



## Investigation of the stress-strain state of domical masonry vaults

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**Abstract.** The results of analytical study of stress-strain state of one of the varieties of vaults erected in various function buildings, i.e. domical masonry vaults covering Orthodox churches, are presented in this article. Due to the ongoing restoration of architectural monuments and religious buildings, as well as in connection with the construction of such facilities as Orthodox churches, the study of stress-strain state of masonry arches and vaults has become especially important in recent decades. Despite the rapid development of new building technologies in the construction of temples, use of these new technologies is shown to be limited and focused on the use of traditional materials. The current level of engineering of building structures with the use of software enables to significantly improve the calculation models and to obtain more reliable results, in particular for the structures under consideration. The domical vault considered herein is formed by wall extensions inclined along a given curve, converging in a horizontal crown under rectangular plan or in one point under square plan. The authors of the article have performed calculations of vaults in both linear and nonlinear formulations, with the results thereunder the forces and deformations of vaults have been analyzed. As a result profiles of vertical pressures and lateral pressures of the vault mounting groups, isopoles of principal stresses, vertical displacements, bending moments and normal forces were calculated.

**Keywords:** thrust system, arch, masonry, domical vault of roof, stress-strain state, linear and nonlinear calculations, analysis of stress-strain state evaluation results, failure factor

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## 1. INTRODUCTION

Masonry as a structural material for the construction of buildings and structures has been used in construction practice for many millennia. Brick/stone and mortar are the materials used in masonry construction, and (if applied properly) they enable to make a single monolithic material i.e. the masonry itself. Despite the significant number of new materials and technologies that have been developed over the last century, the way bricks, blocks and stones are laid in masonry has remained basically the same as it was thousands of years ago. It is largely based on empirical experience and only in recent decades various theories for calculating and designing masonry structures have begun to appear.

One of the least studied areas is the behavior of damaged masonry in structures. There are a large number of buildings of various ages around the world with load-bearing masonry structures that are continuously aging and require restoration and reinforcement with minimum intervention in the masonry structure to save their historic appearance.

Insufficiently developed methods for calculating masonry under its complex stress state, the lack of a unified approach to this kind of calculations and the lack of reliable information about the effect of certain damages on the strength and stability of masonry structures lead to the fact that in each case it is necessary to carry out a large number of strength tests and, with an individual approach, simulate the design in any special software package.

Masonry vaults were initially used by the Romans and Persians and for two and a half thousand years until the implementation of metal and reinforced concrete were the main type of flooring in civil, religious and defense architecture. Arches and vaults were also a major aesthetic element in Roman, Byzantine, Romanesque, Gothic and Renaissance architecture.

Vaults are usually sprung arch structures of continuous cross-section, with the length thereof in the directions perpendicular to the axis being comparable to the span. Arches are a special type of vault, the flat model thereof.

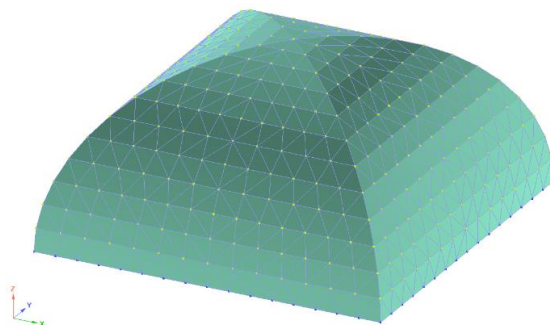
Due to the restoration of architectural monuments and religious buildings, as well as in connection with the construction of such facilities as Orthodox churches, the study of arches and vaults operation becomes especially important nowadays [1-9].

Despite the rapid development of new building technologies in the construction of churches, the application of these new technologies should be limited and focused on the use of traditional materials. In particular, the walls and vaults of temples are recommended to be made of solid bricks. This is due to two reasons. The first reason has to do with the symbolic significance of each structural or architectural element of the temple. Indeed, all the structural elements of the temple must consist of individual bricks, just as the church fundamentals are formed by a number of believing people. As a rule, the estimated temperature of 14-16°C is constantly maintained in an empty temple [10,11]. However, with the beginning of the service, when 150 or more people gather in the temple, the microclimatic parameters change dramatically. At least twice a day, structural elements and individual materials are subjected to the strongest moisture and thermal shocks. Moreover, the properties of masonry are also affected by the use of lime mortar with the addition of cement, as the current economic conditions for the manufacture of building materials prevent lime from the required binding properties (SP 15.13330.2020 Code of Regulations "Masonry and Reinforced Masonry Structures").

Over the last decades the practice of calculation and engineering of building structures has included the use of universal PC software, that made it possible to significantly improve the calculation models, bringing their functioning as close as possible to the operation of the actual structure [12, 13].

## 2. METHODS AND MATERIALS

The development of software systems for the calculation of structures enables us to analyze the load-bearing behavior of arches and vaults of almost any configuration. When selecting the curvilinear vault formations, it should be kept in mind that the higher the arc elevation, the lower the thrust force of the vaults transmitted to the walls and supports [14-16]. One of the most common types of vaults is the domical vault consisting of two intersecting cylinders with the same elevation and resting on four sides (Fig. 1).



