Chapter

The Strength of Masonry Based on the Deformation Characteristics of Its Components

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Abstract

The chapter presents a new approach to determining the strength of masonry reinforced with transverse meshes in mortar joints. The method consists of using the values of the modulus of elasticity and limiting deformations of the stone material, mortar for joints, and both steel and composite reinforcements. An analytical notation is proposed that integrally takes into account the characteristics of the initial materials. The results of physical tests of centrally loaded masonry pillars reinforced with steel and composite meshes are given. To test the masonry, widely used materials were used: solid brick and cement-sand mortar. The values of the bearing capacity, deformations, and internal stresses of the masonry are obtained. It is determined that the stresses in the reinforcing bars of the meshes are unevenly distributed in the horizontal plane of the mortar joint and amount to 20–37% of the design resistance of the mesh material. The strength of masonry reinforced with composite meshes is 65–75% of steel of the same cross section. It is shown that there is a good convergence of test results with the presented analytical dependence.

Keywords: masonry, reinforcement, deformation, strength, testing, modulus of elasticity, composite reinforcement, steel reinforcement, basalt bars, reinforcement mesh, design, the percentage of reinforcement

1. Introduction

In construction practice, the method of increasing the bearing capacity of masonry using mesh reinforcement in horizontal mortar joints when working in central compression is quite widespread. Recently, along with a metal mesh, meshes made of composite reinforcement (fiberglass, basalt plastic, and others) have been used. The physical essence of the method is to contain the transverse deformations of the masonry and transfer the part of the forces to the reinforcing bars located in the horizontal mortar joints. The calculation of such masonry is regulated by the set of rules SP 15.13330.2020 “Stone and reinforced masonry structures.” According to this standard, masonry is considered a homogeneous structure, while the given physical and mechanical characteristics of its components (brick and mortar) are used. The same concept underlies Eurocode 6. The second approach is to represent masonry as a complex composite structure with materials of different modulus having significantly different characteristics.

In any case, like concrete, masonry is reinforced to give a brittle material—stone, which has high compressive strength and greater tensile strength. Reinforcement makes it possible to increase the strength of the stone by preventing lateral expansion caused by a force applied perpendicular to the mesh. According to the standard (set of rules) SP 15.13330.2020, the strength of reinforced masonry doubles.

In fact, reinforced masonry is a composite material consisting of the main mass in the form of stone, interlayers of mortar, and rarely ordered inclusions in the form of steel or composite rods. The use of materials with different characteristics requires the creation of calculation methods that take into account their initial characteristics. Appropriate diagrams of material deformation are needed—in tension, shear, and compression.

According to the works of V.A. Ivanov, L.I. Vucin, M.V. Skobeeva, A.I. Kibets, Yu. I. Kibets [1, 2] in a material where the binder matrix has numerous differently directed more rigid inclusions, it is difficult to establish the actual distribution of strains and stresses.

In the work of S. Babaeidarabad [3], it is established that in order to increase the strength when strengthening the masonry, there is a ratio of parameters, in particular, reinforcement coefficients. However, in this work, only external reinforcement is discussed.

The continuum model of masonry without reinforcement has its place, especially when analyzing the nonlinear behavior of a structure. Models, according to A.H. Akhaveissy, can take into account microcracks in masonry, which lead to softening and destruction [4].

In any case, researchers proceed from the definition of the parameters that make up the masonry, using them for either discrete or continual model building. For masonry, predictive analytical dependencies can be obtained to build nonlinear graphs of masonry work. In the work of T.C. Nwofor [5], obtained nonlinear tension curves with characteristic points highlighted the tension curves with a stress level of 0.4 from the breaking load, which corresponds to the limit of the near linear region.

Works of V.A. Ivanov, L.I. Vucin, M.V. Skobeeva, A.I. Kibets, Yu. I. Kibets [1, 2] showed that in this case, brickwork can be modeled as a continuum multimodular medium, the properties of which depend on the type of stress-strain state and the current level of damage to the material. To calculate masonry, a simplified model can be applied that takes into account the deformability of joints, the strength of brick, and mortar in tension and shear, as well as the contact interaction of masonry fragments. Each brick is divided into a number of segments (blocks). The brick material is assumed to be isotropic and ideally elastic. The destruction of masonry along horizontal and vertical seams and along sections of bricks that bind vertical seams is considered. At the initial stage (before destruction), when analyzing the interaction of two blocks of one brick, the contact pressure components are calculated from the conditions of rigid gluing. The stresses in the joints of the masonry are determined through the deformation of the binder.

With a contact pressure component $q_n > 0$, the tensile and shear strength criteria are checked in succession. With compression ($q_n < 0$), only the fulfillment of the criterion for shift is analyzed. If at least one of the strength criteria is violated, it is considered that local destruction of the brickwork has occurred, and in the future, the contact interaction at this point is modeled using the friction algorithm.

Recently, more and more two described methods penetrate into each other and are used in a complex, as can be seen from a number of works.

Based on the theory of resistance of anisotropic materials, A.B. Antakov and B.S. Sokolov [6, 7] obtained masonry deformation diagrams under compression. Taking into account a large number of tests, the stages of the stress state were described. The values of the tear, shear, and crush forces were determined using the strength characteristics of the masonry: tensile strength R_t, shear R_{sh}, compression R, and geometric parameters—the areas of the corresponding surfaces A_t, A_{sh}, A_{ef}.

Techniques for modeling masonry by the finite element method are being developed with the introduction of a number of specific physical and mechanical parameters. G.G. Kashevarova [8] introduced criteria into the model that take into account orthotropy, strength, strain softening, and layer shear coefficients while ensuring a minimum level of resistance. The criteria for the strength of individual components are accepted: brick and mortar in tension and shear, and strength of contact between brick and mortar in a horizontal joint. The angles of inclination of the load and the ratio of types of load that affect the strength of the masonry are determined.

