Parametric research of the two-layered grouted shells’ bearing strength

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Abstract. The results of the two-layer reinforced cement shells’ bearing strength parametric study under the temperature and power effects of a standard fire are presented. The influence and the magnitude dependence plots of the power load, concrete class and the thickness of the shell layers on the structure’s bearing strength are revealed. The optimal thicknesses of the structural and protective layers of the shell, as well as the cement-cement layer concrete grade, are determined.

Introduction

Thin-walled, spatial structures, including shells, are widely used in construction [1-3]. Despite their advantages, their implementation in construction practice is constrained by low fire resistance.

The building structures’ fire resistance is determined by testing on a fire chamber, which is a time-consuming and expensive task [4-5]. Therefore, the urgent task is to develop calculation methods.

In [6], we proposed a method for calculating the two-layer reinforced-cement shells on temperature and force effects in a standard fire, taking into account physical nonlinearity. This method is based on the gentle shells’ theory equations. The hypothesis of direct normals is used. Total deformations represent the sum of the middle surface deformations and the deformations caused by the change in curvature. Deformation of reinforcement is determined from the conditions of its joint work with concrete. In deriving the resolving equations, the change in the material characteristics over the thickness was taken into account due, firstly, to the two-layer structure, and secondly, to the inhomogeneity caused by the temperature effect. Total concrete deformations include the forced deformations, which represent the algebraic sum of thermal expansion and shrinkage deformations. The tangent modulus for concrete is determined in accordance with the genius deformation theory of plasticity [7-8]. Reinforcement is assumed to be elastic until the yield point is reached; its temperature deformations are taken into account. As a result, a matrix relationship is obtained between internal forces and generalized deformations. The calculation was performed using the finite element method.

The variational principle of the minimum total energy was used. The solution of the physically nonlinear problem was carried out step by step by the Newton-Raphson method [9-10].

The developed methodology testing [6] was carried out on the experimental data of Professor T.A. Khezhev. He conducted fire tests of single-layer and two-layer reinforced cement shells of double curvature (Figure 1).
Reinforcement was carried out using two layers of woven mesh no. 8 with a wire diameter of 0.7 mm, as well as longitudinal reinforcement from the ordinary B – I wire with a diameter of 5 mm (14 pieces), located between the grids. The concrete class of single-layer shells is B30, and for the two-layer shells - B20. For a vermiculite-concrete layer of two-layer shells of compressive and bending strength, respectively $R_{\text{comp}} = 2.3$ MPa, $R_{\text{prod}} = 1.3$ MPa.

The tests were conducted on the combined effect of temperature and power loads. The temperature regime corresponded to the conditions of a standard fire regulated by GOST 30247.0–94. Power loads were represented by three options: a load uniformly distributed over the entire shell area - 1 kPa and 2 kPa, as well as a concentrated force in the middle of the span – 16 kN.

The experimental changes’ curves in time displacements for the single-layer and two-layer shells are presented, respectively, in Figure 2 and Figure 3. The “+” sign on the indicated graphs corresponds to the upward movement.

**Figure 1.** Section of a single-layer (a) and two-layer (b) shell

**Figure 2.** The change in displacements (in mm) in time for the single-layer shells: 1 – $q = 1$ kN/m$^2$, 2 – $q = 2$ kN/m$^2$, 3 – $F = 16$ kN

**Figure 3.** The change in displacements (in mm) in time for the two-layer shells: 1 – $q = 1$ kN/m$^2$, 2 – $q = 2$ kN/m$^2$, 3 – $F = 16$ kN
The resulting graph of the change in time of displacements in the middle of the span for a single-layer shell at \( q = 2 \text{ kPa} \) is presented in Figure 4.

![Figure 4](image_url)

**Figure 4.** The change in indexing time in the middle of the span for the single-layer shell at \( q = 2 \text{ kN/m}^2 \)

As the shell moves up in the experiment, at its initial stage, however, the calculated value of the deflection at \( t = 0 \) is significantly different from the experimental value. According to the calculation results, the value \( W_{\text{max}} \) at \( t = 0 \) amounted to 0.0205 mm, and based on Fig. 4.4, the experimental value is approximately 3 mm. This significant deviation can be explained by the reference contour displacement at the load application time. Maximum theoretical displacement \( W_{\text{max}} = 14.7 \text{ mm} \) is in good agreement with the experimental value (taking into account the reference contour settlement \( W_{\text{exp}} = 10 + 3 = 13 \text{ mm} \)). On the theoretical curve for \( t = 25 \text{ min} \) there is a kink. From this moment in time, the structural stiffness matrix was close to degenerate, which indicates the beginning of the shell destruction.

Figure 5 presents the calculation results of the two-layer shell under load \( q = 1 \text{ kN/m}^2 \). The destruction of the shell for 200 min, as in the experiment, does not occur. In contrast to the experimental data, there is a significant difference with the results for \( q = 2 \text{ kN/m}^2 \), as in the case of a single-layer construction, it is not detected. Also Figure 3 unlike Figures 4, 5 noteworthy shows the noticeable initial deflection, probably related to the reference contour displacement.
Figure 5. The change in the maximum displacements time of the two-layer shell at $q = 1 \text{kN/m}^2$

Figure 6 presents the maximum deflection dependence graph of the two-layer shell considered in the previous experiment on the load at the initial instant of time. From this graph it is seen that the destruction of the structure occurs under load $q = 44 \text{kPa}$.

