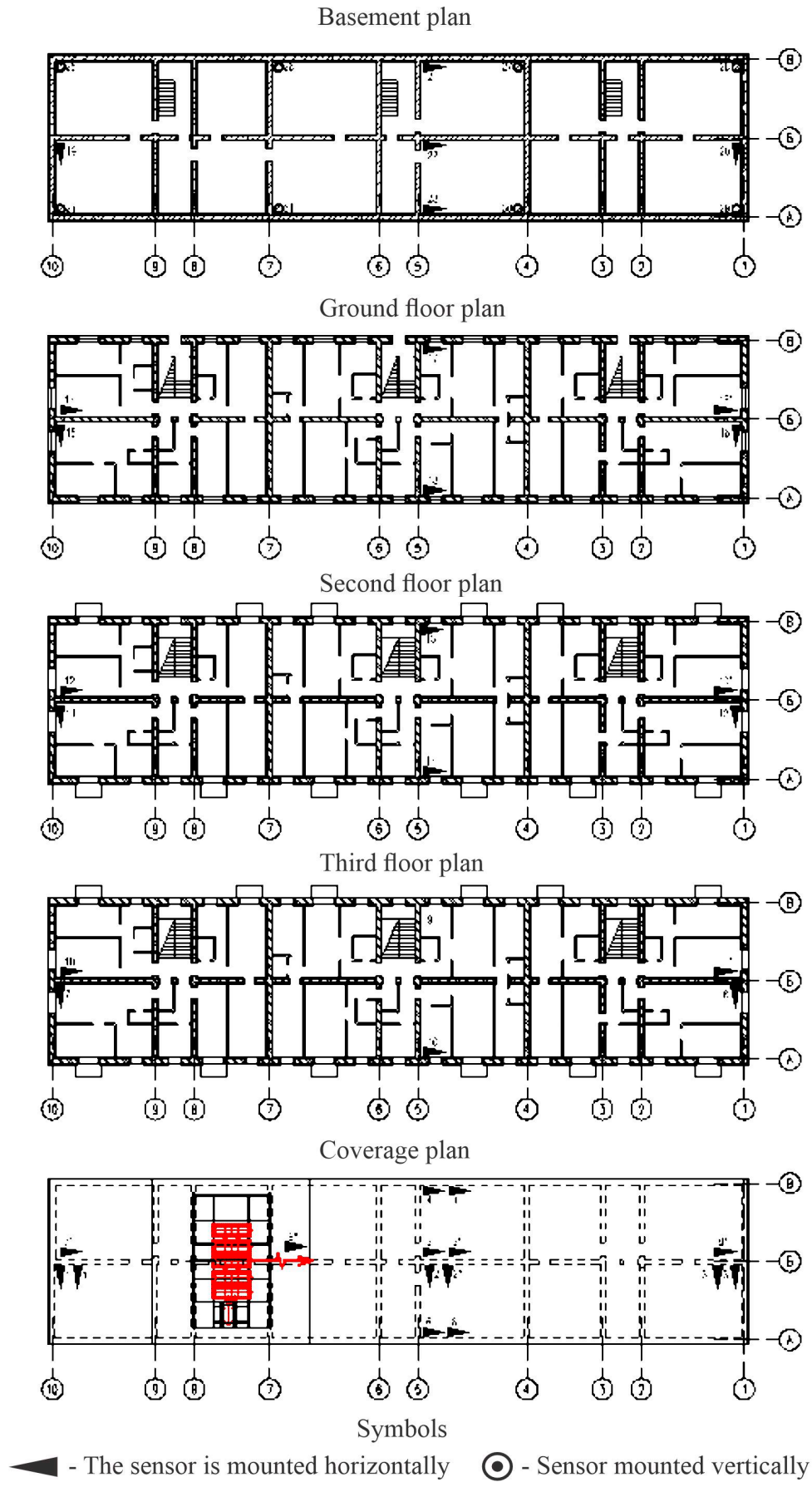

Figure 6. Arrangement scheme of devices.

Before the start of the vibration tests, microseismic vibrations of the experimental object were recorded.

The vibration tests included 7 steps described in Table 3.

Table 3. Characteristics of vibratory machine loads.

Test Stage Number	Quantity of Interlocked Vibrators	Number of Unbalanced Loads on Each Vibrator
I	1	0
II	2	0
III	2	12
IV	2	28
V	2	28

Each stage of the tests consisted of a two-fold smooth passage through the resonances: first, by increasing the rotational frequency of the unbalances ("direct resonance"), and then by reducing the rotational frequency of the unbalances ("reverse resonance").

During the vibration tests, the following were performed: vibration registration and pre-processing of data, visual inspection of structures, and photo and video recordings of the object.

The registration of instrumental data was carried out using a special software and hardware complex developed according to the technical specifications of the KazNIISSA Institute specialists. The general view of the hardware and software complex and the sensors mounted on the building structures are shown in Figure 7.



Figure 7. Inertial vibration machine.

The hardware part of the complex consists of an analog signal input unit with an analog-to-digital conversion device, to which a PC-based signal recording and processing unit is connected (Figure 7).

