Determine the Earthquake of the Consolidated Reinforced Concrete Structure of the Subway Tunnel in the Form of a Single-Domed Semicircle

Shakhboz Normurodov and Gulchekhra Ismailova

November 23, 2023

Determine the earthquake of the consolidated reinforced concrete structure of the subway tunnel in the form of a single-domed semicircle

Sh Normurodov[1], G Ismailova[1]

[1]Tashkent State Transport University, 100067, Tashkent, Uzbekistan

[1]normurodovsh25@mail.ru

Abstract. In the work, work was carried out to determine and assess seismic resistance of various types of single-vaulted structures of subway tunnels under seismic influences and appropriate conclusions were given. The design and construction of small-scale station tunnels from single-vaulted structures have now shown many of its positive factors. However, studies have shown that in seismic zones they are more vulnerable than those built in deeper conditions.

The consolidated reinforced concrete structure of the subway tunnel in the form of one dome is currently designed for the future and has a number of advantages over other structures. One of the most pressing issues is the assessment of the objectivity of this design and the conduct of research work on it and the adoption of seismic impacts as an external load.

1. Introduction

An analysis of the experience of subway construction in our country shows that not only certain technical and economic advantages of the construction and operation of such lines contribute to the growth of the length of shallow laying lines, but also the widespread use of a closed method of tunneling, which significantly reduces the negative impact of construction work on the normal living conditions of the city.

It is known that underground structures, both during construction and during the operational period, are objects of increased danger to the personnel working in them. This is caused by the objective presence of natural and man-made factors, a dangerous combination of which is often difficult to foresee, and therefore eliminate in advance. In most cases, the prediction of possible undesirable situations and effective measures to prevent or eliminate them should take into account the experience gained by world practice to the maximum extent [1].

It is known that transport tunnels are considered as capital structures designed for a long service life (more than 100-150 years). During this period, they must meet the requirements of operational reliability, ensuring reliability, durability, preservation and maintainability of the structure as a whole and its components, i.e. the ability of the structure to perform specified functions.

2. Methods

When calculating a closed monolithic workout of a non-circular cross-section, a plane contact problem of elasticity theory on the equilibrium of an arbitrary-shaped ring with one axis of symmetry supporting a cutout in an elastic weighty half-plane is considered. The formulation and solution of the problem belong to N. N. Fotieva, K. V. Ruppeneit, based on the assumption of the hydrostatic household stress state of the massif, considered the axisymmetric problem of the interaction of a mountain massif with a ring lining [2]. According to the theory of K. V. Ruppeneit, when mining is carried out in its vicinity, stresses are redistributed in the surrounding rock mass, the points of the contour of the mine receive elastic displacements inside the mine.

Since rocks have limited strength, inelastic deformations begin to develop in places of stress concentration, covering a certain area. Interesting results on the calculation of the seismic resistance of shallow tunnels were obtained by N. N. Fotieva, N. S. Bulychev and others on the basis of consideration of quasi-static problems of the theory of elasticity for a medium weakened by unsupported or reinforced holes, experiencing non-infinity biaxial compression or net shear, simulating, respectively, the action of long (more than 3 times the size of the holes) longitudinal and transverse waves of arbitrary direction [3,4,5].

The loads on the lining are determined separately for the vertical and horizontal directions of seismic impacts:

a) The horizontal and vertical components of the seismic load from the own weight of the lining are calculated by the formula

$$S_k = Q_k k_c \qquad (1)$$

Where Q is the weight of the lining element assigned to the point k; b) the intensity of the horizontal inertial pressure of the soil on the lining within the height of the lining wall is determined by the formula

$$p_{c(y)} = p_y \cdot k_c \cdot tg(45° + \frac{\varphi}{2}) \qquad (2)$$

Where p_y - active ground pressure;

b) The horizontal component of the inertial mass of the filling soil above the tunnel within the span of the workings, applied to the upper part of the lining, is determined by the formula

$$P_c = g\, Hlf, \qquad (3)$$

Where l - is the working width, m; H - is the distance from the day surface, m; f - is the coefficient of friction of the soil on the lining.

c) The intensity of the vertical component q_c^{vert} determined by the formulas $q_c^{vert} = \gamma H k_c$ (from the weight of the full column H of the soil above the tunnel) or $q_c^{vert} = \gamma h_1 k_c$ (if vaulting is possible, where h_1 is the height of the vault).

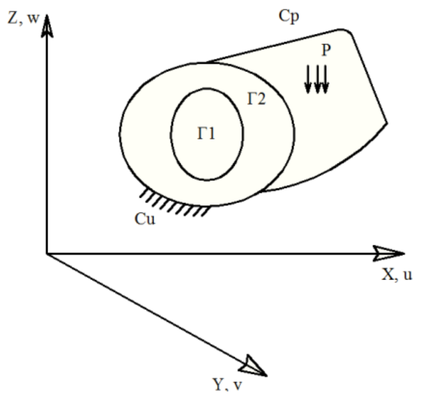


Fig. 1. System model «tunnel- soil environment»

A separate summation of horizontal forces from their own weight and inertial masses of the soil (the first combination) and vertical loads from their own weight and inertial masses of the soil (the second combination) is performed, two static calculations are performed for the first and second combination and the strength of the lining sections is checked. Under the influence of dynamic loads, a system in which an underground structure (structures of single–arch metro stations) and its surrounding environment soil (rock masses) interact under the pressure of external and volumetric forces is considered to change over time.