**Fig. 1.** Geometric model of a domical vault.

The domical vault considered herein is formed by wall extensions inclined along a given curve, converging in a horizontal crown under rectangular plan or in one point under square plan [17]. A domical vault is derived from a cylindrical vault and transmits vertical pressure and thrust force to the walls along its entire length (Fig. 1).

In accordance with SP 31-103-99. Code of Rules for Engineering and Construction "Buildings, Structures and Facilities of Orthodox Churches" the calculation scheme of a domical vault may be considered in general as a system of elementary semi-arches, forming troughs and transferring the strut to the conditional diagonal ribs of the vault if there is a central light drum and the support ring thereof [18, 19]. The lower support (fifth) of the semi-arches transfers the thrust and load pressure to the support ring.

The vault design under consideration being curvilinear, a number of operations should be considered for the formation thereof. So, when partitioning curved surfaces into finite elements, the following assumptions may be used: an element of a curved surface can be considered flat if the inequality  $r = a^2/t$  is satisfied, where  $r$  is the radius of curvature of the surface;  $t$  is the thickness of the surface;  $a$  is the size of the element. That is, when dividing the vault into finite elements, the following condition must be satisfied:

$$a \leq (r \cdot t)^{\frac{1}{2}}. \quad (1)$$

The calculation is made in the LIRA CAD program. The side of the shell is divided into at least 8-10 elements. Finite elements with dimensions of 0.3-0.4 m are accepted in the considered case.

Boundary conditions are used for fixing from linear displacements along x,y and z.

Vaults in linear and nonlinear formulations were calculated by the authors of the article, and the results thereof were used to make a comparative analysis of forces and deformations of the vaults [20].

To reveal the influence of the vault height on the stress-strain state, four versions of the lifting boom were considered: 3 m, 2.25 m, 1.5 m, 0.75 m (Table 1).

The following finite elements were used for linear calculation:

- universal triangular finite element of shell KE42;
- universal quadrangular finite element of shell KE41, 44.

**Table 1.** Calculation values.

Calculation version	span, m	Shell thickness, m	Lifting boom, m
1	6,0	0,51	3,00
2	6,0	0,51	2,25
3	6,0	0,51	1,50
4	6,0	0,51	0,75

Loads taken into account in the calculation:

1. own weight is taken into account in the program automatically;
2. payload – calculated value 800 kg/m<sup>2</sup>;
3. short-term part of the load 600 kg/m<sup>2</sup>;
4. long-term part of the load 200 kg/m<sup>2</sup>..

Materials of masonry:

1. ceramic brick grade - M200;
2. cement-lime mortar grade 150.

Estimated resistance of masonry in compression (long-term loads) R = 3.0 MPa (Table 6).

Estimated resistance of masonry to tensile bending strength over the tied section R<sub>tb</sub> = 0.4 MPa.

Estimated resistance of masonry to tensile bending strength over the untied section R<sub>tb</sub> = 0.12 MPa.

Temporal resistance of masonry to compression (short-term loads) at k=2:

$$R_u = k \cdot R = 2 \cdot 3 = 6,0 \text{ МПа.}$$

Modulus of elasticity of masonry (initial modulus of deformation) at masonry elastic characteristic  $\alpha = 1000$  for unreinforced masonry:

$$E_0 = \alpha \cdot R_u = 1000 \cdot 6 = 6000 \text{ МПа.}$$

Modulus of elasticity of masonry at short-term load:

$$E = 0,8 \cdot 6000 = 4800 \text{ МПа.}$$

Modulus of elasticity of masonry at long-term load:

$$E = \frac{E_0}{\nu} = \frac{6000}{2,2} = 2727 \text{ МПа,}$$

where,  $\nu$  – coefficient of creep.

Shear modulus of masonry:

$$G = 0,4 \cdot E_0 = 0,4 \cdot 6000 = 2400 \text{ МПа.}$$

The relative deformations of the masonry under short-term load are calculated according to the following formula:

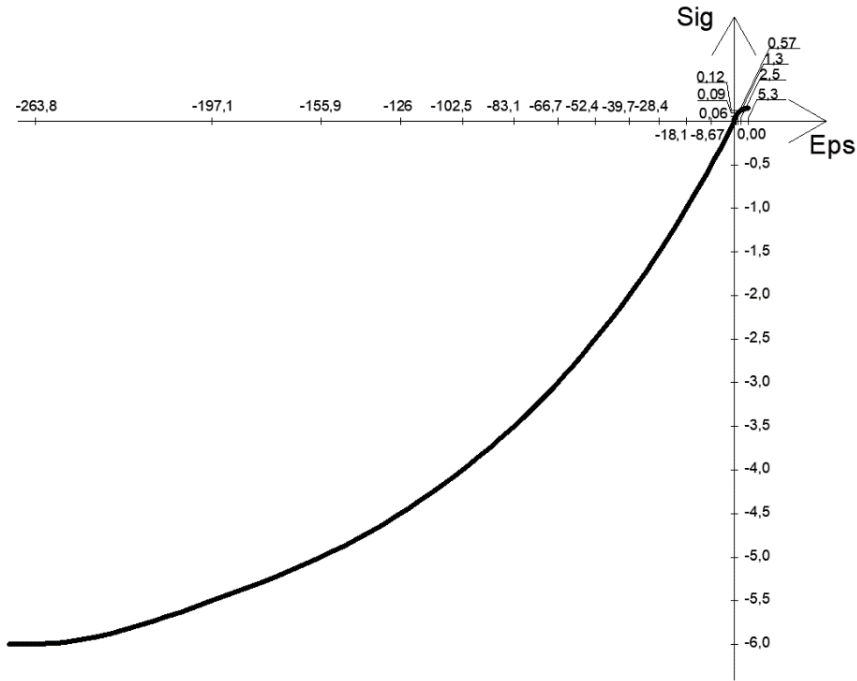
$$\varepsilon = \frac{1,1}{\alpha} \ln \left( 1 - \frac{\sigma}{1,1 R_u} \right), \quad (2)$$