A deep analysis of the influence of masonry components, including the location of brick faces, in the form of finite element models with the inclusion of empirical data, was carried out by V.V. Pangaev [9]. This makes it possible to select the composition and system of bonding masonry while taking into account the different nature of deformation and destruction of typical elements of masonry—bonded and spoon rows, and vertical and horizontal mortar joints. It is shown that a small physical sample of five spoon rows of bricks is sufficient to obtain reliable data on the stress-strain state and can be accepted as a masonry element.

The complex model of O.V. Kabantsev [10] combines discreteness and has elements in the form of individual bricks and layers of mortar, and continuity, a material with properties that take into account the contact interaction of the constituent components of the masonry.

Most modern authors use piecewise homogeneous physically nonlinear functions of individual components to build models [1, 11–15].

Recently, the use of reinforcement in the form of meshes of composite rods in horizontal joints of masonry has been growing [16–23]. This increases the heat transfer resistance of the outer walls, increases the corrosion resistance of reinforcement, and, in some cases, reduces the cost of reinforcement. However, the use of composite reinforcement in masonry is constrained by the lack of calculation methods and experimental data.

2. Materials and research methods

The traditional method for calculating masonry reinforced with meshes, given in the design standards (SP 15.13330.2020), is based on empirical dependencies obtained by L.I. Onishchik [24, 25]. On the whole, it has justified itself for several decades of application for steel meshes, but it does not take into account the peculiarities of the physico-mechanical properties of composites at all. Composite, in particular, basalt-plastic-reinforced, as part of the structure, manifests itself as a very strong material in tension, having a tensile strength of at least R_f = 1000 MPa. However, the elastic modulus, in this case, is only E_f = 50,000 MPa [7]. For steel, this ratio is different (Rs = 400 MPa, Es = 200,000 MPa).

Transverse reinforcement in the form of meshes is used to increase the bearing capacity of the masonry in compression. According to current standards, the amount of reinforcement in the masonry is determined by the percentage of reinforcement by volume:

$$\mu = \frac{V_a}{V_k} 100, \tag{1}$$

where $V_a = (C_1 + C_2)A_{st}$—reinforcement volume, $V_k = C_1C_2S$—masonry volume, S—height spacing of grids.

The minimum percentage of reinforcement is assumed to be μ_{min} = 0.1% and maximum μ_{max} = 1% .

The tensile strength of masonry with mesh reinforcement is determined by the formula:

$$R_{sku} = kR + \frac{2R_{sn}\mu}{100}, \tag{2}$$

where R_{sn} is the normative tensile strength of reinforcement; R is the tensile strength of the masonry; k is a coefficient that takes into account the type of stone.

For reinforcement made of class B500 steel, R_{sn} is taken with a reduction factor of working conditions of 0.6. Therefore, it is considered that the limit of resistance is not reached in the reinforcement during the destruction of the masonry. However, there are practically no experimental data confirming this norm.

The fracture mechanics of masonry assumes the occurrence of critical tensile stresses in the transverse direction of the vertical element under the influence of the Poisson effect at a stress level of 0.4–0.7 of Ru (tensile strength) depending on the ratio of strength and modulus of elasticity of stone and mortar. More often, vertical power cracks occur above vertical mortar joints, less often along the stone, when the mortar bed is not made evenly enough. The destruction occurs from the rupture of stones, and the masonry is divided into separate columns, a multiple of half the brick in size.

To increase strength and reduce deformations in the transverse direction of the masonry, reinforcement with metal or composites in the form of meshes in horizontal mortar joints is used [1, 2, 14, 16–23]. Part of the stress is transferred to the reinforcement. Cracking, in this case, is not so intense, and cracks appear at stress levels above 0.7 Ru. The division of masonry into separate columns does not occur. Therefore, the level of stress in the reinforcement is important for the calculations of reinforced masonry.

To consider the stresses in the volume of masonry, it is necessary to connect them with Gook-law (**Figure 1**):

$$\varepsilon_x = \frac{1}{E}\left[\sigma_x - \nu\left(\sigma_y + \sigma_z\right)\right] \tag{3}$$

The Poisson ratio for masonry used here is not uniquely defined and depends on the type of stone and mortar.

The design resistance of the masonry is determined by the stage of formation of the first cracks that cross no more than two rows [24]. In this regard, let us consider the cracking force in the mortar joint N_{crc} (**Figure 2**).

The mortar joint and the rows of bricks adjacent to it resist stretching together, provided that the necessary adhesion is provided. The crack initiation stress

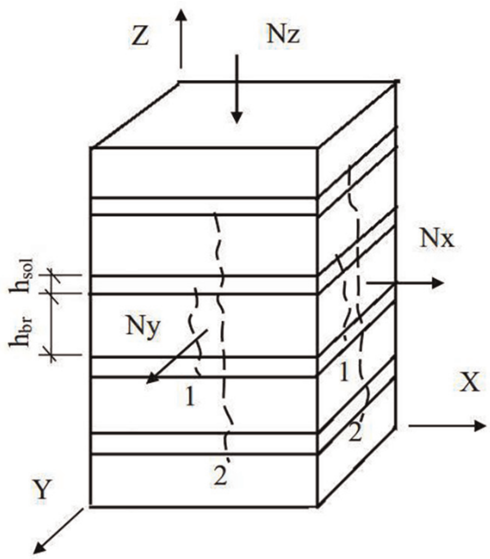


Figure 1.
Masonry element with force distribution. 1: initial stage of cracking, 2: destructive cracks.

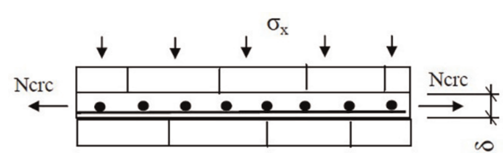


Figure 2.
Scheme of forces in the masonry layer.

corresponds to the tensile strength of the masonry over the tied joint Rt. Taking into account that usually the deformations of the mortar joint grow faster than the stone, we attribute the stress Rt only to the sum of four layers of the mortar joint, since the effect of transverse reinforcement is manifested when the grids are located at least after four rows.