Here, under the body G_1, we understand the structures of single–arch metro stations, and under the body G_2 - the ground environment. We write the basic dynamic equations [6] for a system with the corresponding boundary and initial conditions:

$$A\vec{\sigma} + \vec{J} = 0$$
$$\vec{\varepsilon} = A^T\vec{U} \tag{4}$$
$$\vec{\sigma} = D\vec{\varepsilon}$$

Here according to the principle of Dalembert

$$\vec{J} = -\rho\frac{\partial^2\vec{U}}{\partial t^2} \tag{5}$$

Kinematic boundary conditions on the contour C_u

$$\vec{U} = \vec{U}_u \tag{6}$$

Let us determine in advance that at the border of the region at a sufficiently remote distance

$$\begin{cases} x \to \pm\infty \\ y \to \pm\infty \\ z \to \infty \end{cases}, \vec{U}_u = 0$$

Boundary conditions on the contour C_σ

$$A_c\vec{\sigma} = \vec{P}_\sigma \tag{7}$$

Let us determine in advance that if there are no external forces on the surface of the boundary of the region, then on this surface $\vec{P}_\sigma = 0$, otherwise $\vec{P}_\sigma \neq 0$.

Here $\vec{U}_u, \vec{P}_\sigma$ - vector of specified displacements on the contour - C_u and the vector of the given efforts on - C_σ, A_c - matrix of guiding cosines.

Initial conditions

$$\vec{U}_{(t=0)} = \vec{U}_0 \tag{8}$$

Let us determine in advance that when $t = 0$, $\{\vec{U}_0 = 0$

Then $ADA^T\vec{U} = \rho\frac{\partial^2\vec{U}}{\partial t^2}$ or

$$(\lambda+\mu)\frac{\partial\Delta}{\partial x} + \mu\nabla^2 u = \rho\frac{\partial^2 u}{\partial t^2}$$

$$(\lambda+\mu)\frac{\partial\Delta}{\partial y} + \mu\nabla^2 v + P_y = \rho\frac{\partial^2 v}{\partial t^2}$$

$$(\lambda+\mu)\frac{\partial\Delta}{\partial z} + \mu\nabla^2 w + P_z = \rho\frac{\partial^2 w}{\partial t^2}$$

$$\nabla = \varepsilon_x + \varepsilon_y + \varepsilon_z = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z}$$

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$$

$$\lambda = \frac{vE}{(1-2v)(1+v)}$$

Permanent Lames

$$\mu = \frac{E}{2(1+\nu)} \tag{9}$$

Equations (4, 9) with boundary and initial conditions (7, 8) constitute a complete direct statement of the problem of the dynamic theory of elasticity in displacements. Now we write the third equation of Hooke's law from system (4) according to the hereditary Boltzmann-Volterra theory [7]:

$$\vec{\sigma} = D(1 - R')\vec{\varepsilon} \tag{10}$$

Where R' - Volter integral operator, $R'f(t) = \int_0^t R(t-\tau)f(\tau)d\tau$

$R(t-\tau)$ - the core of heredity having a weakly singular feature of the Abel type

$$R(t-\tau) = \bar{\varepsilon}e^{-\beta(t-\tau)}(t-\tau)^{\alpha-1},\ 0 < \alpha < 1$$

To move from a continuum model of the problem to a discrete one, we use the Ostrogradsky-Hamilton variational principle [8].

Let some viscoelastic body be given in the space R, occupying the volume Ω and bounded by the surface $\mathrm{C} = C_u + C_\sigma$.

At the same time. Kinematic boundary conditions are set on C_u, a system of loads *f=(fx,fy,fz)* acts on C_σ, and there is no system of volumetric forces for the volume of the body Ω.

We consider that the vector is a function of displacements in the form

$\vec{U}(x,y,z,t) = [u(x,y,z,t)\ v(x,y,z,t)\ w(x,y,z,t)]^T$, delivers a minimum of the functionality of the total energy of the system in the form of:

$$I = \int_{t_1}^{t_2}(\Pi - T + \bar{A})dt = \int_{t_1}^{t_2}\int\int\int F(u,v,w,\frac{\partial u}{\partial t},\frac{\partial v}{\partial t},\frac{\partial w}{\partial t},\frac{\partial u}{\partial x},\frac{\partial u}{\partial y},\frac{\partial u}{\partial z},\frac{\partial v}{\partial x},\frac{\partial v}{\partial y},\frac{\partial v}{\partial z},\frac{\partial w}{\partial x},\frac{\partial w}{\partial y},\frac{\partial w}{\partial z})dxdydzdt \tag{11}$$

When restricted

$$\vec{U}\big|C_u = \vec{U}_u(x,y,z,t), \vec{U}\big|_{t=0} = \vec{U}_u(x,y,z,t), \vec{U}\big|_{t=0} = \vec{U}_u(x,y,z) \tag{12}$$

Where, $\vec{U}_u = \vec{U}_u(x,y,z,t), \vec{U}_1(x,y,z), \vec{U}_2(x,y,z)$ – given functions on C_u at t=0

$$T = \frac{1}{2}\oiiint_\Omega \rho(\left(\frac{\partial u}{\partial t}\right)^2 + \left(\frac{\partial v}{\partial t}\right)^2 + \left(\frac{\partial w}{\partial t}\right)^2)dxdydz,$$

$$\bar{A} = \frac{1}{2}\oiiint_\Omega (f_x u + f_y v + f_z w)dxdydz, \qquad \Pi = \oiiint_\Omega \bar{\Pi}dxdydz$$

Here, by introducing a viscoelastic potential $\Pi = \Pi_u - \Pi_v$ the Lagrange functional becomes potential.