The accepted arrangement of the sensors made it possible to fix:

- spatial waveforms of a building;
- horizontal deformations of floor-to-floor ceilings due to their malleability;

- horizontal and vertical deformations of the building base;
- horizontal misalignment of building floors.

3. Result: Dynamic Parameters of the Object and Features of Deformation

3.1. Initial Dynamic Parameters of the Object

The initial dynamic parameters of the object under study were identified based on the study of instrumental recordings of its microseismic oscillations.

It follows from the records obtained that before the start of the vibration tests, the periods of free oscillations of the experimental object in the longitudinal direction were 0.15 s.

3.2. Features of Behavior and Dynamic Parameters of an Object Under Vibrodynamic Influences

Instrumental data and the results of visual observations indicate that at all stages of testing, the building's oscillation amplitudes in the longitudinal direction in the area in axes 6–10 were significantly greater than in the area in axes 1–5.

This fact is explained by the following reasons:

- The vibrating machine was located on the edge section of the building covering in the axes;
- The prefabricated horizontal discs of the coating and floors did not have sufficient rigidity to evenly distribute horizontal loads from the vibrating machine along the entire length of the building;
- There were essentially no internal longitudinal walls within the stairwells, which significantly weakened the connections between the building sections adjacent to the stairwells.

A large number of instrumental recordings of displacements and accelerations have been obtained. In Figure 8, the blue and red lines represent samples of instrumental recordings from stage IV in Table 4. The highest acceleration values, up to 1400 cm/s^2, were recorded during stage IV. The damping ratios across the different test stages ranged from 0.25 to 0.52.

Table 4. Periods of resonant oscillations.

The Periods of Resonant Vibrations (Seconds) of the Test Object at the Stages						
I	II	III	IV	V	VI	VII
0.173	0.222	0.243	0.371	0.448	0.493	0.531

In particular, with the simple harmonic nature of the external effects from the vibrating machine, the vibrations of all fragments had a complex periodic (i.e., inharmonic) character, most clearly observed on the acceleration recordings.

The main components of periodic fluctuations of each fragment of the building can be distinguished:

- a component in the form of a simple harmonic having the frequency of the disturbing force of the vibrating machine; the frequency of this component of periodic oscillations was the lowest (main);
- a component in the form of short pulses, repeated with the frequency of the disturbing force of the vibrating machine, and small oscillations between them.

The resonant oscillation periods of the object under study at different test stages are shown in Table 4.

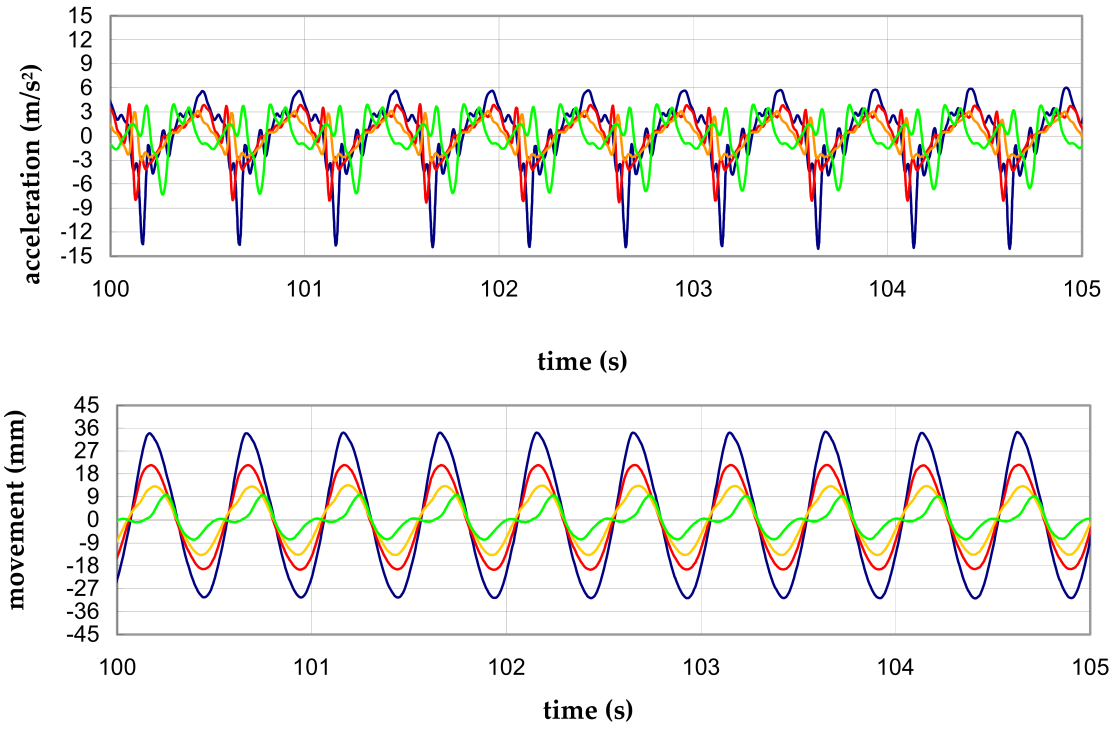

Figure 8. Experimental recordings of accelerations and displacements, different colors correspond to different stages of testing.