The force N*crc* is resisted by the force N*s* that occurs in the reinforcement. As a result:

$$N^I = N_{crc} - N_s \tag{4}$$

Writing (4) through the mechanical parameters of materials,

$$N^I = R_t A_j - \varepsilon_s E_s A_s, \tag{5}$$

where A_j is the cross-sectional area of four mortar joints in the vertical plane; *εs*—deformation of the reinforcement corresponding to the deformation of the formation of cracks in the mortar joint is accepted $\varepsilon_s = \varepsilon_u$ (maximum for mortar and fine-grained concrete 1.5×10^{-4}). For composite reinforcement, the second term of the expression changes to $\varepsilon_f E_f A_f$; A_s,A_f—total cross-sectional area of reinforcement in one direction within four rows of masonry (steel and composite).

The structure of formula (6), given in SP 15.13330.2020, assumes a linear increase in the strength of unreinforced masonry R with an increase in the volumetric reinforcement coefficient μ, while a restriction is imposed $R_{sk} \leq 2R$. Simple logical reasoning leads to the fact that the strength of the reinforced masonry should increase asymptotically and not end abruptly after a linear steep takeoff.

$$R_{sk} = R + \frac{p\mu R_s}{100} \tag{6}$$

Unlike steel reinforcement used in masonry and having a physical or possibly conditional yield strength, composite reinforcement does not have such a concept, as follows from the available sources, for example, the set of rules for strengthening with composite materials SP 164.1325800.2014 "Reinforcement of reinforced concrete structures with composite materials design rules." To calculate the longitudinal reinforcement, in this case, a number of coefficients of operating conditions are introduced to the temporary resistance. Bearing in mind, the determining value for the resistance of the material of low modulus of elasticity, expression (7) can be written as follows:

$$R_{sk} = R + \frac{p\mu\varepsilon_{s,u}E_s}{100} \tag{7}$$

However, as practice shows, the stresses in steel reinforcement during the formation of cracks in the masonry, corresponding to the onset of the limit state, are still far from the design resistance of the reinforcement and its ultimate tension.

The relative deformations in (7) must be replaced by the ultimate deformations of the mortar joint in tension ε_u. Formulas (6) and (7) are comparative in nature, that is, show how much the strength of unreinforced masonry increases when it is reinforced. Therefore, this increase can be represented as a ratio of the initial bearing capacity of the mortar joint in tension to the increased bearing capacity due to the tensile resistance of the reinforcement.

In (5), the effect of reinforcement is infinite, as in the formula of the set of rules. To compensate for this shortcoming, it is proposed to introduce a restriction that would lead to an asymptote at maximum reinforcement. To do this, the decaying increase of the second term in terms of the natural logarithm function is introduced into expression (5), as the most common in analytics, and decomposed into a rapidly convergent series. Based on the general properties of the logarithm function, an argument of the form (1 + x) is introduced to exclude negative and physically nonexistent values of the function. Expression (5), taking into account the limitation on the tensile strength of the mortar joint, takes the form:

$$N^I = R_t A_j - \varepsilon_u E_s \ln\left(1 + A_s\right) \tag{8}$$

As a result, to calculate the bearing capacity of masonry reinforced with composite meshes, A.N. Plotnikov proposed a formula that takes into account the increase in the strength of unreinforced masonry due to the elastic resistance of composite reinforcement in the joints:

$$R_{sk} = R\left(\frac{R_t A_j}{R_t A_j - \varepsilon_u E_s \, ln\left(1 + A_s\right)}\right) \tag{9}$$

An analysis of the obtained function R_{sk} depending on A_s showed that it has an increasing and asymptotic character (**Figure 3**), starting from zero values of the cross-sectional area of the reinforcement. **Figure 3** (Graph 1) shows the dependence according to (9) of the increase in the bearing capacity of solid brick masonry on the

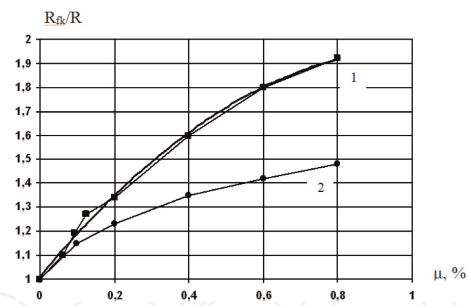


Figure 3.
Dependence of the R_{sk}/R ratio on the percentage of masonry reinforcement (1) according to the formula (8) with a logarithmic approximation, (2) according to the formula (9).

traditionally determined percentage of reinforcement. The maximum possible increase in the bearing capacity is two times.

Formula (9) can also be applied to masonry reinforced with composite rods connected into meshes, taking in the value of the elastic modulus of the composite Ef.

At the present stage of the use of composite meshes in masonry, one has to talk about a number of design limitations in determining the bearing capacity. The question of the adhesion of the composite in the body of the cement-sand mortar and, accordingly, the anchoring of the reinforcement remains unexplored. The currently used methods of connecting rods by gluing them with molten polyethylene do not give great strength. According to manufacturers, the average breaking force of the connection of rods with a diameter of 3.2 mm is Nsh = 338 N. For a masonry element with a cross section of 510 × 510 mm and a mesh of reinforcing mesh 50 × 50 mm, one rod resists shear in each of four directions from the center no more than four connections.

The modulus of elasticity of polyethylene is only about E = 300 MPa, which is significantly lower than the corresponding values of the composite rod and mortar joint. In this regard, the connections of the rods in the nodes are significantly pliable, which is reflected in the tensile strength of the reinforcement. The value of compliance can be estimated from the proportion of the location of polyethylene on the length of the rod. The length of the polyethylene section is 10 mm with a grid cell of 50 × 50 mm; that is, connection with the solution of the seam has no more than 0.8 of the length of the rod.