Π_u, Π_v - elastic and viscous potential – introduced by academician Y.V. Rabat. Then according to [9], taking into account (5-12), we write an expression for potential energy in the form:

$$\begin{aligned}\bar{\Pi} = \frac{1}{2}\big[&(2\mu\varepsilon_x^2 + \lambda\Delta\varepsilon_x) + \left(2\mu\varepsilon_y^2 + \lambda\Delta\varepsilon_y\right) + (2\mu\varepsilon_z^2 + \lambda\Delta\varepsilon_z) + \mu(\gamma_{yz}^2 + \gamma_{zz}^2 + \gamma_{xy}^2\big] \\ &- \big[(2\mu\varepsilon_x^*\varepsilon_x + \lambda\Delta^*\varepsilon_x) + \left(2\mu\varepsilon_y^*\varepsilon_y + \lambda\Delta^*\varepsilon_y\right) + (2\mu\varepsilon_z^*\varepsilon_z + \lambda\Delta^*\varepsilon_z) + \mu(\gamma_{yz}^*\varepsilon_{yz} \\ &+ \gamma_{zz}^*\varepsilon_{zx} + \gamma_{xy}^*\varepsilon_{xy})\big]\end{aligned}$$

Where, $\varepsilon_x^* = \int_0^t R(t-\tau)\varepsilon_x(\tau)d\tau, \ldots,\quad \gamma_{yz}^* = \int_0^t R(t-\tau)\gamma_{yz}(\tau)d\tau,\ \ \Delta^* = \int_0^t R(t-\tau)\Delta(\tau)d\tau$

The stationary condition of the functional on the basic of (11) with the above boundary and initial conditions becomes equivalent to the direct formulation of the problem [10]. Now, using the basic functions to determine the displacements in a continuum system through its nodal values, we can obtain the following system of equations of the finite element method in a dynamic formulation:

$$M\ddot{\vec{Z}}(t) + K(1 - R^*)\vec{Z}(t) = \vec{P}(t) \quad (13)$$

Where, $R^*\vec{Z}(t) = \int_0^t R(t-\tau)\vec{Z}(\tau)d\tau$ and M, K is the matrix of masses and stiffness of a finite element system, $\vec{Z}(t), \vec{P}(t)$ - vectors of movement of nodes and external forces.

Depending on the applied forces, the vector of nodal forces may be different, for example, if seismic vibrations are considered, then $\vec{P}(t) = M\ddot{\vec{Z}}(t)_{\Gamma P}$, if vibrational harmonic loads, then $\vec{P}(t) = p_0 sin\theta t$, where $\ddot{\vec{Z}}(t)_{\Gamma P}$ - is the known acceleration of the ground in the form of an accelerogram of earthquakes, p_0, θ - is the amplitude value and frequency of the external harmonic load.

When considering the oscillations of the system from the standpoint of the well-known seism dynamic theory of underground structures, the dynamic equations of motion of a finite element system acquire a different form. It takes into account the existence of relative movements between the ground and the structure. Interaction forces appear, and the underground structure is in motion under the influence of these forces.

$$M\ddot{\tilde{\vec{Z}}}(t) + K(1 - R^*)\tilde{\vec{Z}}(t) + \bar{K}\tilde{\vec{Z}}(t) = \vec{P}(t) \quad (14)$$

Where $\bar{K}$ the matrix of interaction of the structure with the ground is, $\tilde{\vec{Z}}(t), \ddot{\tilde{\vec{Z}}}(t)$ is the vector of relative displacements and accelerations of the underground structure.

If external damping is taken into account, the equation can be written in the form

$$M\ddot{\tilde{\vec{Z}}}(t) + K(1 - R^*)\tilde{\vec{Z}}(t) + C\check{Z} + \bar{K}\tilde{\vec{Z}}(t) = \vec{P}(t) \quad (15)$$

Here C is a diagonal damping matrix, including coefficients (attenuation modules – x (sec)) external damping. Zeroing out $\bar{\varepsilon} = 0$ and the right part in equations (13-14) and assuming that there are no external forces, we come to the equation of free linear oscillations of the system.

