Strength assessment of tunnel lining considering soil conditions

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Abstract. At present, Tashkent city operates low-depth frame-type underground railway stations. There are currently no single-arch deep metro stations designed. As the city's economic potential and population density continue to grow, there is a pressing need to create deep stations. Consequently, it is necessary to calculate the strength of the lining while considering the surrounding soil. The authors propose a method for calculating the lining of a structure in accordance with the design standards for deep underground structures, which makes it possible to determine the strength of the most vulnerable areas. An example of calculating the lining of a single-arch deep “Beruniy-Karakamysh” metro station is given; it may be applicable in design. The calculation results for a specific station were presented to Tashkent Metroproject LLC for further use in the design of similar structures.

1 Introduction

As cities continue to grow rapidly, there is an increasing demand for efficient transport systems. It has become evident that surface transport alone cannot fully meet the needs of passenger transportation. In today's world, the solution to this challenge lies in developing underground transport networks. When designing and planning the metro system, it is crucial to consider that the underground station is the most complex and expensive part of the entire structure.

Currently, the main goal in underground construction is to reduce material usage in station structures by enhancing the structures and construction technology. The existing calculation methods for assessing the strength of linings for underground metro structures do not take into account the soil conditions of the Tashkent city, the features of the structures' static operation, and the various conditions and factors influencing its stress state. This study introduces a method for calculating an underground structure, such as a single-arch deep structure.

Numerous studies were devoted to determining the loads on the support of capital workings; a review of these studies is presented in [6-10].

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Currently, two key directions of theoretical and scientific research consider the interaction of the lining of an underground structure with the surrounding medium.

Static calculation of the lining of an underground structure is the first direction; loads are preset, including reactive rock resistance and active rock pressure, determined by analytical methods obtained from field observations and based on regulatory documents. It should be noted that reactive rock resistance acts on the lining in the point where the movement of the structure is directed toward the rock. It is calculated considering the compatibility of the lining-soil system movements. The disadvantage of this technique is that it creates a solution to an additional problem that requires determining the value of active rock pressure.

In [11-13], M.M. Protodyakonov, P.M. Tsimbarevich, L.M. Shevyakov conducted research to determine the main load when calculating the support for the active load arising from rock pressure.

Subsequent studies in this direction showed that underground structures laid in clays form a soil arch, which led to the emergence of the theory of arch-forming regarding small spans [14].

The study showed that the value of rock pressure equal to the weight of a rock column is not suitable for calculating all structural types of underground metro stations [15-16].

2 Materials and methods

Janssen and Ketter [17] considered a model of a descending rock column. The volume of rocks that constitute the load on the lining is equal to the rock column above the excavation to the ground surface. The pressure on the lining is determined as a rock column above an excavation with a width of 2a, located at depth H in an ideally cohesive medium (Fig. 1).

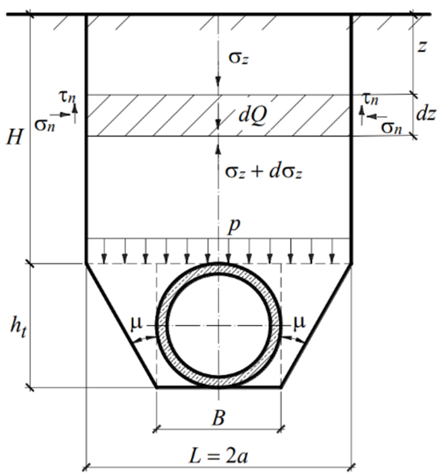

Fig. 1. Jansen-Ketter scheme [17]