where,  $\sigma$  – stresses,  $\alpha$  - elastic characteristic of masonry, R<sub>u</sub> - masonry time resistance.

Data for plotting the deformation diagram under short-term load are presented in Table 2 and Fig. 2.

**Table 2.** Data for plotting the deformation diagram under short-term load.

Compression side		Tension size	
Stresses $\sigma$ , MPa	Relative deformation $\varepsilon$	Stresses $\sigma$ , MPa	Relative deformation $\varepsilon$
0,0	0,00	0,00	0,00
0,5	8,67E-05	0,06	5,67224E-06
1,0	0,000181	0,09	1,3335E-05
1,5	0,000284	0,12	2,51929E-05
2,0	0,000397		5,27537E-05
2,5	0,000524		
3,0	0,000667		
3,5	0,000831		
4,0	0,001025		
4,5	0,001260		
5,0	0,001559		
5,5	0,001971		
6,0	0,002638		



**Fig. 2.** Deformation curve under short-term load in LIRA CAD program.

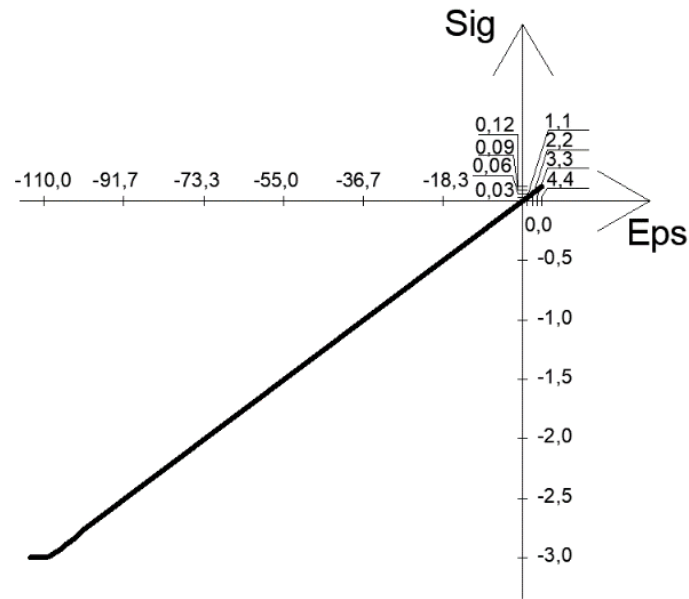
In accordance with p. 6.24 of SP 15.13330.2020 Code of Regulations "Masonry and Reinforced Concrete Structures" relative deformations of masonry under long-term load are calculated according to formula [21]:

$$\varepsilon = \nu \frac{\sigma}{\varepsilon_0} \quad (3)$$

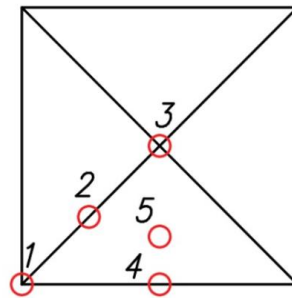
Data for plotting the deformation curve of the material under long-term loads are given in Table 3 and Fig. 3.3.

**Table 3.** Data for plotting the deformation curve under long-term load.

Compression side		Tension size	
Stresses $\sigma$ , MPa	Relative deformation $\varepsilon$	Stresses $\sigma$ , MPa	Relative deformation $\varepsilon$
0,0	0,00	0,00	0,00
0,5	0,000183	0,03	0,000011
1,0	0,000367	0,06	0,000022
1,5	0,000550	0,09	0,000033
2,0	0,000733	0,12	0,000044
2,5	0,000917		
3,0	0,001100		



**Fig. 3.** Deformation curve under long-term load in LIRA CAD program.



**Fig. 4.** The forces points in table 4.

### 3. RESULTS AND DISCUSSION

The results of calculations of domical vaults for different options of the lifting boom in linear and nonlinear formulations are summarized in Table 4. The forces are determined at the points shown in Fig. 4.