Assuming that the solution has the form

$$\vec{Z} = \vec{\delta} \cdot \cos(\omega t)$$

It can be shown [24] that these above matrix equations have a solution if the following condition holds

$$(K - \omega^2 M)\vec{\delta} = 0 \quad (16)$$

Where $\omega^2, \vec{\delta}$ are the frequencies and the vector of the oscillation form of the system. To solve the problem of eigenvalues (15) (determination of frequencies, then the forms of natural oscillations), the Jacobi method was used [11]. Then, according to the spectral theory [12], it is possible to determine the seismic forces corresponding to the i-th mass of the k-th form of the natural oscillations of the system

$$\vec{S}_{ik} = AK_1K_2K_\psi\vec{Q}_i\beta_k(t)\eta_{ik} \quad (17)$$

Here A is a coefficient that takes into account the calculated seismicity of the construction site, K_1=0.25 is a coefficient that takes into account the permissible damage to the lining, K_2 is a coefficient that takes into account the structural solutions of tunnels (in the absence of data equal to one), K_ψ is a coefficient that takes into account the dissipative characteristics of the structure (in the absence of data equal to one), $\beta_k(t)\eta_{ik}$ - is a coefficient the dynamism corresponding to the k-th form of natural oscillations of the system and the coefficient depending on the k-th form of oscillations of the system and the location of the load.

3. Numerical results and discussion

The problem of calculating reinforced concrete structures of vaulted backfill tunnels of the metro is very complex, but, given the prospects and advantages of structures of this type, at the same time, it is an urgent task. For an objective assessment of the design (fig. 2.) it is necessary to conduct research work in which the model of the system is loaded with seismic loads.

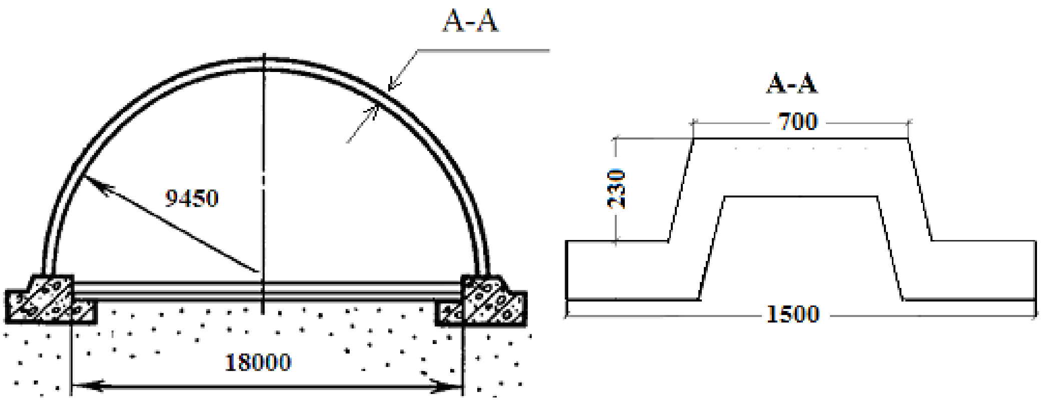


Fig. 2. Cross-section diagram of the arched subway tunnel, (mm)

In this case, the results of numerical studies of the behavior of vaulted structures under the action of ground filling can be used to determine the nature and law of the distribution of internal forces in the cross sections of the arch, modeling the construction of tunnel lining. The span of the reinforced concrete arch is 19,950 m, the boom of the lift in the key is 9,450 m. The vault is covered with backfill soil 2,0 m high from its surface with a volume weight of γ = 0,019 MN/m^3, 0,25 m thick there is an asphalt concrete road surface with a volume weight of γ = 0,024 MN/m^3. The vault is made of concrete of strength class B27,5, the modulus of elasticity of concrete E_b = 31,5 * 10^3 MPa, the volume weight of concrete y_b = 0,025 MN/m^3. At the same time, we introduce the following parameters for calculation, where F =0.3629469 m^2 is the cross–sectional area and J=0,009763039m^4 is the moment of inertia of the cross section of the tunnel structure. The soil surrounding the tunnel has sedimentary characteristics. The circumference of the semicircle is from 1 to 29 knots, starting from the lower part of the arch to the central part of the tunnel.

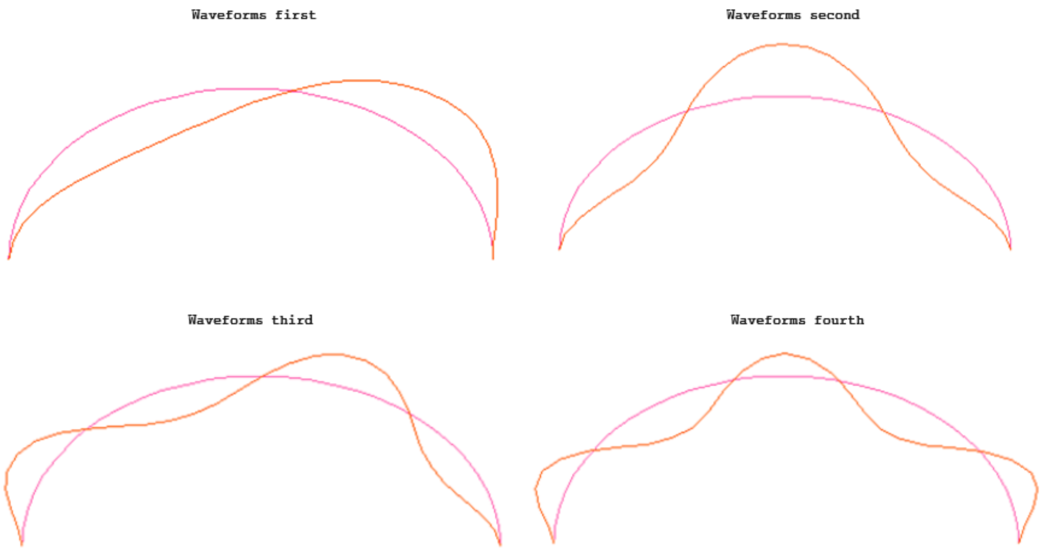


Fig. 3. Waveforms for frequencies:
1 – ω_1=5,73 rad/sec, 2 - ω_2=16,54 rad/sec, 3 - ω_3=17,36 rad/sec, 4 – ω_4=20,35 rad/sec