3.3. Damage to the Test Object Due to Vibrodynamic Effects

The first clearly visible damage to the building was noted during test phase II. At this stage of the tests, the following damage was observed:

- mass shedding of plaster pieces from floor slabs;
- shedding of pieces of plaster from some areas of the walls;
- contouring of reinforced concrete lintels over openings with thin cracks;
- formation of diagonal cracks in some lintels above the window openings of the first and second floors.

During the testing phase IV, the building was severely damaged:

- Diagonal and horizontal cracks with large openings appeared in the load-bearing walls of the exterior walls;
- Diagonal cracks formed in almost all the lintels above the window openings of the exterior walls;
- The end wall along the 10 axis was separated by cracks from the longitudinal walls;
- Cracks formed in the inner longitudinal wall, separating it from the transverse walls;
- Many of the partitions on the second and third floors collapsed or sustained significant damage; some of the partitions on the first floor were damaged;
- The mortar had partially or completely fallen out of the joints between most floor slabs.

The condition of the building structures after test phase IV is shown in Figures 9–12.

During the testing stages V–VII, the volume of structural damage to the building increased significantly:

- The vast majority of the vertical load-bearing structures of the building were severely damaged; some of the piers collapsed;
- Many of the lintels above the window openings collapsed and the floor slabs were held only in connection with antiseismic belts;

- In the areas of the collapsed lintels, the floors received small deflections from the plane; some slabs of prefabricated floors and coverings shifted relative to their original position;
- The upper section of the end wall collapsed;
- Some sections of the brickwork became stratified due to its low strength;
- Most of the partitions in the second and third floor levels collapsed or suffered severe damage; most of the partitions on the first floor were damaged;
- There was movement of the staircases.



Figure 9. Experimental object after the test.



Figure 10. Typical object damage (diagonal cracks).



Figure 11. Typical object damage.



Figure 12. Damage inside the building.

The characteristic damage to the building after the completion of the tests is shown in Figures 11 and 12.

Quantitative assessment of building damage caused by vibrodynamic impacts, performed on the basis of the CH RK 2.03-28-2008 The "MSK-64 (K) earthquake intensity scale" is given in Table 5.

Table 5. The degree of damage to the object.

Test Stages	Degree of Damage
I	0
II–III	2
IV	3
V–VII	4

Experimental studies using a vibrating machine are based on the excitation of resonant vibrations of the tested object. The assessment of the intensity of the experimental intensity c occurring during an earthquake is based on the "Seismic Intensity Scale MSK—64(K)". In the last stages of the test, even the upper section of the end wall collapsed, and most of the partitions on the second and third floors collapsed or were severely damaged. During vibration tests, a brick building (type Bs on the scale) was damaged to the fourth degree. Therefore, the intensity of the created resonant dynamic impacts in terms of damage degrees can be estimated as equivalent to a seismic impact of 9 points.

Based on experimental data, it was found that the fourth degree of damage occurred at building skew values [h] = h/240, where h is the height of the building. The height of the building is 8.4 m. Therefore, [h] = 3.5 cm.

The above value will be used in Section 4 to assess the reliability of the building.

3.4. Comparison of Measurement Results from the Medeo Explosion and Vibration Tests

The free oscillation period, according to vibration tests, is 0.17 s; according to the Medeo tests, it is 0.2 s. During the Medeo explosion, the oscillation period varied by up to

0.5 s. During the step-by-step vibration tests, the period was changed to 0.53 s at the final stage of testing. Thus, the measurement results are quite close.

The acceleration on the building's roof during the Medeo explosion is 1200 cm/s^2. During vibration tests, this characteristic is 1400 cm/s^2. The difference here is approximately 8.5%. Thus, the differences are not very large, but the intensity of the vibration tests was higher than the results of the Medeo explosion. The results of the vibration tests and the Medeo explosion are generally comparable. Dissipative characteristics during vibration tests (oscillation decrement) vary within the range of 0.25–0.54, while during tests in Medeo, they ranged within the range of 0.40–0.50.

Changes in the building's oscillation period are a quantitative characteristic of the nonlinearity of deformation. The oscillation period during vibration tests varied by more than three times. Approximately the same result was obtained during the explosion in Medeo.

It was revealed in the previous section that the building received a fourth degree of damage during resonance tests with a vibrating machine. During seismic and explosive impacts in Medeo, the building received a similar degree of damage. According to the seismic intensity scale MSK-64(K), this corresponds to a seismic impact intensity of at least 9 points. Thus, the macroseismic intensity estimates coincided.

The above dynamic parameters are further used in the calculation of reliability and failure values (Section 4).

3.5. Assessment of the Degree of Nonlinear Deformation of a Building

The experimental data obtained indicate that fragment No. 1 was deformed as a significantly nonlinear system during the vibrodynamic tests.

The nonlinear nature of the deformation of fragment No. 1 was caused by structural damage to its aboveground part. The contribution of the base to the deformation of the fragment was insignificant.

The degree of development of nonlinear deformations in the structures of fragment No. 1 can be estimated by comparing the periods of its oscillations during passage through resonances with periods of free oscillations under microseismic influences.