The equilibrium of an elementary layer of unit length with a thickness of *dz* at an arbitrary depth *z* is expressed as:

$$\sum z = dQ + \sigma_z 2a - (\sigma_z + d\sigma_z)2a - 2\tau dz = 0 \tag{1}$$

where $dQ = 2a\gamma dz$ – is the layer weight; $\sigma_z -$ is the sought-for pressure on the layer; $\tau = (f\sigma_x + k) -$ is the shear stress at the contact of the rock column with the stationary massif; $\sigma_x = \lambda_c \sigma_z -$ is the horizontal pressure on the massif; $\lambda_c -$ is the coefficient of lateral expansion in the arch for granular media:

$$\lambda_c = \frac{1-sin\rho}{(1+sin\rho)} \tag{2}$$

Substituting this into the previous equation, we obtain a first order differential equation with separable variables:

$$\frac{d\sigma_z}{1-\frac{\lambda_c f c}{\gamma a-k}\sigma_z} = \frac{\gamma a-k}{a}dz \tag{3}$$

Direct integration considering the obvious boundary condition for z=0 and $\sigma_z = 0$ gives the following solution:

$$\sigma_z = \frac{\gamma a-k}{\lambda_c f_c}\left[1 - \exp\left(-\lambda_c f_c \frac{z}{a}\right]\right. \tag{4}$$

which allows us to determine the vertical pressure in the column at an arbitrary depth *z*. The pressure on the excavation support $\sigma_z = q$ for z=H is:

$$q = \frac{\lambda a-k}{\lambda_c f}\left[1 - \exp\left(-\lambda_c f_c \frac{H}{a}\right)\right] \tag{5}$$

The experience of underground construction work and statistical analysis of collapses, of the so-called “arches of natural equilibrium (pressure arches)”, showed that this phenomenon corresponds to the formation of an arch-like zone of collapse (a pressure arch), which puts pressure on the support. Methods for determining the size of arches are currently a completely unsolved practical problem in determining the possible load on the support. W. Ritter in [18] defined the value of rock pressure as the difference between the weight of the pressure arch and the strength of the “pressure arch–non-displaced rock mass contact”. It is assumed that the soil strength along the contour of the arch at the limit stage of the operation is determined only by normal stresses, which reach soil strength R_t in uniaxial tension along the entire boundary of the arch (Fig. 2).

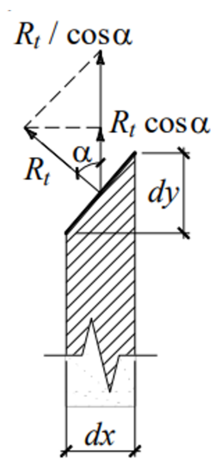

Fig. 2. Scheme for determining rock pressure according to W. Ritter’s theory [18]

The equilibrium condition according to W. Ritter has the following form:

$$P = \gamma\int_0^L ydy - R_t\int_0^L \frac{1}{\cos\alpha}ds$$

where *P* – is the pressure on the support; *L* – is the mining excavation span; γ – is the specific gravity of soil; α – is the angle of inclination of the tangent to the curve of the arch at the point under consideration; *ds* – is the differential of the arc of the curve limiting the excavated part.

W. Ritter's idea is to find, using the calculus of variations, such an equation for curve y(x), limiting the pressure arch, at which the pressure on the support P will be maximum. Note that if the projection is defined as $R_t \cdot \cos\alpha$, then the maximum of function (5) will be located on $+\infty$. In other words, the higher the pressure arch, the greater the rock pressure.

R. Fenner [19] considered an alternative approach for a massif with internal friction only. The relationship between stresses in the plastic zone around the shaft is determined by the Coulomb-Mohr criterion (with cohesion equal to 0). It is assumed that the load on the support is formed due to both rock pressure in the zone of inelastic strains and the elastic interaction of the rock mass. The load on the support is calculated using the following formula:

$$P = \gamma H(1 - sin\varphi)(\frac{r_0}{r_e})^{\alpha} \tag{6}$$

where *e* - is the radius of the plastic strain zone.