Compliance is also characteristic of the contact of steel reinforcement with a seam solution. The rods have the maximum compliance value at the maximum percentage of reinforcement, because at the same time, maximum stresses develop in the seams. The function of this dependence is nonlinear; in order to achieve physically defined parameters, an increasing function of the type is proposed with the introduction of compliance $k = \cos^5 x$. It has limits at x = 0: k = 1, at x = 1: k = 0.5. It is proposed to use the traditional reinforcement factor $\mu \le 1$ expressed in radians as the function argument. As a result, we get:

$$R_{fk} = R\left(\frac{R_t A_j}{R_t A_j - \cos^5 \mu \varepsilon_u E_f \ln\left(1 + A_f\right)}\right) \tag{10}$$

Figure 3 (Graph 2) shows the dependence of the increase in masonry strength depending on the percentage of reinforcement, and the maximum increase is achieved by 1.5 times.

The analytical dependence was verified by testing samples of masonry reinforced with steel and composite meshes in the joints.

The dimensions of the samples in the section are 0.51 × 0.51 m. A sample with steel meshes was a prism with a height h = 1.34 m. Ceramic bricks of the M125 brand were used on a cement-sand mortar of the M100 brand; reinforcement was made with meshes of wire Ø4 Vr500 with a cell measuring 50 × 50 mm, laid horizontally every three rows of bricks (**Figure 4**).

Material parameters: ultimate strength of brick in bending R_{ben} = 2.6 MPa, in compression R = 13.4 MPa; cement-sand mortar grade M100 with cubic strength R = 10 MPa; reinforcing wire Ø4 Vr500 (As = 12.57 mm^2) normative tensile strength R_{sn} = 500 MPa, calculated—R_s = 415 MPa. The masonry was created immediately on the press plate.

To determine the physical and mechanical characteristics of the working reinforcement, tensile tests were carried out. For the purpose of carrying out subsequent measurements, calibration dependence was built for strain gauges.

To measure deformations and stresses in the rods as part of the structure, strain gauges with a base of 20 mm and a resistance of 100 Om were glued to them. The strain gauge glued to the rod was covered with sealant, and the wires were removed from the masonry. The sensors were located on two grids: above the ninth and fifteenth rows of masonry (**Figure 5**).

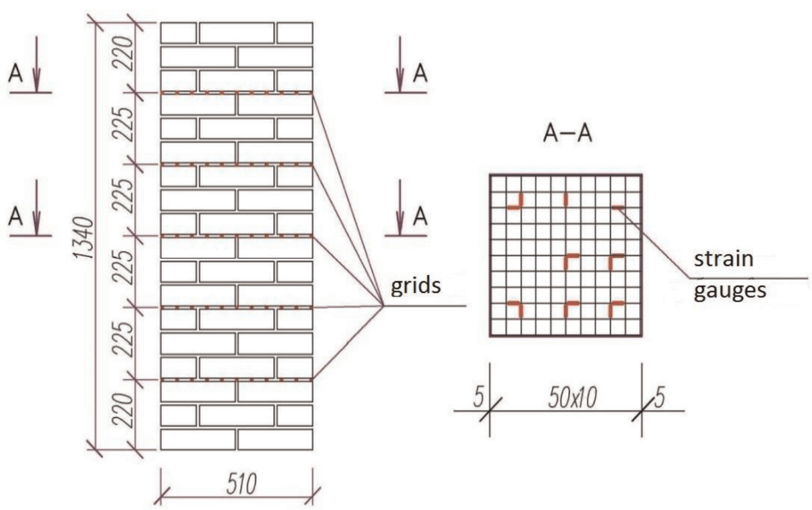


Figure 4.
Arrangement of reinforcing meshes and sensors.

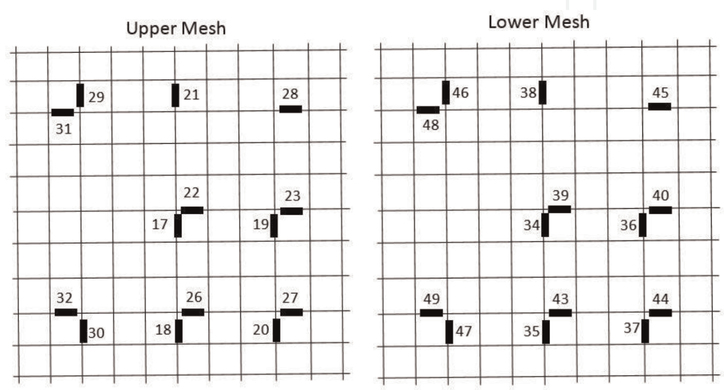


Figure 5.
Location of strain gauges on reinforcing masonry meshes.

During the test, the following measuring instruments were used:

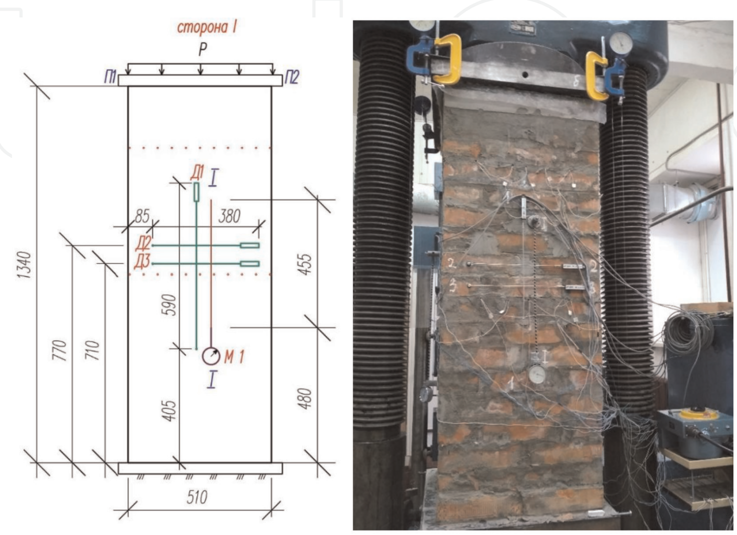
- deflection meters Aistov 6-PAO (P1–P4) with an accuracy of 0.01 mm (**Figure 6**) to assess the total vertical deformations of the masonry column;
- mechanical linear meters (M1–M4), with an accuracy of 0.01 mm for measuring longitudinal deformations of the masonry;
- electronic strain gauges DPL-10 with connection to the recorder "Terem-4.0" with an accuracy of 0.001 mm for measuring surface deformations of the masonry:
    a. D1, D4, D6, D9—for measuring the longitudinal deformations of the masonry (duplicating M1–M4);
    b. D2, D3, D5, D7, D8, D10—for measuring the transverse deformations of the masonry.