Here, in all three cases, the maximum stresses are horizontal, which decreases as they approach the center of the tunnel. Vertical stresses, on the contrary, increase as they approach the center of the tunnel, but tangential stresses change the sign at the same time. Further, on the basis of the given stresses, diagrams of bending moments and longitudinal forces are constructed for all cases of loading of the tunnel structure, which are shown in Figures 4-6. At the same time, the maximum values of moments arise in the lock of the vault and longitudinal forces in the lower part of the vault[13].

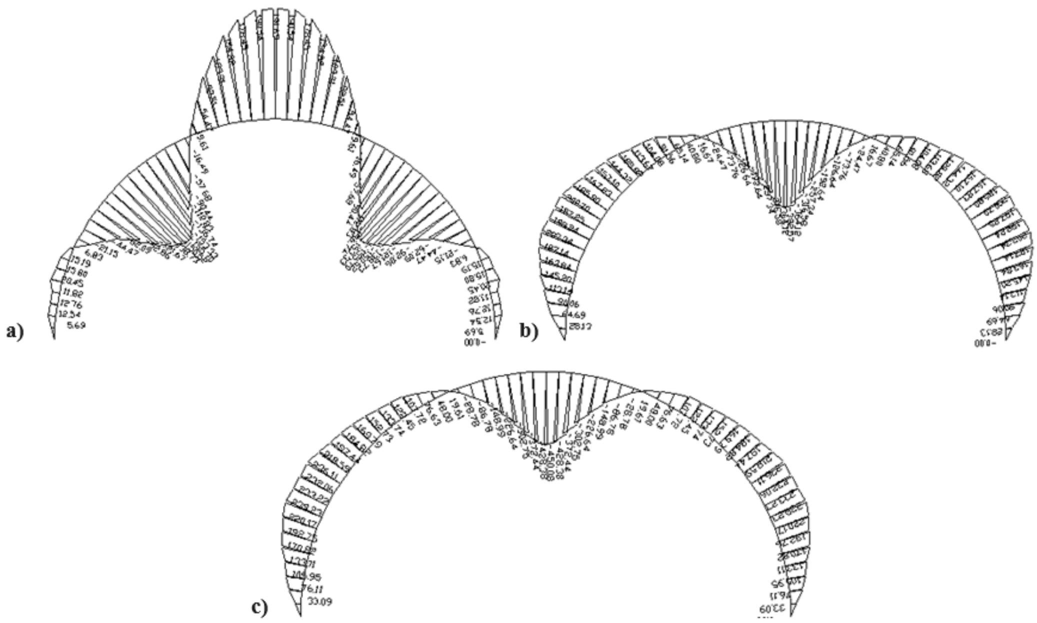


Fig. 4. Bending moment diagrams, (kNm)