**Table 4.** Calculation results.

		Linear analysis					Nonlinear analysis				
		Mx	My	Nx	Ny	f, mm	Mx	My	Nx	Ny	f, mm
Option 1 Elevation height 3 m	Point 1	0.12	0.39	-24.48	-111.26		0.26	0.51	-19.28	-87.62	
	Point 2	-1.21	-0.47	-3.01	-31.17		-1.48	-0.94	-4.7	-28.71	
	Point 3	-1.62	-1.47	-61.3	-61.14	-0.135	-1.84	-1.72	-56.81	-56.28	-0.091
	Point 4	0.30	1.07	-22.58	-102.64		0.32	1.23	-25.55	-116.15	
	Point 5	0.27	0.58	-8.93	-74.64		0.11	0.54	-4.33	-76.64	
Option 2 elevation height 2.25 m	Point 1	0.16	0.33	-17.66	-80.28		0.29	0.43	-12.93	-58.77	
	Point 2	-0.87	-0.22	-3.57	-22.49		-1.28	-0.84	-5.65	-20.91	
	Point 3	-2.47	-2.36	-69.47	-69.51	-0.149	-2.54	-2.46	-62.85	-62.52	-0.098
	Point 4	0.29	1.15	-21.13	-96.04		0.23	1.23	-24.18	-109.91	
	Point 5	0.26	1.36	-14.15	-79.96		0.04	1.20	-9.55	-81.09	
Option 3 elevation height 1.5 m	Point 1	0.13	0.23	-8.02	-36.47		0.24	0.31	-5.2	-23.64	
	Point 2	-0.16	0.37	-6.78	-16.40		-0.87	-0.6	-10.49	-18.77	
	Point 3	-3.84	-3.78	-77.98	-78.32	-0.188	-3.5	-3.45	-70.67	-70.71	-0.115
	Point 4	0.30	1.28	-22.08	-100.36		0.27	1.17	-24.85	-112.97	
	Point 5	0.12	2.37	-22.82	-93.43		-0.17	1.87	-19.3	-91.47	
Option 4 elevation height 0.75 m	Point 1	-0.19	-0.12	5.71	25.93		-0.24	-0.02	8.64	39.09	
	Point 2	0.69	1.08	-19.51	-27.78		-0.91	-0.99	-26.71	-34.85	
	Point 3	-6.39	-6.37	-90.20	-90.89	-0.307	-5.24	-5.24	-90.85	-91.41	-0.178
	Point 4	0.29	1.38	-28.37	-128.94		0.22	1.05	-30.02	-136.46	
	Point 5	-0.67	2.58	-39.21	-129.16		-0.94	1.42	-37.76	-121.35	

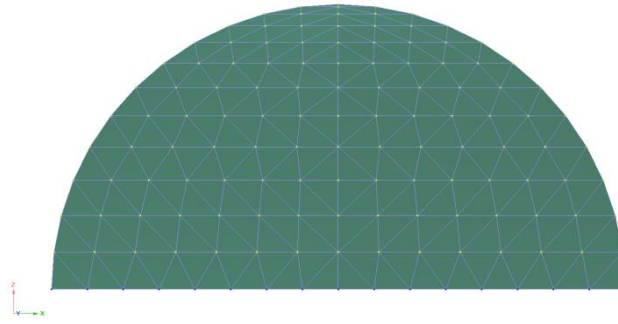


Fig. 5. Finite element mesh for a vault with an elevation height of 3 m.

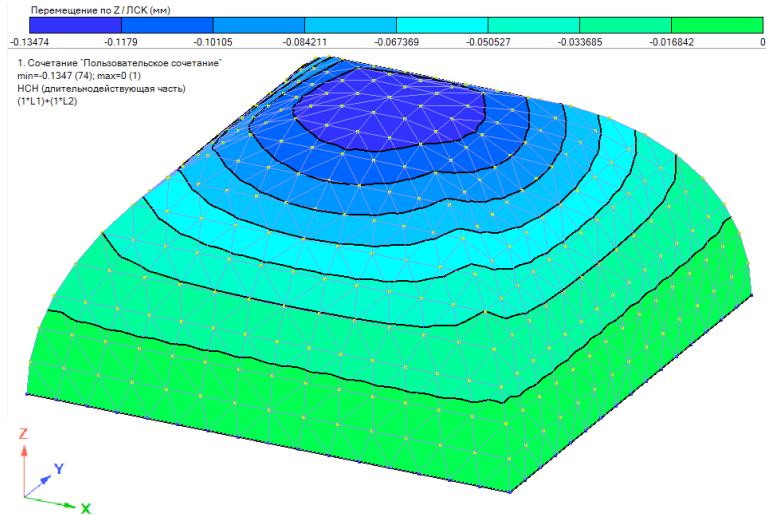


Fig. 6. Vertical deformations, mm, for a vault with an elevation height of 3 m.