Current regulatory documents [20-21] provide for determining the load from rock pressure according to two schemes. According to the first scheme, in the process of opening the excavation cross-section, an arch of natural equilibrium (a pressure arch) is formed above the excavation. The load from rock pressure q is determined by the height of the pressure arch h1, according to the theory proposed by M.M. Protodyakonov [11].

$$h = \frac{L}{2f} \tag{7}$$

where *L* – is the span of the pressure arch; *f* – is the strength coefficient by M.M. Protodyakonov’s scale.
The load from rock pressure p in this case is determined by the weight of the arch, that is:

$$P = \gamma h \tag{8}$$

where γ – is the specific gravity of soil in the arch.

Most of the publications that study the issues of determining the load from rock pressure are devoted to the problems of determining the stress-strain state of the soil mass around the excavation according to design schemes.

3 Results and discussion

In this article, a lining option for the projected metro station in the Beruniy - Karakamysh direction was developed. As a result of engineering and geological surveys, it was revealed that at a depth of 23 meters, there are hard clays favorable for construction, they have sufficient natural strength f = 0.8 m, are a hydraulic stop, have a relatively high density and have elastic resistance, which is most favorable for the construction of subway stations. therefore, the station could be deep. In this case, the soil can keep the massif from collapsing into the excavation. This means that the area of expected collapse highlighted by the contour of the arch is held by normal and shear stresses acting along the contour of the arch. Generally, deep metro stations come in three types: pylon, column, and single-arched. A single-arch station allows us to mechanize the excavation of soil in the face and carry out transit shield tunneling of stage tunnels along the line. This design of the station allows all the structures of the station complex to be placed under a single arch.

Therefore, for further development, we accept a single-arch station made of precast reinforced concrete with triangular supports (Fig. 3).

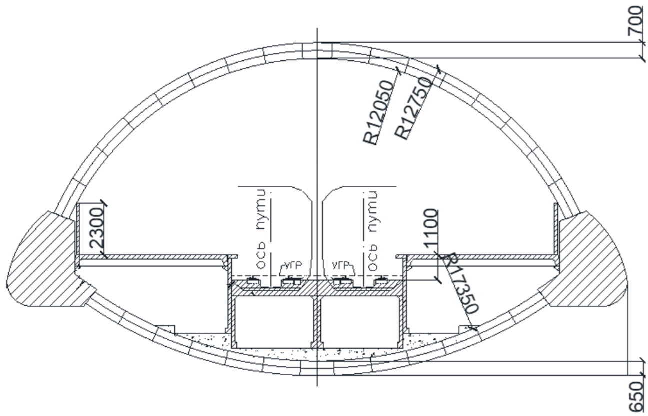

Fig. 3. General view of a designed single-arch station

The main task of the static calculation of structures is to assess their load-bearing capacity. This assessment is based on the calculation of linings on limit states. To do this, it is necessary to determine the stress-strain state of the structure, that is, to calculate internal forces and strains in the lining. The obtained values of these forces are compared with the maximum permissible ones, i.e. the strength of the most stressed sections of the lining is assessed.

Calculation of the lining should be conducted for the most unfavorable, but realistic combination of loads and impacts that can act simultaneously during the construction or operation of the structure. In this case, basic and special combinations of loads should be considered.

The rated vertical load from rock pressure is assumed equal to the load from the weight of the entire thickness of the soil above the station [20-21].

The rated vertical and horizontal loads are, respectively:

$$q^{\text{н}} = \sum_{i=1}^{n} \gamma_i h_i \tag{9}$$

$$p^{\text{н}} = \sum_{i=1}^{n} \gamma_i h_i \cdot \text{tg}^2(45° - \varphi/2) \tag{10}$$

where:

$\gamma_i -$ is the rated specific gravity of soil of the corresponding bedding layer, t/m^3;

h_i is the thickness of the corresponding bedding layer, m; φ is the angle of internal friction of the soil mass within the section of the tunnel lining.

The calculation is performed for the width of the lining ring $b_к$=0.5 m. When calculating load-bearing structures and foundations of tunnel structures, the safety factor for responsibility should be 1.1 as is for structures I of the high-level responsibility. Safety factors for constant loads when calculating lining structures for loss of bearing capacity are taken according to regulatory documents.