The values of the oscillation periods of the fragment during passage through the resonances significantly depended on the level of the loads acting on it. At test stage II, they were 1.5 times higher than the initial values of the oscillation periods. At test stage IV, the oscillation periods during passage through resonances were 2.5 times higher than the initial values of the oscillation periods, and at test stage VII, 3.5 times higher.

From the above data, it follows that the stiffness of fragment No. 1 at test stage IV was 16% of the initial, and at stage VII, 8% of the initial.

The repeated decrease in stiffness indicates that after the testing stage IV, fragment No. 1 lost its ability to resist seismic influences.

Reinforcement of masonry with reinforcing mesh is not provided for the exterior walls of buildings of the 308 series. For this reason, the external longitudinal and transverse walls of the studied object were significantly more damaged than the inner wall.

Prefabricated bar lintels were used in the buildings of the 308 series, which caused their massive collapse.

It should be particularly noted that according to the test results, not only load-bearing structures, but also partitions in buildings of the 308 series have an extremely low ability to resist seismic impacts.

The massive collapse of partitions made of gypsum boards fixed in a wooden frame with thin steel plates on nails poses a direct threat to the safety of people and the safety of their property during earthquakes.

According to the scientific and methodological foundations of modern standards "Construction in seismic areas", earthquake-resistant objects:

- when applied to them by design loads, they should not have any significant damage;
- in case of real seismic impacts, they may have damage that does not threaten the safety of people.

According to the descriptive part of the MSK 64-(K) seismic intensity scale and the description of the condition of structures at test stages II and IV, it follows that the buildings of the 308 series do not have the above properties.

The tested object, under the influence of single-component horizontal loads, the values of which corresponded to the calculated values and did not exceed those possible in real earthquakes with an intensity of 8 points, received severe damage threatening human life and safety.

The actual seismic resistance of the tested object, based on the scientific and methodological foundations of modern standards, does not exceed 7 points.

3.6. Forecast of the Condition of Buildings of the 308 Series After Real Earthquakes

The analysis of these subsections allows us to make the following forecast of the conditions of buildings of the 308 series after real earthquakes.

Buildings of the 308 series are able to withstand an earthquake intensity of 6 points without significant damage to load-bearing and non-load-bearing structures.

Degree of damage to CH RK 2.03-28-2004: The "MSK-64 (K) earthquake intensity scale" for most buildings of the 308 series will not exceed grade 1.

In case of earthquakes with an intensity of 7 points, grade 2 damage may occur in the load-bearing structures of many buildings of the 308 series. Some curtain structures (partitions) will receive damage of grade 3.

In case of earthquakes with an intensity of 8 points, grade 3 damage will occur in the load-bearing structures of many buildings of the 308 series, and in some, grade 4. Quite a few partitions will be severely damaged or collapse.

In case of earthquakes with an intensity of 9 points, many buildings of the 308 series will receive grade 4 damage, and some of them will receive grade 5 damage.

The forecast of the conditions of the 308 series buildings after real earthquakes was made based on the test results of a three-story representative facility. Other buildings in this series may have slightly different degrees of damage, depending on their height, construction quality, and earthquake spectral composition. However, deviations from the estimates made to a greater or lesser extent will not be so significant.

3.7. Conclusions Based on the Results of Experimental Studies

The analysis of the results of vibrodynamic tests of a three-story building of the 308 series allows us to draw the following main conclusions.

- The building behaved as a significantly nonlinear system and sustained substantial damage. At this level of damage, restoration with subsequent strengthening is generally considered impractical.
- The standard designs of three- and four-story buildings with load-bearing brick walls of the 308 series do not meet the requirements of the current regulations governing construction in the seismic regions of the Republic of Kazakhstan.
- Analysis of the results of calculations, tests and the effects of earthquakes, as well as verification of the actual strength of the brickwork walls indicate that buildings of the 308 series should be considered earthquake-prone.
- The low level of earthquake resistance of the 308 series buildings poses a direct threat to the safety of the people living in them.

- The buildings of the 308 series are to be reinforced or demolished.

4. Mathematical Model for Reliability Assessment

The assessment of the reliability of a building with load-bearing brick walls is understood here as the value of the probability of trouble-free operation W, obtained using experimental data from Section 3, taking into account the quantitative spectral and temporal characteristics of the seismic impact model and the recurrence of earthquakes in Almaty city region.

Reliability is understood as the probability of trouble-free operation W = W(|x| < [h]), where the value [h] is taken from Section 3.3 according to experimental data and is equal to [h] = 3.5 cm.

At the same time, instrumental recordings of the engineering seismometric service of "KazNIISA" JSC and seismic stations of the Institute of Seismology (now the "National Scientific Center for Seismological Observations and Research of the Ministry of Emergency Situations of the Republic of Kazakhstan" LLP) are used. This allows us to take into account the regional features of the seismic impact in the Northern Tien Shan region.

In general, a single-mass cantilever system with a concentrated mass and a deformation diagram (DD) can serve as an acceptable design model for analyzing the reaction and evaluating the reliability of such buildings. The differential equation is integrated

$$m\ddot{x} + \mu\dot{x} + R(x) = -m\ddot{x}_{0i} \quad (1)$$

where

$R(x)$—nonlinear restoring force (deformation diagram),

$\ddot{x}_{0i}$—Ist accelerogram,

μ—coefficient of inelastic resistance (Voigt's hypothesis), accepted according to experimental data.

m—building mass, m = Q/g, g—acceleration of free fall,

x—horizontal movement.