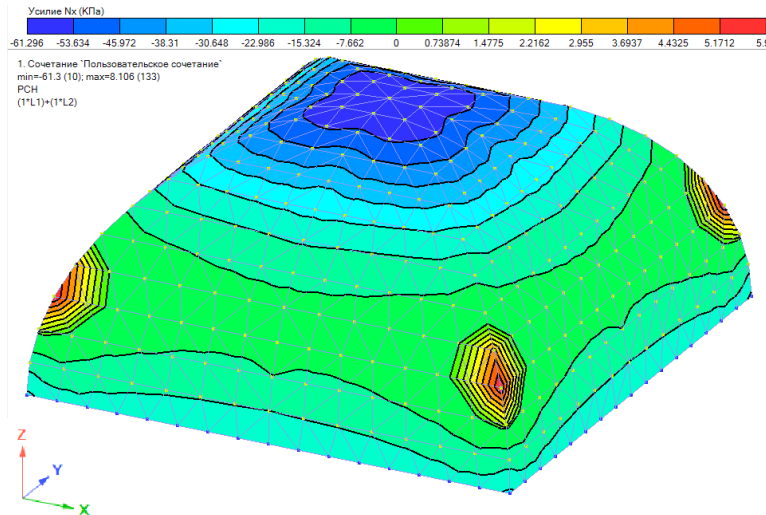
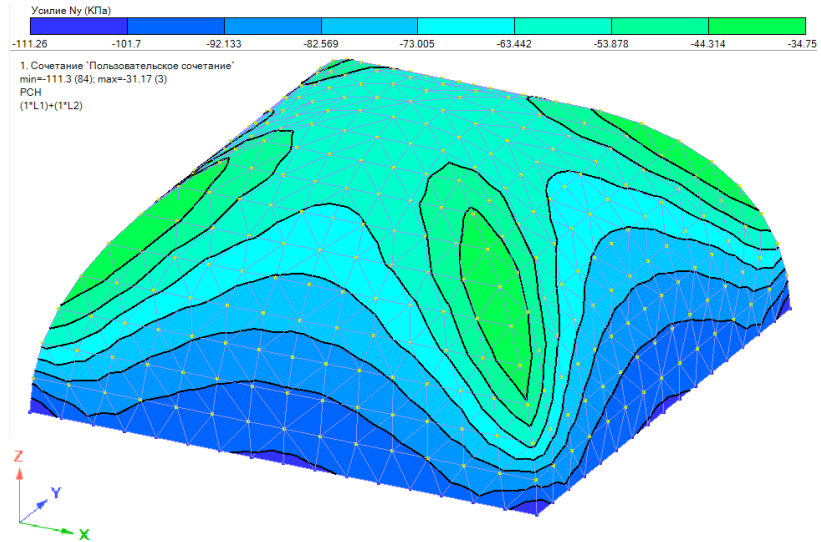
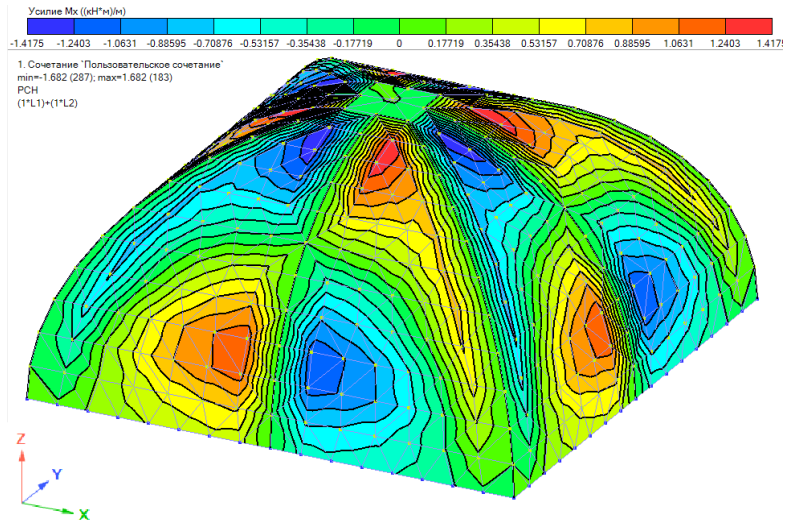


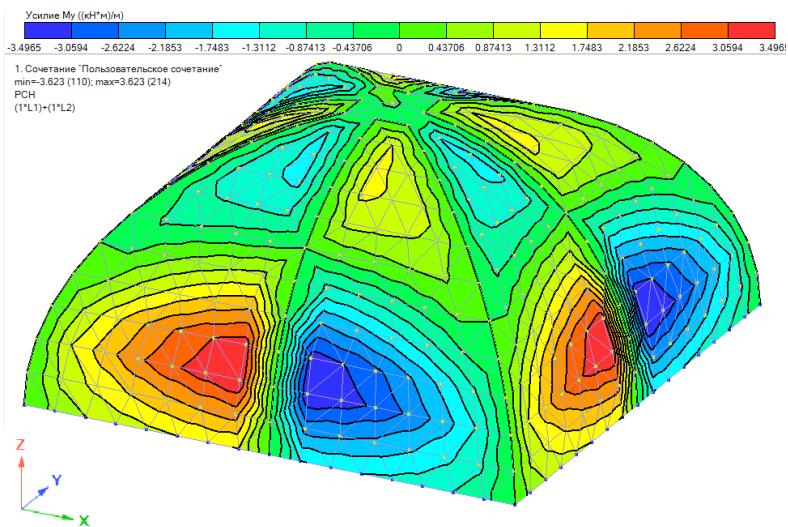
Fig. 7. Force  $N_x$ , kPa, for a vault with an elevation height of 3 m.



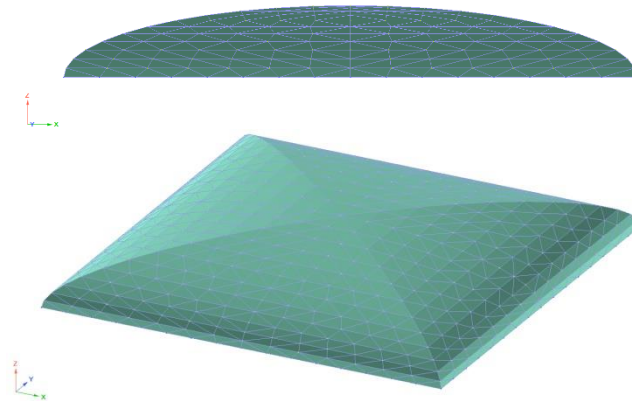
**Fig. 8.** Force  $N_y$ , kPa, for a vault with an elevation height of 3 m.