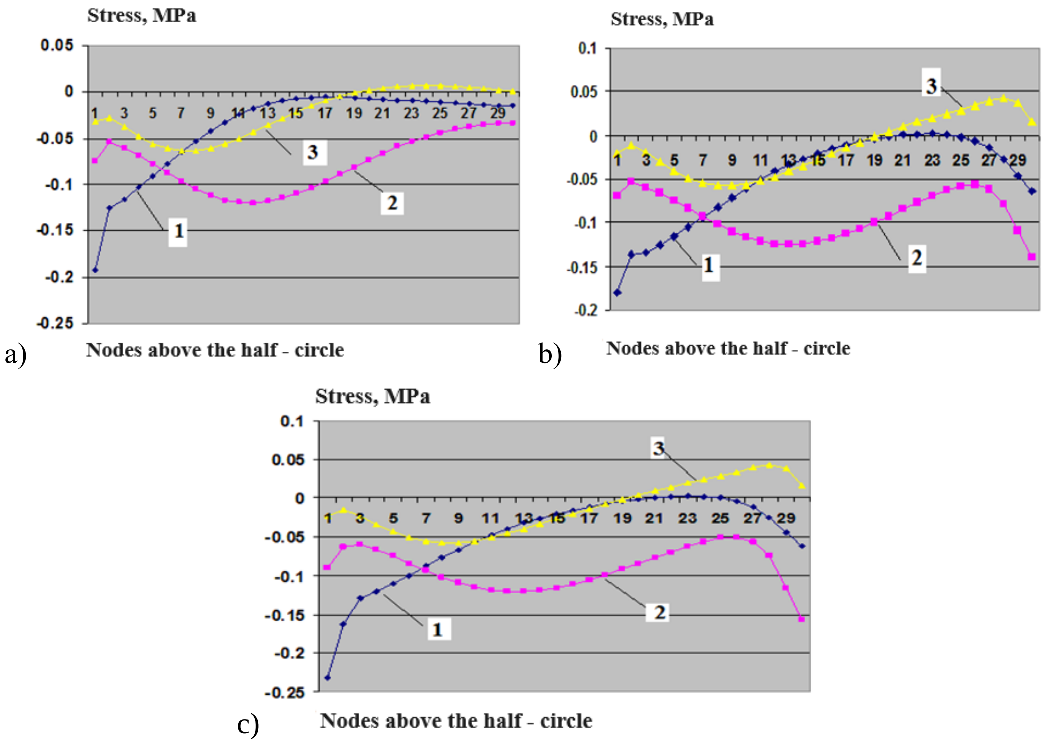


Fig. 5. Stress distribution σ_x (1), σ_y (2), τ_{xy} (3), MPa.

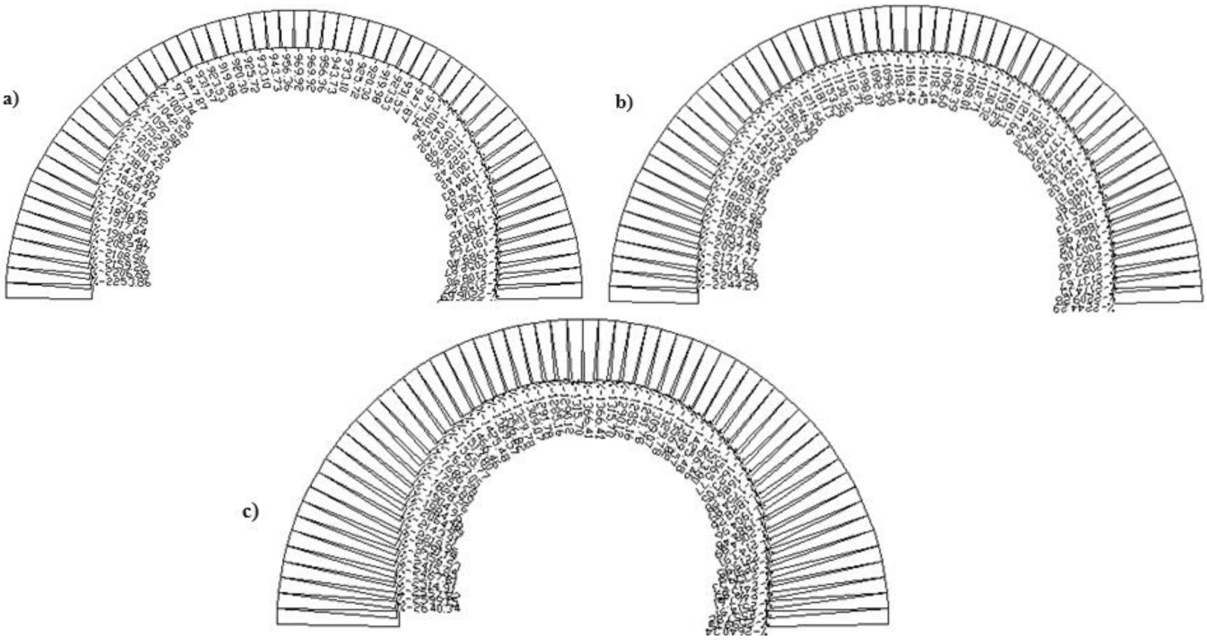


Fig. 6. Plots of longitudinal forces, (kN)