Calculations assume Q = 9500 kN, μ = 250 kN c/sm.

The deformation diagram is approximated by a piecewise linear relationship. The inflection points are shown in Table 6.

Table 6. Inflection points of the nonlinear deformation diagram.

Parameters	Inflection Points						
Reaction, kN	−8500	−7822	−6000	0	6000	7822	8500
Movement, cm	−2.5	−1.0	−0.8	0	0.8	1.0	2.5

The seismic impact was modeled by an unsteady random process, which is obtained by multiplying the realizations of the stationary process by a deterministic envelope.

The correlation function of a normal stationary random process is assumed as

$$K(\tau) = \sigma^2 e^{-\alpha|\tau|}(\cos(\omega\tau) + \left(\frac{\alpha}{\omega}\right)\sin(\omega\tau)) \quad (2)$$

where α is the correlation parameter, ω is the carrier frequency of the impact, and σ is the acceleration standard based on the calculated model.

The deterministic envelope is assumed to be a fractional rational function of F. F. Aptikaev.

$$A = A_{max}\frac{3td}{9t^2 - 9td + 4d^2} \quad (3)$$

The value d denotes the effective duration of the seismic impact (the duration of the impact with an intensity of at least half the maximum is the pulse width), and t is time. The shape of the envelope of the accelerogram is an important characteristic of the seismic impact, which significantly affects its intensity. The type of envelope (3) also corresponds to the regional features of seismic impact for the Northern Tien Shan region and Almaty city.

Let us calculate the building, taking into account the regional features of seismic impact for Almaty city, a megalopolis with a population of over 2.5 million people. According to the current seismic zoning map of the Republic of Kazakhstan, the median acceleration values in the city are 0.38 g at a repeat rate of one time in 475 years, and 0.73 g at one time in 2475 years. Here, g is the acceleration of gravity.

The two indicated acceleration values correspond most closely to two accelerograms of the 1990 Baysorun earthquake with acceleration peaks of 699.2 cm/s^2 and 436.9 cm/s^2. Accelerograms were obtained by seismic stations of the Institute of Seismology. The differences in peak acceleration values from the values normalized by the map of seismic zoning of Almaty city and adjacent territories are 2–8% here. Therefore, these accelerograms can be taken as the initial ones for modeling the seismic impact by a random process. Table 7 shows some parameters of instrumental recordings of accelerograms.

Table 7. Accelerogram parameters.

Parameters	Component, 1 (N-S)	Component, 2(E-W)
Maximum accelerogram, cm/s^2	699.2	436.92
Minimum accelerogram, cm/s^2	−589.85	−375.53
Dispersion, cm^2/s^4	69.64	72.05
RMS value, cm/s^2	8.34	8.49
Range, cm/s^2	1289.05	812, 450
Average value, cm/s^2	4.193	6.098
Median value, cm/s^2	1.600	9.845
Maximum spectral density	42.20; 36.58	39.69
Frequency, 1/s	3.66; 4.76	4.88
Period, s	0.27; 0.21	0.20
The duration of oscillations with an amplitude of more than half of the maximum (effective duration), s	2.52	2.50

A cursory analysis of Table 7 shows that the accelerograms are well centered. Instrumental recordings are high-frequency. The predominant periods and carrier frequencies are determined by spectral density using the SCM MATLAB R2018b Signal Processing Toolbox package. With a total exposure duration of more than 8 s, the effective duration (the duration of oscillations with an amplitude of more than half the maximum) is 2.5 s, i.e., approximately 30% of the total duration. The effective duration is one of the characteristics of the recommended deterministic envelope (3).

The ratio of the maximum acceleration values of the component is 1.6 times.

The reliability value W is a characteristic of the earthquake resistance and quality of the building.

Calculations were performed by statistical testing (Monte Carlo method) using the capabilities of the MATLAB and SCILAB computer mathematics systems. Sequentially, the implementations of a random process with the above correlation function and envelope were formed using the shaping filter method. The RMS acceleration values were selected in such a way as to ensure that the average acceleration value coincided with the maxima of the accelerograms from Table 7. The nonlinear differential Equation (1) is integrated numerically. A combination of the multistep Adams method and the inverse differentiation

method was used for this purpose. For each implementation, the kinematic parameters of the building model were determined and the failure criterion was checked.

In order to evaluate the accuracy of determining the statistical characteristics of kinematic parameters and the reliability of W, several series of calculations were performed using 100, 500, 1000 and 5000 implementations. The results were then compared. The calculation results are shown in Tables 8 and 9.

Table 8. Probabilistic characteristics of the reaction of a brick building with a different number of implementations (repeatability 1 time in 475 years).