On the general views of the sample (**Figure 7**), the numbers of mechanical deflection meters for measuring vertical deformations, electrical strain gauges for transverse and longitudinal deformations of the masonry are indicated. AID-5 recording equipment was used.

The dimensions of the cross section of the masonry (510 mm) were sufficient to determine the deformations along the width of its section.

Loading was carried out in steps of 200 kN with central compression on a hydraulic press with a capacity of 5000 kN. At each stage, the load was kept for at least 10 minutes. Longitudinal strains were measured using mechanical gauges mounted on a base 455 mm high on all four sides; longitudinal and transverse deformations by electrotensometers with a base of 150 mm.

Comparison of the work of solid brick masonry reinforced with composite mesh with reinforcement with traditional steel mesh (Vr500 wire) was carried out on samples with dimensions of 380 × 380 × 600 mm with the same percentage of reinforcement.



a) b)

Figure 6.
Test stand. a—general scheme; b—general view.

Figure 7.
Placement of strain gauges on the sample surface.

3. Results and problems

The greatest interest in the tests carried out was the distribution of stresses in the reinforcing bars of the meshes over the cross section of the masonry. According to preliminary calibration graphs and data measured during loading from strain gauges on reinforcing meshes, the forces and stresses in the rods of the masonry mesh were determined. The stresses in the central and peripheral parts of the reinforcing mesh depending on the load are shown in **Figures 8** and **9**.

According to the test results, it was determined that the stresses in the reinforcing bars are 37% in the center of the masonry and 20% in the peripheral sections of the designed steel resistance. Stresses along the height of the sample are distributed unevenly. In the upper grids, the stresses in the center of the masonry section are 1.36

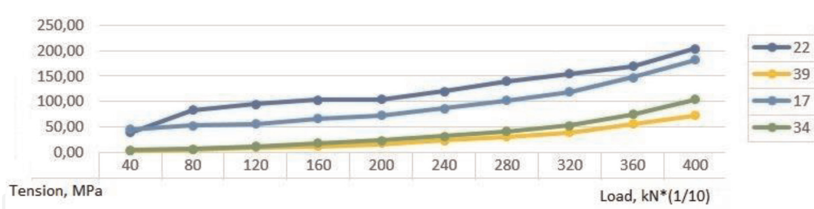


Figure 8.
Stresses in the central part of reinforcing meshes.

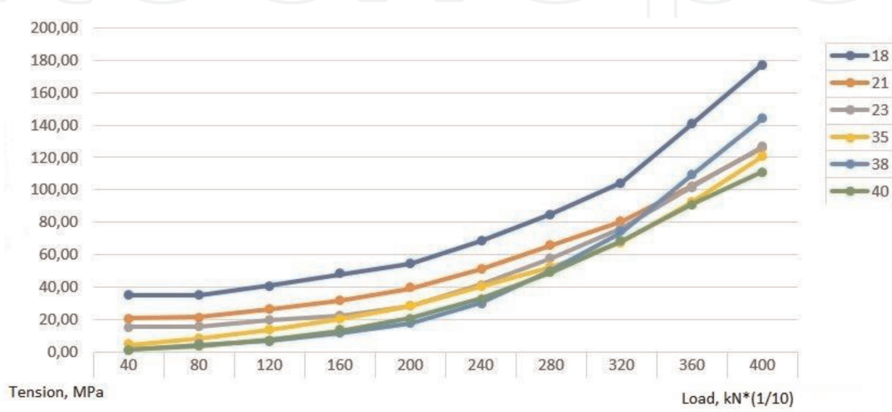


Figure 9.
Stresses in the peripheral part of reinforcing meshes.

times higher than the values of the lower grid. In the peripheral zones of the upper grids, the voltage is 1.33 times higher.

During the formation of cracks in the masonry, the maximum stresses in the rods were 92 MPa at the center of the section and 48 MPa at peripheral points at the corners of the masonry.

Up to a load of 2000 kN, the stresses in the rods increase linearly, above the stresses increase nonlinearly, while cracks in the masonry are not yet formed. This indicates the plastic nature of the work of the masonry; that is, there is a collapse of the mortar joint under the action of reinforcing bars and there is some movement of the stones relative to the mortar joints.

There is a margin of bearing capacity for tension of reinforcing bars. The norms specify the resistance of the bars as 0.6 Rsn. It is determined, according to the test, that this value is higher and is 0.72 Rsn.

Longitudinal strains were measured on four sides of the sample (**Figure 10**).

Longitudinal deformations are determined equally for all groups of sensors (mechanical and electronic).

The transverse deformations of the masonry in the reinforced and non-reinforced layers have a nonlinear nature of work, which indicates an increase in the intensity of cracks inside the volume of the masonry (**Figure 11**).