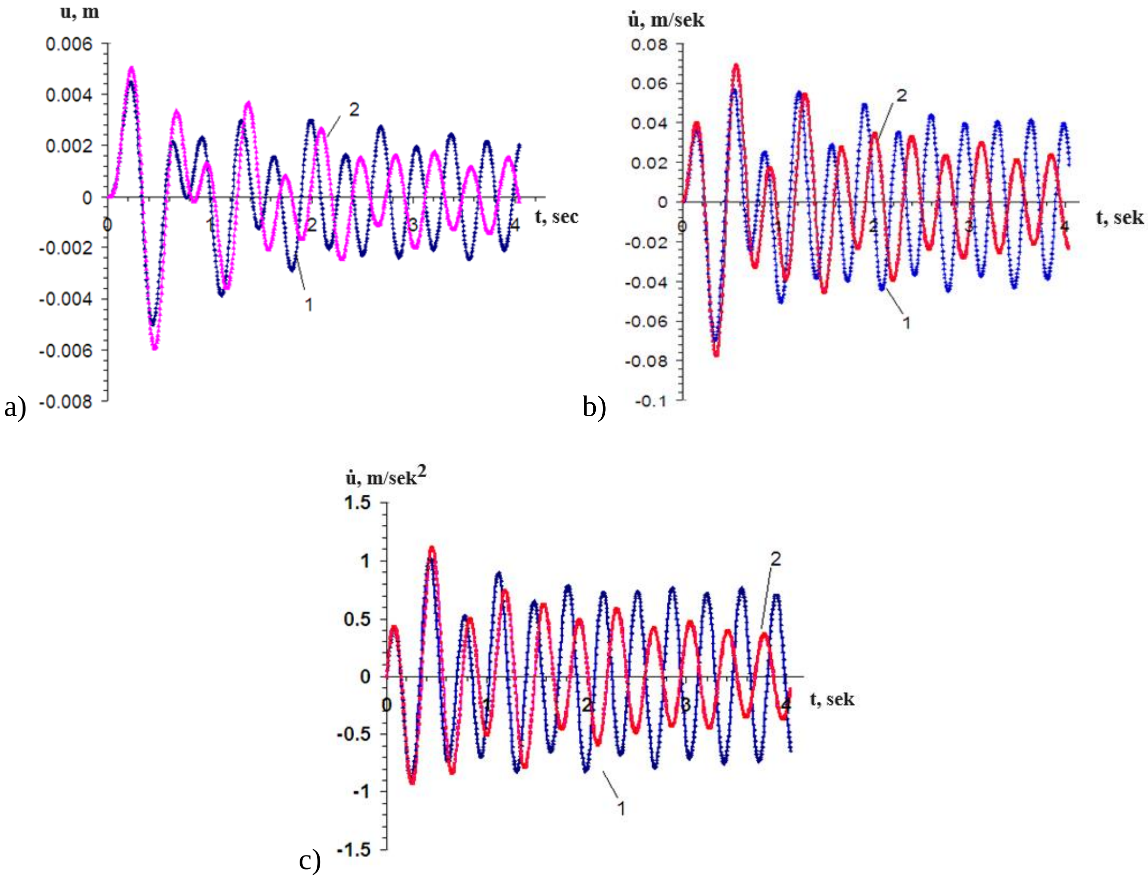
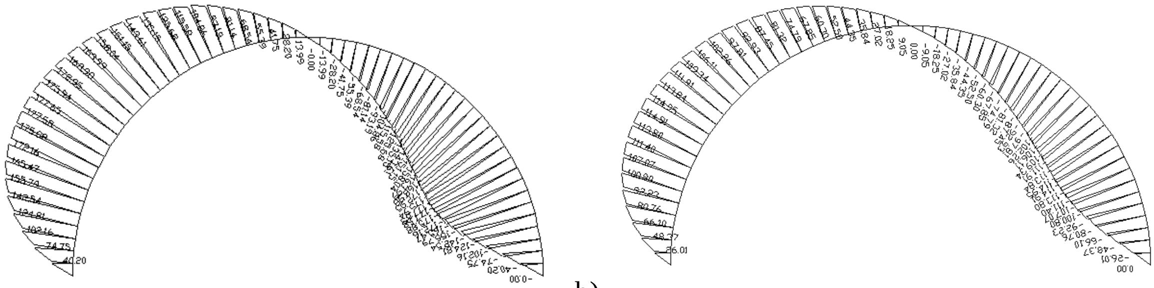
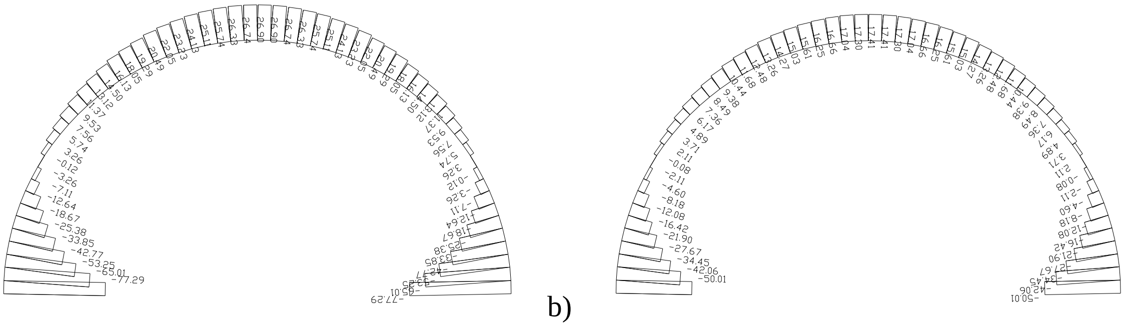


Fig. 7. Change in maximum horizontal values: a - displacement, b - speed, c - acceleration, 1 - without taking into account viscosity, 2 - taking into account viscosity



a) b)

Fig. 8. Diagrams of bending moments, (kNm): a - without taking into account the damping coefficient, b - taking into account the damping coefficient



a) b)

Fig. 9. Diagrams of longitudinal forces, (kN): a - without taking into account the damping coefficient, b) taking into account the damping coefficient

The behavior of a vaulted tunnel structure under dynamic influence is of interest. At the same time, the attached mass of soil above the tunnel was taken into account, since the structure is being erected by an open method of work. Figure 7 shows the oscillation patterns of the structure for 4 natural frequencies. The lowest frequency falls to the first form of oscillation. The specified dynamic (seismic) load is taken in the form of in the horizontal direction.

At the same time, accepted: $\ddot{\vec{Z}}$ =5,2·10-3m, Θ=10 sec.-1, ε0=0,8 sec.-1, time step dt=0.05sec. As the relaxation core, rheological parameters and [8] $\bar{\varepsilon}$=0,01594, α=0,1, β=0,00000011.