Figure 6. The dependence of the shell’s maximum deflection on the load at $t = 0$

In the experiment considered earlier, the force loads were 1 and 2 kPa, which is significantly lower than the destructive load; therefore, the temperature field turned out to be the main factor affecting the shell’s SSS, and no difference in the curves of changes in displacements was revealed.

Let us consider the power load magnitude effect on the fracture time. The graphs of the deflection time variation at the central point of the two-layer shell in time are presented at various values $q$ on the Figure 7.
Figure 7. Variation in time of deflection in the center of the shell at various load values

At a load of 25 kPa, which is 57% of the instantaneous limit, the destruction of the structure within 200 minutes is not observed. At $q = 30$ kPa degeneracy of the stiffness matrix, indicating proximity to the limiting state, occurred at a time $t = 146$ min, by $q = 35$ kPa – by $t = 40.5$ min, and when $q = 40$ kPa destruction time was 11.5 minutes. Figure 7 shows that with the case exception $q = 40$ kPa the deformations due to force are small compared to the thermal deformations.

The influence of the fine-grained concrete class on the bearing strength of the structure was also investigated. Graphs of the deflection in the center of the shell versus time for a different class of concrete of the bearing layer, constructed at $q = 25$ kPa, are given on Fig. 8. For the class B20 concrete and higher, at a given load, failure does not occur within 200 minutes, with class B15, the destruction occurs after 102.5 minutes, with the class B12.5 - after 64.5 minutes, with the class B10 - after 29 minutes.

Figure 8. The change in deflection in the center of the shell depending on the concrete class

In addition to the indicated load, the calculation was performed at $q = 2$ kPa. In this case, a change in the concrete class did not lead to a decrease in bearing capacity, the structure did not collapse within 200 minutes even with class B7.5.

Next, we studied the layer thickness effect on the strength characteristics of the shell. We take $h_1$ as the cement-cement layer thickness. The calculation was performed with the following initial data:
load $q = 2 \text{kPa}$, fine concrete class B20, total shell thickness $h = 33 \text{mm}$. Values $h_1$ were taken equal to 15, 20, 25 and 28 mm. The time variation corresponding graphs of the maximum displacements are given in Figure 9.

![Figure 9](image_url)

**Figure 9.** The change in time of maximum movements at various layers’ thicknesses

**Summary**

From the presented graphs it is seen that with a decrease in the fire-retardant layer’s thickness, the shell bending increases. If the thickness of the flame-retardant layer is less than the minimum allowable, the structure is destroyed. In the considered example, when the fire-retardant layer’s thickness was 8 mm or more, the structure did not collapse, and when the vermiculite-concrete layer’s thickness was 5 mm, the destruction occurred after 85 min.

In this way, the methodology [6] for calculating two-layer reinforced-cement shells for temperature and force effects in a fire was tested, taking into account physical nonlinearity on the available experimental data. The coincidence of the results is good; the deviations of the theory from experiment can be attributed to the following possible reasons:

1. The scatter of the physical and mechanical characteristics of materials in the tested structures and the inaccuracy of their determination;
2. The age of the shells at the test time (there may have been a change in properties over time);
3. The introduced assumption about the shape of the shell surface;
4. Errors of the deformation theory of plasticity by G.A. Geniev;
5. Errors of the shallow shells’ theory.

**References**

[1] Lysenko E F 1981 Reinforced cement structures (Vishka School, Kiev).
[2] Mitrofanov E N 1973 Reinforced cement (Leningrad).
[3] Baykov V N, Hampe E Raue 1990 Design of reinforced concrete thin-walled spatial structures (Stroyizdat, Moscow).
[4] Panarin S N, Khezhev T A, Somov V I 1982 Fire resistance of cementitious cement with a fire-retardant layer based on expanded vermiculite (Ways to increase the fire resistance of building materials and structures: materials from the seminar MDNTP named after F.E. Dzerzhinsky, Moscow).
[5] Popov P I, Chekel T V 1963 Experimental studies of fire resistance of an armored cement corrugated arch (Armored cement structures in construction, Leningrad).
[6] Zhurtov A V, Khezhev T A, Chepurnenko A S, Saibel A V 2019 Two-layer ferrocement shells stress-strain state modeling under the fire conditions *Materials Science Forum* **974** 515 – 520.
[7] Geniev G A, Kissyuk V N, Tyupin G A 1974 Theory of plasticity of concrete and reinforced concrete (Stroyizdat, Moscow).

[8] Geniev G A, Kissyuk V N, Levin N I, Nikonova G A 1978 Strength of light and cellular concrete in difficult stress conditions (Stroyizdat, Moscow).

[9] Polyak B T 2006 Newton’s method and its role in optimization and computational mathematics Transactions of ISA RAS 28 44-62.

[10] Vazan M 1972 Stochastic approximation (Mir, Moscow).