Theoretical values of the bearing capacity of unreinforced and reinforced samples of the considered sizes were $N_{ur} = 624$ kN,. and $N_u = 973$ kN respectively, an increase of 1.56 times. Strength limit of reinforced masonry $N_u = 1600$ kN

The load at which the destruction of the sample began was 4020 kN. The margin of bearing capacity is 2.5 times. This reserve can be attributed to a different technology for the manufacture of masonry in comparison with the stipulated norms and created in the laboratory. The design resistance of the reinforced masonry according to the formula (10) R_{sk} = 14.85 MPa. Four rows of masonry of the experimental sample are taken into account. Seams are accepted with a thickness of 1 cm. Eleven wire rods are located in one horizontal seam.

Numerical data: ε_u=1.5*10^{-4}—reinforcement deformation corresponding to the deformation of mortar joint crack formation (ultimate deformations); A_j —cross-sectional area of four mortar joints in the vertical plane, A_j = 51*1*4 = 204 cm^2; A_s is the total cross-sectional area of reinforcement in one direction within four rows of masonry, $A_s = 12{,}57 * 10^{-2} * 11 = 1{,}3827$ cm^2; R_t is the tensile strength of the masonry

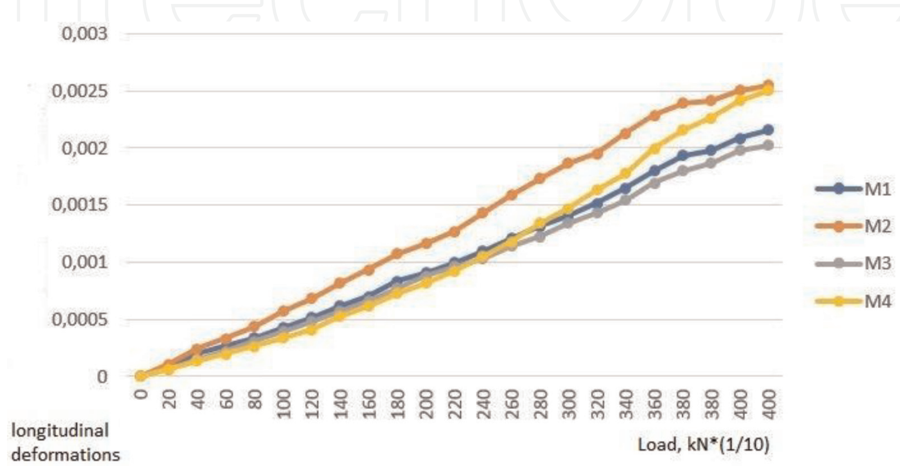


Figure 10.
Longitudinal deformations of masonry on four sides of the sample.

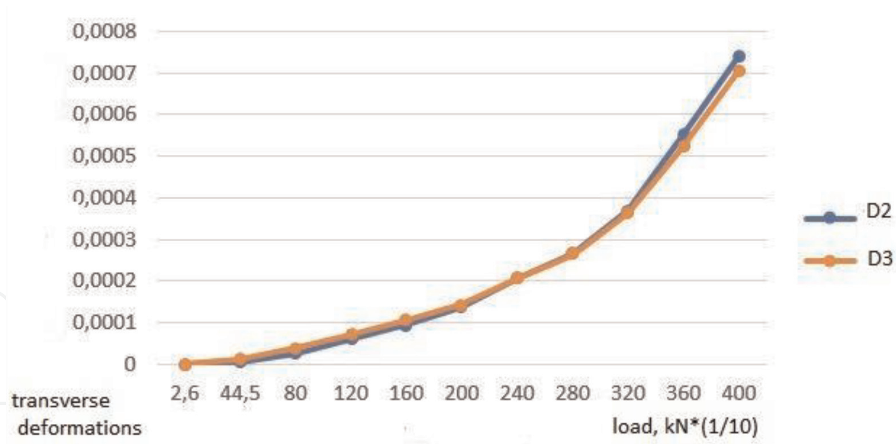


Figure 11.
Transverse masonry deformations. D3—closer to the reinforced layer, D2—to the layer without reinforcement.

along the tied seam, R_t = 0.16 MPa (according to SP 15.13330.2020); E_s = 200,000 MPa—elastic modulus of reinforcing steel.

In this case, N_u = 3862 kN, which is close to the ultimate test load of 4020 kN.

The relationship between stresses and strains was nonlinear. The initial deformation modulus of masonry with mesh reinforcement according to SP 15.13330.2020 (6.21) is taken to be the same as for unreinforced:

$$E_0 = \alpha R_u, \tag{11}$$

According to the results of the experiment, for masonry of solid ceramic bricks at α = 1000, E_0 = 15,450 MPa is taken. According to the results of measurements, E_0 = 10,666 MPa.

Poisson's ratio for the area of deformations in the first third of the increase in load

$$\nu = \frac{\mathcal{E}_x}{\mathcal{E}_y}, \tag{12}$$

where $\mathcal{E}_x$ and $\mathcal{E}_y$ are the relative transverse and longitudinal strains, respectively.

At a maximum load of 4020 kN, the deformation modulus E = 6925 MPa. A decrease in E as a result of the nonlinearity of the processes was noted by 1.6 times.

The increase in Poisson's ratio for reinforced masonry was insignificant and amounted to 0.23, compared with the standard ν = 0.25, by less than 10%.

The magnitude of the absolute vertical deformation of the sample was Δ_y = 4.17–4.58 mm. Relative vertical deformations $\varepsilon_y = 3.11{*}10^{-3}$ - $3.41{*}10^{-3}$.

The effect inherent in the reinforcement of the masonry with meshes in the mortar joints in the test proved to be quite complete. The pattern of masonry cracking changes, and no main cracks appear. Visible cracks occur in the brick in the layer above the mesh. A material with a high modulus of elasticity increases the resistance of mortar joints and masonry in general.

In the previous experiments [24, 25], at maximum loads, small fragments of brick and mortar peel off, which does not occur in unreinforced masonry. At the same time, stresses in the mesh rods reach the yield strength of steel before the failure of the masonry.