With horizontal seismic impact of ground movement, the upper part of the arch receives maximum displacement. When the viscosity is taken into account, the fluctuations fade over time. With an elastic and elastic-viscous model, the absolute displacements, velocity and accelerations of the upper point of the arch are shown in Fig. 7 Taking into account the viscosity, the displacements were reduced by 15-20%. Figures 8 – 9 show diagrams of bending moments and longitudinal forces for two cases as indicated in these figures for the moment of time t = 3.2 sec. Analysis of solutions the seismic load makes a significant contribution to the increase in deflection and internal forces in the sections of the tunnel lining.

4. Conclusion

The structure, which is used for the construction of a metro station at the junctions of the crossing lines, consists of two structural elements: a thin-walled shell, and a flat tray element. Unlike the dome covering, the flow element together with the support ring forms a powerful plate structure that perceives the expansion of the shell. Here, you can see that the maximum bending moments and longitudinal forces are obtained in the heel of the station, at the junction of the tray with the dome. Radial moments are obtained more than annular ones. After a sharp jump, approaching the center, the radial and annular moments decrease. The various peaks existing in the accelerogram with different periods lead to a complex interaction of the lining structure with the ground.

References

[1] Miralimov, M., Normurodov, S., Akhmadjonov, M., & Karshiboev, A. (2021). Numerical approach for structural analysis of Metro tunnel station. In E3S Web of Conferences (Vol. 264, p. 02054). EDP Sciences.
[2] Bulat A. F. Applied Mechanics of elastic-hereditary media. – K.:Science. Dumka, 2012. -614 p.
[3] Dorman I. Y. Seismic transport tunnel. M.: Stroyizdat, 2000, p. 307
[4] B. Onouye and K. Kane. Statics and Strength of Materials for Architecture and Building Construction. 4-th edition, University of Washington, New Jersey, USA, 2016, p. 589
[5] Frolov Y.S. and Promising development of transport infrastructure Spain (based on the materials of a practical conference). Zhl "Metro and Tunnel," No. 3. Moscow, 2012, S.11-19
[6] Broere W. Influence of excess pore pressures on the stability of the tunnel face. In (Re) Claiming the underground space, pp. 759-765, Routledge (2003).
[7] Barkanov E. Introduction to the finite element method. Institute of Materials and Structures Faculty of Civil Engineering Riga Technical University, pp.1-70, (2001).
[8] Miralimov, M. X., & Normurodov, S. U. (2019). Construction features of transport tunnels in the mountain areas of uzbekistan. Journal of Tashkent Institute of Railway Engineers, 15(3), 26-35.
[9] Ulugbekovich, N. S. (2022). Stress-strain state of the construction of a subway tunnel under seismic impacts. World scientific research journal, 8(1), 3-11.
[10] Khamitovich, M. M., Ulugbekovich, N. S., & Shomansur o'g'li, T. S. (2021). Calculation technique for typical circular tunnel linings with taking into account the interaction of the structure with the ground. Galaxy International Interdisciplinary Research Journal, 9(6), 362-368.
[11] Construction of the second stage of the Yunusabad metro line on the section from Shakhristan station to Turkiston station. Section 8. Project of works. Tashkent Metroproekt LLC, p. 120, (2017)
[12] Guglielmetti V., Grasso P., Mahtab A., and Xu S. (Eds.). Mechanized tunnelling in urban areas: design methodology and construction control. Taylor and Francis. (2008).
[13] Miralimov, M., & Shakhboz, N. (2023). Ground behaviour and settlements analysis on tunnelling of shallow-buried metro in Tashkent city. In E3S Web of Conferences (Vol. 401, p. 01062). EDP Sciences.
[14] Salixanov, S. S., Zokirov, F. Z., Xakimova, Y. T., & Ismailova, G. B. (2023). The effect of increasing loads on foundations of operating bridges. In E3S Web of Conferences (Vol. 401, p. 01080). EDP Sciences.
[15] Raxmanov, U. S., & Ismailova, G. B. (2020). Calculation of seismic resistance of reinforced concrete railway spans without prestressing reinforcement. Journal of Tashkent Institute of Railway Engineers, 16(3), 164-169.
[16] Shermuxamedov, U., Shaumarov, S., & Uzdin, A. (2021). Use of seismic insulation for seismic protection of railway bridges. In E3S Web of Conferences (Vol. 264, p. 02001). EDP Sciences.
[17] U.Z. Shermukhamedov, A.B. Karimova, B.S. Zakirov. Calculation of Continuous Reinforced Concrete Bridges And Overpasses Inseismically Hazardous Areas // V International Scientific Conference "Construction Mechanics, Hydraulics and Water Resources Engineering" (CONMECHYDRO - 2023), E3S Web of Conf. Volume 401, 2023. https://doi.org/10.1051/e3sconf/202340101078
[18] Turgunbayeva, J. R., Ismoilova, G. B., & Juraev, K. M. (2023). Investigation of mechanical activation of steelmaking slag and obtaining fine filler. In E3S Web of Conferences (Vol. 401, p. 02039). EDP Sciences.
[19] AZ Khasanov, UZ Shermukhamedov, AR Abdullayev. The method for determining the designed resistance of soils with considering the theory of soil strength proposed by the authors // Smart Geotechnics for Smart Societies. CRC Press. - p. 464-467.