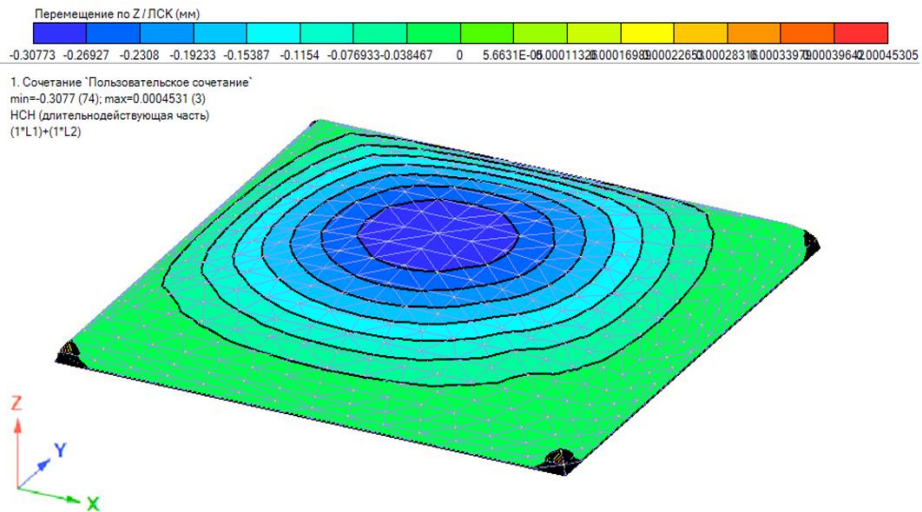
**Fig. 9.** Force  $M_x$ ,  $kN \cdot m/m$ , for a vault with an elevation height of 3 m.



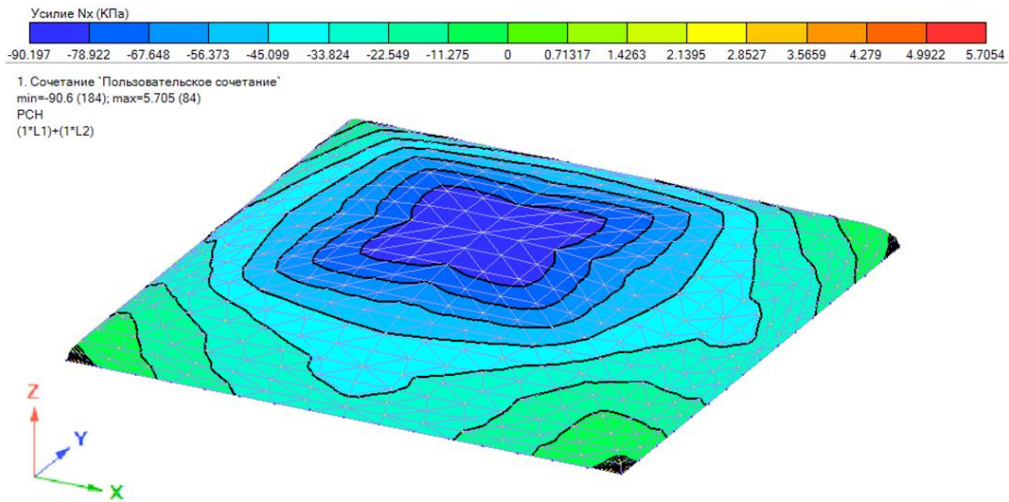
**Fig. 10.** Force  $M_y$ ,  $kN \cdot m/m$ , for a vault with an elevation height of 3 m.



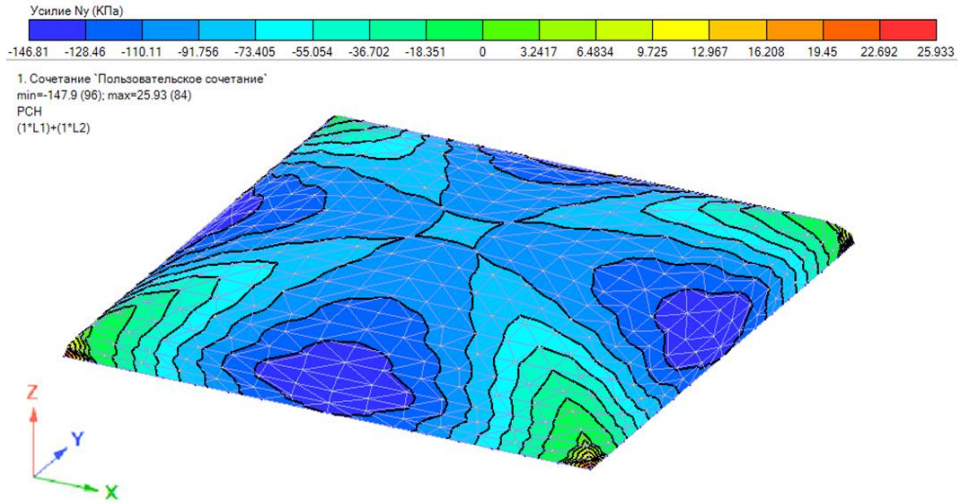
**Fig. 11.** Finite element mesh for a 0.75 m high vault.