Parameter	100	500	1000	2000	5000
Average displacement value, cm	2.10	2.73	2.75	2.75	2.787
Standard of movement, cm	0.55	0.74	0.75	0.75	0.774
Average speed value, cm/s	17.90	19.95	19.92	19.97	20.105
Speed standard, cm/s	3.26	3.65	3.73	3.71	3.7378
Reliability value W	0.97	0.84	0.84	0.84	0.827
Failure rate Q	0.03	0.16	0.16	0.16	0.173

Table 9. Probabilistic characteristics of the reaction of a brick building with a different number of implementations (repeatability 1 time in 2475 years).

Parameter	100	500	1000	2000	5000
Average displacement value, cm	4.99	4.76	4.76	4.76	4.775
Standard of movement, cm	1.17	1.17	1.24	1.24	1.277
Average speed value, cm/s	20.50	20.48	20.58	20.56	20.550
Speed standard, cm/s	3.42	3.87	3.99	3.80	3.859
Reliability value W	0.06	0.13	0.17	0.16	0.153
Failure rate Q	0.94	0.87	0.83	0.84	0.847

An analysis of Tables 8 and 9 shows that with 500 or more implementations, the values of the average values and standards of movement and speed practically do not change. The values of reliability W and failure Q are stabilized with the number of implementations from 1000 and above. The first decimal place stabilizes already at 1000 implementations. Monte Carlo schemes with 5000 implementations can only be recommended for ultra-precise calculations of reliability and failure values.

The obtained values of Q_{475} (Table 8) and Q_{2475} (Table 9) correspond to two earthquake recurrence maps with different PGA. Let us estimate the overall probability of failure, considering Q_{475} and Q_{2475}. According to the probability addition theorem (Q_{475} or Q_{2475} failure event occurrence scheme) and taking into account the independence of these events, we can write

$$Q_O = Q_{475} + Q_{2475} - Q_{475} * Q_{2475} \quad (4)$$

Substituting the values from Tables 8 and 9, $Q_O = 0.173 + 0.843 - 0.173 * 0.843 = 0.870$. The overall probability of failure is very high. A building with load-bearing brick walls can be considered earthquake-resistant (reliability $W_O = 1 - Q_O = 0.130$). The same thing happens after multiplying the reliability values $W_O = W_{475} \times W_{2475} = 0.130$, assuming these events are independent. Thus, the overall reliability of a brick building is equal to or less than the reliability value with a rarer seismic impact. This reliability value is a quantitative measure of a building's earthquake resistance.

Note that this method of determining reliability or failure is based on expensive experimental studies, two earthquake recurrence maps, methods for solving nonlinear differential equations, and the use of expensive mathematical systems. The experimental diagrams of deformation and dissipative characteristics of the building also take into account

the aging of building structures, the appearance of micro-damage from weak earthquakes and other dynamic loads.

Dissipative and rigid characteristics have a probabilistic character. Therefore, in Equation (1), we will assume the coefficient of inelastic resistance to be a normally distributed random variable with a given mathematical expectation value of m_μ and an RMS value of σ_μ.

Tables 10 and 11 show the results of calculating the probabilistic characteristics of the kinematic parameters of the dynamic model and the magnitude of reliability, as well as failure.

Table 10. Probabilistic characteristics of the reaction (repeatability one time in 475 years).

Parameter	σ_μ = 1.2	σ_μ = 5.0	σ_μ = 10.0
Average displacement value, cm	2.779	2.781	2.784
Displacement standard, cm	0.769	0.765	0.769
Average velocity value, cm/s	20.183	20.199	20.192
Speed standard, cm/s	3.790	3.788	3.802
Reliability value W	0.834	0.826	0.830
Failure rate Q	0.176	0.174	0.170

Table 11. Probabilistic characteristics of the reaction (repeatability one time in 2475 years).

Parameter	σ_μ = 1.2	σ_μ = 5.0	σ_μ = 10.0
Average displacement value, cm	2.953	4.776	4.761
Standard of movement, cm	0.996	1.265	1.241
Average speed value, cm/s	20.247	20.567	20.780
Speed standard, cm/s	3.817	3.720	3.982
Reliability value W	0.150	0.154	0.164
Failure rate Q	0.850	0.846	0.836

The analysis of Tables 10 and 11 shows that the influence of statistical variability of the coefficient of inelastic resistance certainly takes place. When the standard (RMS value) of the μ coefficient changes by a factor of 8, the reliability values change by several percent. But this impact is not significant.

Section 2.2 shows the results of certification of 1609 buildings with load-bearing brick walls. It was found that out of 1609 buildings, 1594 were not earthquake-resistant. The probability of failure of such buildings is very significant—0.9907. Such conclusions were made, in fact, at the expert level. The year of the houses' design and construction, the experience of such buildings in severe earthquakes, the presence of tectonic faults in the vicinity, spatial planning and structural solutions, the category of soil by seismic properties, and the presence of antiseismic measures were taken into account. A brief conclusion was made about the condition of structures and seismic vulnerability.

We believe that the application of experimental data with subsequent analysis of building reliability leads to more accurate estimates of the reliability of brick buildings with load-bearing walls. But in general, the estimates of earthquake resistance are confirmed.

Let us assume that the conditional probabilities of failures based on the certification results are taken from Section 2.2. A scheme or earthquake with a repeatability of one every 475 years, or one every 2475 years, is being implemented. The service life of the building has been adopted for 50 years according to regulatory documents. The last row of Table 12 shows the total failure probabilities for all brick buildings $Q_{475} = 0.1040$ and $Q_{2475} = 0.00208$.