Design vertical load q_p per ring $b_к$=0,5m is:

$$q^{\text{p}} = q^{\text{н}} * K_{\text{о}} * K_{\text{н}}, \text{t/mm}^2 \tag{11}$$

The estimated horizontal load, p^p per ring $b_к$ =0.5m is:

$$p^{\text{p}} = p^{\text{н}} * K_{\text{о}} * K_{\text{н}}, \ \ \text{t/m}^2 \tag{12}$$

Definition of auxiliary quantities:

Elasticity modulus is:

$$E_{пр}=\alpha * E\frac{t}{m^2} \tag{13}$$

where *E* - is the elasticity modulus of concrete class; α is a reduction factor that considers the compliance of joints.

The coefficient of elastic resistance (back pressure) at the specific value and width of the station arch is:

$$K = \frac{2\ к_0}{B}, t/m^3 \tag{14}$$

Cross-sectional area is:

$$S=b_к*h,\ m^2 \tag{15}$$

where $b_к$ - is the width of the lining ring;

h – is the block height.

Moment of inertia is:

$$I = \frac{h_б^3 * b_к}{12}, m^4 \tag{16}$$

The calculation of the lining is performed using the "RK-6" software package (characteristics for the structure and the surrounding medium were provided by Tashkent Metroproject LLC), the calculation scheme is shown in Figure 4.

A single-arch deep metro station on the projected Beruniy – Karakamysh line in the city of Tashkent was used as an example for the calculation. The physical and mechanical characteristics of the surrounding soil mass are γ = 2.1 t/m^3, f = 0.8, Eo ≤ 5 MPa.

As a result of calculations, the diagram of the bending moment was determined (Fig. 5).

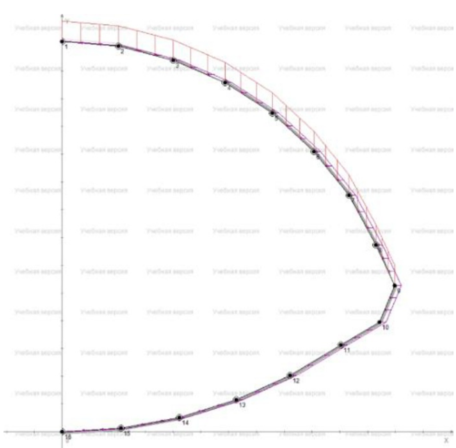

Fig. 4. Design scheme

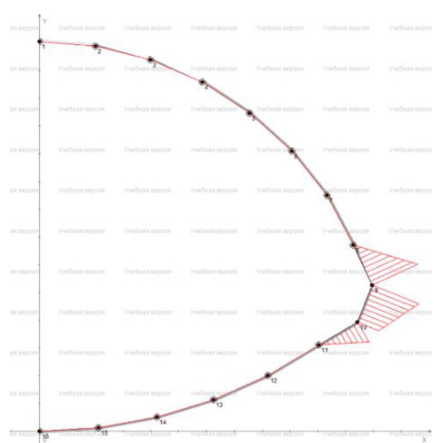

Fig. 5. Moment diagram of station lining

From this diagram, it is clear that the maximum value of the bending moment occurring in the lining of the tunnel under study is M_{max} = 25.64 tm.

A diagram of normal and shear forces was plotted, see Figures 6 and 7.