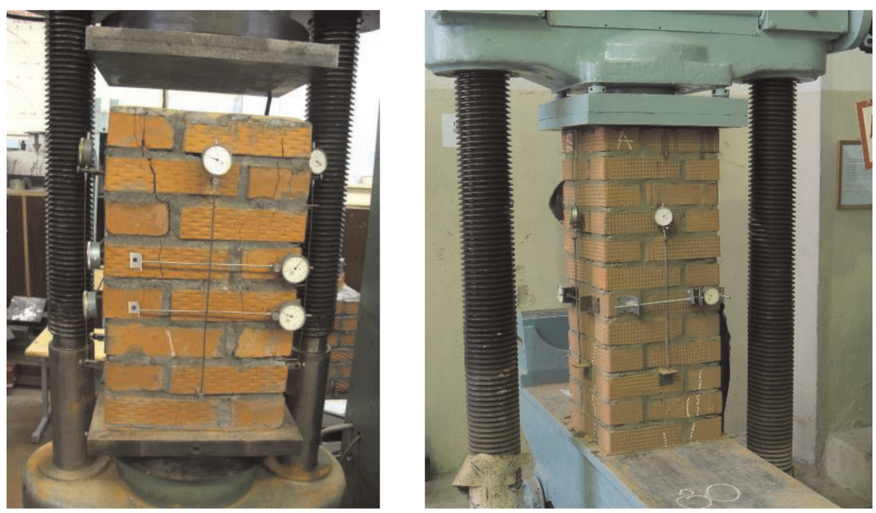
In masonry samples using composite meshes, this phenomenon is not observed or it is less pronounced. This is explained by the significantly lower value of the elastic modulus of the composite relative to steel.

Comparison of the work of solid brick masonry reinforced with composite mesh with reinforcement with traditional steel mesh (Vr500 wire) was carried out on samples with dimensions of 380 × 380 × 600 mm with the same percentage of reinforcement.

Comparison of numerical data obtained by formulas (9) and (10) was carried out with a number of experimental data. The test results are mentioned in the following studies: V.M. Pozdeev, N.P. Soloviev, A.V. Vinogradov, V.V. Nikolaev [22]; A.B. Antakov [23]; A.V. Granovsky, V.V. Galishnikova, E.I. Berestenko [21] (**Figure 12**).

The most relevant information on this experiment was obtained from a comparison of the transverse deformations of the masonry (**Figure 13**), determined near the mortar joint.

It has been found that the transverse deformations of masonry pillars reinforced with composite, in particular, basalt-plastic reinforcement (FRP) are 2.5 times higher



(a) (b)

Figure 12.
Prototypes with measures placed on them: (a) tests by V.M. Pozdeev and (b) tests of A.B. Antakov.

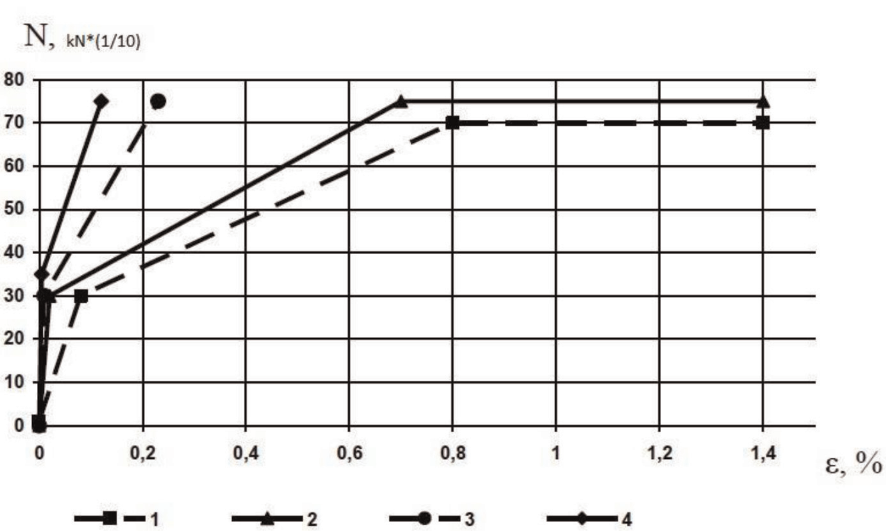


Figure 13.
Graph of transverse deformations of columns and reinforcement in the central cross section according to the tests of V.M. Pozdeev: (1) transverse deformations of columns with FRP; (2) transverse deformations of columns with steel reinforcement; (3) FRP elongation; and (4) elongation of steel reinforcement.

than steel ones. The tensile strength and cracking load are practically the same. Such results were obtained with a relatively small percentage of reinforcement –0.11% and a solution that did not gain full strength, with early cracking.

In relation to unreinforced masonry, reinforcement with composite reinforcement with different percentages of reinforcement according to the test results [23] in samples 380 × 380 × 1000 mm led to an increase in the bearing capacity and crack resistance by 30–33%. The intensity of reinforcement varied in the range of 0.062–0.422% (**Figure 14b**). In all series, the destruction of masonry with composite