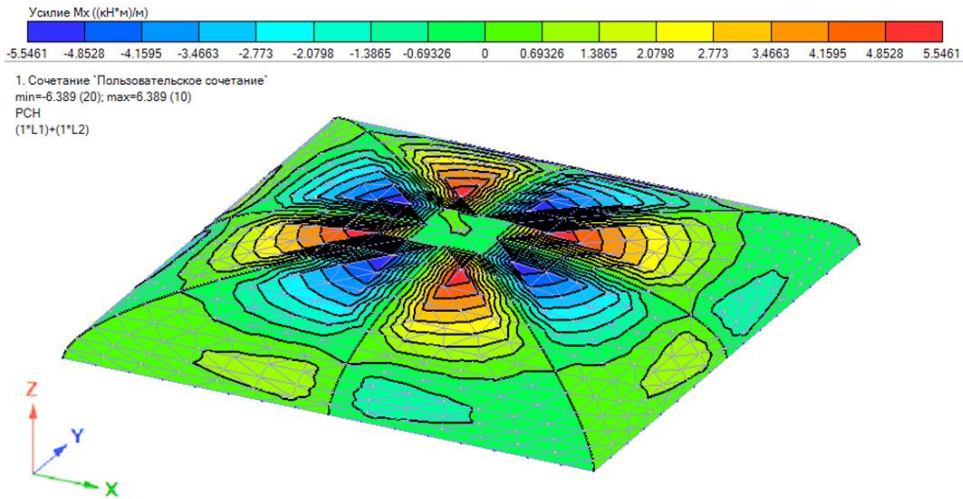
**Fig. 12.** Vertical deformations, mm, for a vault with an elevation height of 0.75 m.



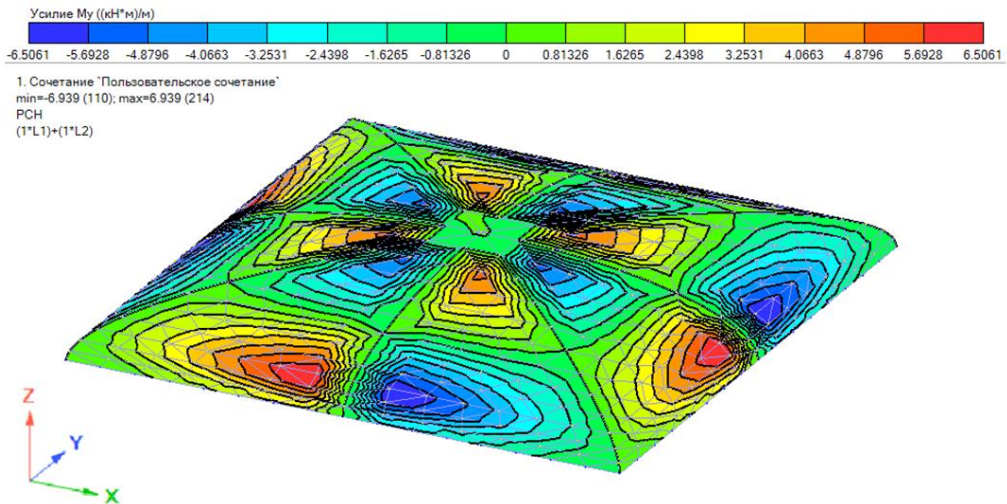
**Fig. 13.** Force Nx, kPa, for a vault with an elevation height of 0.75 m.



**Fig. 14.** Force Ny, kPa, for a vault with an elevation height of 0.75 m.



**Fig. 15.** Force Mx, kN\*m/m, for a vault with an elevation height of 0.75 m.



**Fig. 16.** Force My, kN\*m/m, for a vault with an elevation height of 0.75 m.

The calculation model of the domical vault for option 1 is shown in Fig. 5. The isopoles of vertical deformations, forces  $N_x$ ,  $N_y$ ,  $M_x$ ,  $M_y$  for linear calculations are shown in Figs. 6-10. The distribution of deformations and forces for option 2 and 3 has a similar profile. The isopoles for option 4 with a lifting boom are shown in Fig. 11-15.

The forces indicated in Table 4 are specified at the most characteristic points of the brick vault shown in Figure 4. The calculation model of the domical vault (for option 1) is shown in Figure 5.

For a more clear assessment of the difference in determining the results of forces in the vault elements for linear and nonlinear calculations, the forces and deformations indicated in Table 4 are represented in three-dimensional configuration as 3D graphs. The isopoles of vertical deformations, forces  $N_x$ ,  $N_y$ ,  $M_x$ ,  $M_y$  for linear calculations are shown in Figs. 6-10. Since the distribution of deformations and forces for option 2 and 3 has in general a similar profile, we think their distribution for option 4 (with a minimum lifting boom of 0.75 m), with the isopoles thereof being shown in Fig. 11-15, are to be considered.