Table 12. Failure Characteristics Taking into Account Repeatability.

No.	Constructive Solution	Failure Rate for Brick Buildings	Failure Rates with Repeatability of 475 Years	The Failure Rate for Repeatability Is 2475 Years	The Amount of Damage in % of the Original Cost of U
1	Brick	0.9907	0.10422	0.01832	100

Then, the total value of $Q_K = Q_9\ P_{475} + Q_9\ P_{2475}$.

Then, the value of the overall reliability for a group of brick residential buildings

$$\mathbf{W_k} = 1 - 0.10422 - 0.01832 = 0.87746 \quad (5)$$

The obtained value of the overall reliability $\mathbf{W_k}$ is an objective quantitative characteristic of the condition of a group of brick multi-story residential buildings in Almaty, taking into account the repeatability of seismic effects. Further actions may increase or decrease the specified amount $\mathbf{W_k}$.

The question arises whether the value is $\mathbf{W_k}$ significant or not. It is more correct to say here that this is an objective value; a posteriori assessment of the earthquake resistance of brick buildings, taking into account their actual condition and the seismic events that took place. Brick houses were built mainly in the 1950s and 1960s, i.e., they were 50, 60, 70 years old, and during these periods they were normally operated and there were no strong earthquakes. Therefore, the value $\mathbf{W_k}$ of (3) seems quite logical. If there is a strong seismic event tomorrow, the $\mathbf{W_k}$ magnitude will change and approach the values of the a priori reliability of brick buildings. It is obvious that the measure of earthquake resistance of buildings is an earthquake (seismic event) of calculated intensity, which will give a final a posteriori estimate of the reliability of brick buildings.

Using the data in Table 4 and the previous calculations, it is possible to evaluate the seismic risk of R.

Using the classical definition of risk as the product of the probability of failure Q by the value of the loss function according to Khakimov [22] and Raiser [23]

$$R = QU \quad (6)$$

where U is the damage caused by the failure in fractions of the original cost of the object.

It is clear that classical approaches to assessing the reliability of buildings as a probability of not achieving a given degree of damage are also possible to implement according to Lomnitz et al. [24], Jinchvelashvili [25], Aizenberg [26,27], Koff et al. [28], Tsipenyuk [29], Naprtvaridze [30], Lapin [31], Liu Xiao-Xiao [32], Fathi-Fazi [33], Bunea [34], Naveed [35], and Hare [36]. They are supposed to be used in the continuation of research.

5. Discussion

An a priori assessment of reliability and risk is performed based on the behavior of the first building in an earthquake, using previous information about the behavior of the building in an earthquake and selecting an adequate seismic impact. During the a priori assessment, the probability of an impact of a given intensity was not directly taken into account, since the seismic zoning map, depending on the recurrence of the impact, was not considered reliable.

During the a posteriori assessment, the specified map was already considered reliable and reliability (earthquake resistance) was assessed at the expert level, taking into account previous information about the behavior of such buildings.

These reliability and risk assessments generally do not have to match and can be used to make management decisions.

A priori and a posteriori reliability and risk assessments show that buildings with load-bearing brick walls are not earthquake-resistant. These houses were designed according to the building codes of the 1960s. Therefore, the level of seismic loads is underestimated for them. In principle, the methods of reinforcing buildings with load-bearing brick walls are well known.

We will evaluate the impact of the total reinforcement of such buildings on the earthquake resistance and reliability of the housing stock of Almaty.

Example 1: How will the total probability of trouble-free operation change W_k if all masonry buildings of the 275 and 308 series (667 units) are fully reinforced?

The probability of failure for the group of buildings of the specified series will be equal to 0. The total number of seismically resistant buildings will be equal to 682 units. Then, the failure value for the group of all buildings will be equal to 0.576 (Table 13). Then, according to the results of Table 10 $Q_{475} = 0.061$, $Q_{2475} = 0.0012$.

Table 13. Characteristics of the probability of failure Q for brick buildings.

No.	Constructive Solution	Failure Rate for Brick Buildings	Failure Rates with Repeatability of 475 Years	The Failure Rate for Repeatability Is 2475 Years
1	Brick	0.576	0.0605	0.0012

Then, the overall reliability taking into account repeatability is equal to

$$W_k = 0.9383$$

However, it must be borne in mind that, over the service life of a structure, a design-level seismic event will inevitably cause unacceptable damage to a brick masonry building, approximately of the fourth degree. It should always be remembered that earthquake recurrence is a rather rough measure of seismic action. Its use is meaningful only for certain administrative and managerial decisions.

For the assessment of seismic resistance, one should always rely on the results of experimental studies of specific building types, followed by the evaluation of failure-free performance probability using a seismic action model, which in general represents a non-stationary stochastic process.

The change in overall reliability, taking into account repeatability, will be almost 5%, which is quite significant. The same thing without taking into account the repeatability of almost 42%. This is very significant. The reinforcement of masonry buildings of series 275 and 308 will significantly increase the reliability of the group of masonry buildings and the seismic safety of the population.