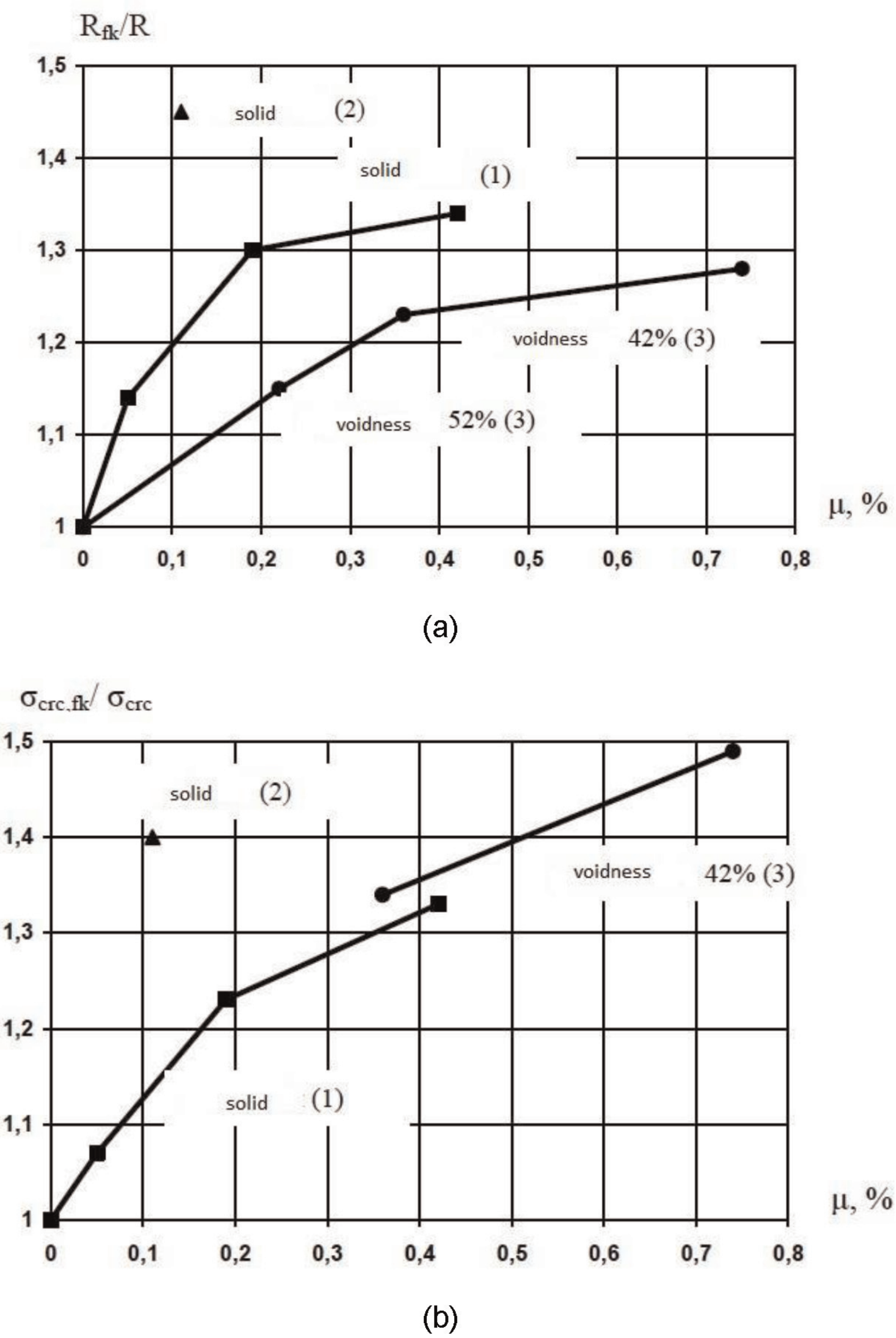


Figure 14.
Ratio dependence: (a) R_{fk}/R;)b) $\sigma_{crc,fk}/\sigma_{crc}$ of the percentage of reinforcement of masonry composite according to experimental data: (1) A.B. Antakov, (2) V.M. Pozdeev, (3) A.V. Granovsky.

reinforcement took place with a main vertical crack, which is not typical for reinforcement with steel meshes [24].

Tests conducted under the leadership of A.V. Granovsky [21], reinforced with composite reinforcement masonry made of ceramic stones with large voids and small sections (sample sizes 250 × 1030 × 1200 mm, 250 × 800 × 1350 mm), showed an increase in the bearing capacity relative to unreinforced ones by 1.2–1.3 times (**Figure 14a,b**).

All experimental data show lower strength values of composite-reinforced masonry compared with steel-reinforced masonry. The reason for this is the lower modulus of elasticity of the plastic, which does not help to contain the transverse deformations of the masonry. However, this disadvantage can be overcome by increasing the percentage of reinforcement by the composite. Another reason is the insufficient adhesion of the surface of plastic rods with the joint solution and the ductility of the nodal joints of composite meshes made in particular with polyethylene. In the case of using more rigid connecting materials, the bearing capacity of the masonry reinforced with a composite can be increased by 1.3 times, as follows from the graph (**Figure 13**). To increase the bearing capacity of such masonry, a thinner mortar joint with the same percentage of reinforcement as with steel meshes can be achieved due to small diameters and frequent placement of rods.

4. Conclusion

To determine the bearing capacity of masonry, primarily reinforced with steel and composite meshes in horizontal joints, it is necessary to use the characteristics of the source materials, using the obtained analytical dependence. It is recommended to use not the design resistance of the reinforcement material, but its modulus of elasticity and the value of ultimate deformation. It gives a convergence with experiments of 4%.

According to the test results of masonry with steel mesh, in comparison with the current standards, a 2.5 times greater strength was obtained. For composite reinforcement, there is no information in the norms.

Compared with unreinforced masonry, steel mesh reinforcement increases strength by a maximum of two times. According to the results of generalized tests, reinforcement with composite meshes increases the bearing capacity of masonry, depending on the types of composites used, by 1.3–1.5 times. From samples with steel reinforcement, this is 65—75%.

No main cracks were formed in the sample of masonry with steel reinforcement. The destruction occurred along small chips of brick and mortar. Stresses in the reinforcing bars of steel meshes did not reach the yield strength of steel and amounted to 37% in the center of the masonry and 20% along the perimeter. During the formation of cracks, they amounted to 92 MPa and 48 MPa, respectively.

The deformation modulus of the reinforced masonry during loading decreased by 1.6 times.

An increase in the bearing capacity of masonry reinforced with composite meshes is possible due to structural improvements, primarily by connecting rods at intersections.

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