Let us analyze in more detail the difference in the results of the calculation of forces and deformations considering linear and nonlinear relationship between stresses and deformations in the vault masonry. At the highest masonry rising height of 3 m (variant 1) for the most loaded lower angle reference point 1, the greatest difference occurs for the moment  $M_x$ , in particular, the linear calculation gives more than double underestimation of the given force in this section (0.12 instead of 0.26). a significant underestimation of the mentioned force  $M_y$  by more than a quarter (0.39 instead of 0.51 for linear calculation) is also given for the bending moment in another mutually perpendicular section of this point 1, and considering the higher quantitative values of  $M_y$  in comparison with  $M_x$  at this corner point, failure to take into account the increase of  $M_y$  may be very dangerous, as it is the bending moments that generate tensile stresses in the brick masonry. Analyzing the compressive stresses  $N_x$  and  $N_y$  in linear and nonlinear formulations of the calculation problem, we observe the opposite picture; compared to the bending moments linear calculation gives an overestimation of longitudinal forces in approximately the same percentage ratio ( $N_x$  from  $-19.28$  to  $-24.48$ , and  $N_y$  – from  $-87.62$  to  $-116.26$ ). It may appear that the overestimation of longitudinal forces by 25-30% for linear calculation, compared to a more accurate non-linear calculation (taking into account not only elastic but also plastic deformations) results in an increase in safety margin, but this is not the case: the higher the bending moments and the lower the longitudinal compressive forces for any section of masonry, the greater the probability of tensile stresses, which are known to be the most critical for masonry, they are the cause of cracks and even sudden collapse of the entire structure. Therefore, one can state with a high degree of certainty that for the most critical angle point of the square vault, failure to take into account the physical nonlinearity of the masonry material negatively affects the reliability of the calculation results, as it results in a significant overestimation (by more than 25%) of the masonry load-bearing capacity.

For the non-angle reference point (point 4), the above tendency is fundamentally preserved, only the quantitative overestimation of bearing capacity in the linear calculation of masonry at this point is significantly lower (by about 10-12 %) than in the accurate nonlinear calculation, considering the elastic-plastic characteristics thereof.

The same tendency of the force ratio is observed for a particularly important point of the ridge vault of the brick masonry (point 3); here, the calculation based on the linear relationship between stresses and deformations gives clearly more than 10% overestimation of the load-bearing capacity of the masonry at this point for all acting forces ( $N_x$ ,  $N_y$ ,  $M_x$ ,  $M_y$ ), compared to the non-linear calculation. Analyzing the results of these calculations for the intermediate points of the vault (points 2 and 5), we can state that, basically, the same tendency remains, but the difference in the quantitative values of the main forces (longitudinal forces and bending moments) for the linear and nonlinear formulation of the calculation problems is even smaller (approximately 5-8 %) and, in general, this difference can be neglected, since these areas in masonry vaults are not the most critical ones.

Analyzing the distribution of deformations for the first option, the linear calculation may be noted to give a significant overestimation of deformations for equal loads on the brick vault by more than 40%. Therefore, the maximum permissible (or safe from the point of view of the masonry operating under load) deflections, considering a more accurate non-linear calculation, will be much smaller, and

this important circumstance should be taken into account both in the engineering of new and in the assessment of the load-bearing capacity and operational reliability of existing masonry structures.

Analyzing the data of the calculations made to identify the forces in the brick vault in linear and non-linear formulations of the calculation problems at lower values of the masonry rising height (options 2, 3), we may conclude that the dependencies described above for the first option (3 m rising height) in the most loaded angle reference cross-sections (point 1) are generally maintained in approximately the same quantitative ratios, in the span cross-sections (points 3, 2, 5), basically, similar differences in the estimation of forces in linear and nonlinear formulations are also observed, but the quantitative differences are very small (5-10%) and practically disappear with the reduction of the masonry rising height.

Analyzing the distribution of forces and corresponding stresses in the masonry with the minimum height of the vault elevation (0.75 m, option 4), which is no longer an arch structure but a plate, we can note a very slight difference in the quantitative values of forces in the linear and non-linear calculation versions (actually within the permissible error of the methodology of 5 - 7%), but the difference in the maximum vertical deformations (point 3 in the ridge vault) in this case is maximum (0.307 mm for the linear calculation and 0.178 mm for the non-linear calculation), i.e. almost twofold difference in the vertical deformations (point 3 in the ridge vault). i.e. almost a twofold difference in the calculations. In this case, it seems especially important for structural solutions of flat masonry vaults, with the stiffness thereof being significantly lower than that of arched and parabolic vaults, to pay attention to the exact definition of the deformability of the structure and to analyze the relationship between forces and deformations.

#### 4. CONCLUSIONS

1. The calculation of the considered structures in a nonlinear formulation significantly affects their stress-strain state. Therewith the difference is observed in the values of bending moments.

2. The values of deflections are found to have significant differences in linear and nonlinear formulations, reaching 40%. While the elevation height decreases, the values of deflections in linear and nonlinear formulations increase.

3. Normal forces as well as bending moments are redistributed for up to 5 times. Therewith the elevation height decreases, the force values in the nonlinear formulation become more comparable to the linear formulation.

4. Calculations of masonry vaults under consideration (especially those with large elevation heights) are recommended to be made with consideration of physical and geometrical nonlinearities.

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