Based on the materials of this study, brick masonry buildings exhibit a high degree of seismic vulnerability. The obtained results fully correspond to the analyses of the consequences of strong past earthquakes, as evidenced by the research of Ghimire [37]. Significant damage to brick and masonry buildings was observed as early as 1931 during the earthquake in New Zealand. In the earthquake in Zagreb (Croatia), brick masonry buildings sustained severe damage at acceleration values of 0.16 g. During the strongest earthquakes in Turkey (2023) and Myanmar (2024), not only significant damage to brick walls occurred, but also considerable human casualties.

According to the analysis of the 2015 Nepal earthquake in the study by Ghimire [37], vulnerability curves were constructed, indicating that at an acceleration of 0.27 g, 50% of brick and masonry buildings would be destroyed or severely damaged. The above accelerations correspond to an earthquake intensity of 8 on the MSK-64 (K) seismic scale. However, it was noted that the duration of seismic action was considerable. Therefore, the

seismic intensity can be roughly estimated as corresponding to a nine-point level. Consequently, it can be concluded that our results approximately correspond to the above-mentioned earthquake.

Brick buildings of the 308 series in Almaty city are located on soils of category I in terms of seismic properties, and are usually made in three- and four-story versions. Brick walls have the same thickness, and all buildings have approximately the same dynamic characteristics (oscillation period). A three-story building has been tested. The building received the fourth degree of damage on the scale of seismic intensity MSK-64(K), which corresponds to a 9-point intensity. The results of the dynamic tests correspond to the seismic explosion tests in Medeo (Section 2.3). The research results are quite representative.

6. Conclusions

Planning and structural solutions do not meet the requirements of the current earthquake-resistant construction standards, are designed without antiseismic measures and are earthquake-prone. Residential buildings with load-bearing brick walls with a height of two and three floors and wooden beams should be demolished as a matter of priority. Strengthening the buildings of this group is possible, but not economically rational. Residential buildings with load-bearing brick walls of three and four floors with reinforced concrete floors, if necessary, should be subject to reinforcement and reconstruction. The use of buildings with load-bearing brick walls in areas with a seismic intensity of 9 is not permissible. In areas with a seismic intensity of 8, such buildings may be used only in exceptional cases, with strict compliance with regulatory requirements for the strength of brick masonry and the normal bond between bricks and mortar.

The buildings were built in the late 1950s and early 1960s. The physical deterioration of buildings is almost equivalent to 100%. Reconstruction and reinforcement of brick buildings is possible in principle, but involves significant costs. In addition, it is necessary to evict the tenants, which is not possible. A separate problem is the replacement and modernization of communications and water supply networks. Therefore, the modernization of buildings does not seem economically profitable. In addition, the buildings are located in the central part of the city, where land plots are very expensive. Therefore, the city authorities must make a final decision on the demolition of buildings. The desire of the municipality of the city to conduct research on the need for the above-mentioned measures is one of the reasons for setting up this study.

If it is necessary to strengthen the load-bearing brick walls, the following modern methods can be used:

1. Reinforcement of the building by means of belts/puffs made of carbon tapes/fabrics.
2. Reinforcement of buildings with carbon grids based on polymer cement mortar.
3. Reinforcement of damaged walls by gluing vertical elements along the considered area on one side of the wall using carbon tapes, fabric, mesh and carbon fiber lamellas (laminates).
4. Reinforcement of brick walls with reinforced plaster on both sides.

Probabilistic estimates of the reliability of masonry buildings have been obtained based on the results of certification, both taking into account the recurrence of earthquakes and without it. The certification results indicate insufficient earthquake resistance of buildings with load-bearing brick walls, 99% of which were not earthquake-resistant. The seismic resistance of such buildings can be ensured only with the use of various types of seismic insulation systems or the massive use of energy-absorbing elements. KazNIISA JSC has extensive experience in the application of various types of seismic isolation systems (Yerzhanov and Lapin [38], Lapin and Yerzhanov [39], Dyrda et al. [40], Sreeman and Roy [41]).

Author Contributions: Conceptualization: V.L. and Y.A.; methodology, V.L. and Y.A.; validation, Y.A., V.L., Z.O., Z.M. and S.A.; format analysis, Y.A., V.L., Z.M., N.M. and S.A.; investigation, Y.A., V.L., Z.O., N.M. and S.A.; resources, Y.A.; data curation, Y.A. and S.A.; writing—original draft preparation, Y.A. and V.L.; writing—review and editing, Y.A. and N.M.; visualization, Y.A., V.L. and S.A.; supervision, V.L.; project administration, N.M.; funding acquisition, V.L. and Z.M. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the Science Committee of the Ministry of Science and Higher Education of the Republic of Kazakhstan (program No. BR24992763).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Data is available upon reasonable request.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Conflicts of Interest: Authors Yerken Aldakhov, Zhassulan Omarov, Nurakhmet Makish and Serik Aldakhov were employed by the company Kazakh Research and Design Institute of Construction and Architecture JSC. Author Zhangazy Moldamuratov was employed by the company International Educational Corporation. Author Vladimir Lapin was employed by the company The National Scientific Center for Seismological Observations and Research LLP. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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