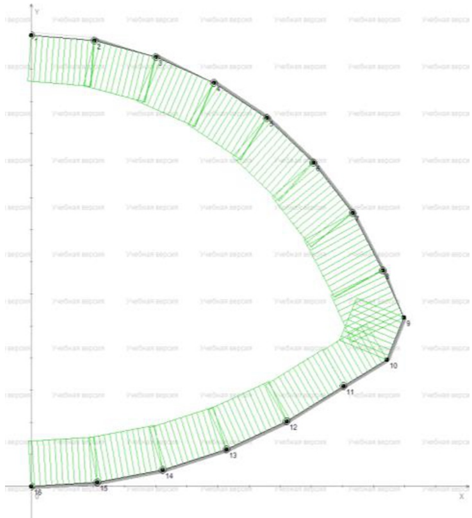

Fig. 6. Diagram of normal forces (Nmax =256,06t)

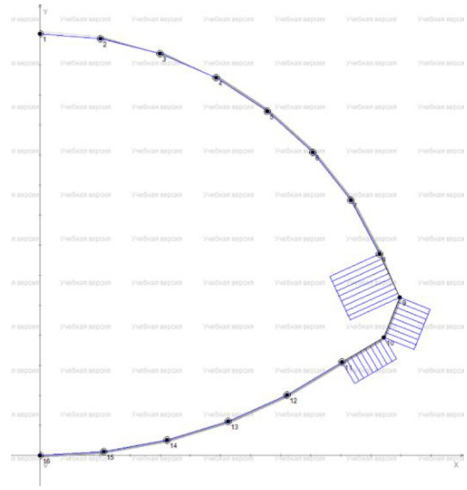

Fig. 7. Diagram of transverse forces, t

From the data obtained from calculations, it can be seen that N_{max} = 256.06 t.

A diagram of horizontal displacements of the station lining was constructed, shown in Figure 8.

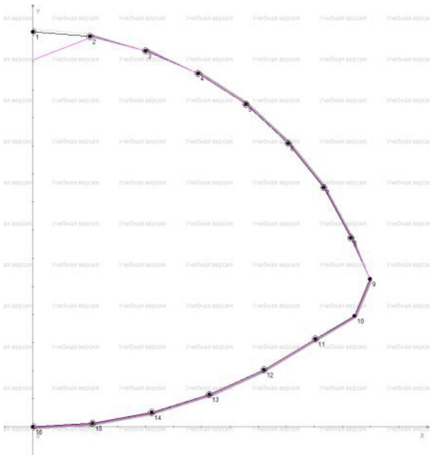

Fig. 8. Diagram of horizontal displacements (Umax = 0.296549 m)

The maximum value of the bending moment is achieved in nodes No. 8, 9, in the junction of the lining arch with the column. Large bending moment values are related to the absence of a hinged joint, that is, this node is rigid.

Having calculated the loads acting on the lining, it is necessary to check the strength of soil from the support of the single-arch station.

The strength of reinforced concrete supports is checked as:

$$\sigma = \frac{N}{A}, MPa \tag{17}$$

The compressive strength of soil is: 2.4 MPa.

Thus, the strength of the structure is ensured.

The design scheme of the lining was calculated using the RK-6 programs. Based on the calculation results obtained, it can be concluded that the design structures of the station have a safety margin, and, therefore, the designed station could be located in the given engineering and geological conditions and has sufficient operational reliability.

4 Conclusion

The analysis of scientific and practical research has revealed that current methods for calculating linings do not take into account the characteristics of the surrounding soil or present them in the form of an interaction law. The strength calculation of the lining of the designed single-arch deep metro station, considering the surrounding soil, has shown that the maximum values of the bending moment occur at nodes No. 8 and 9, specifically at the junction of the lining arch with the column. This method also helps in determining the maximum values of normal shear forces and transverse horizontal displacements. This calculation method allows us to identify the most critical areas of the entire structure, where stress concentrations may occur, potentially leading to undesired effects such as loss of structural strength under given loads.

Today, the priority direction is to improve structural and technical solutions for structure calculations, which require further research into the stress-strain state of the lining during the construction of various types of stations.

References

1. N.M. Bykova, Internet journal – Scientific Studies-Novosibirsk (2008).
2. C. Raupov, U. Shermukhamedov, A. Karimova, E3S Web of Conferences **264,** 02015 (2021).
3. A.Z. Khasanov, U.Z. Shermukhamedov, A.R. Abdullaev, Smart Geotechnics for Smart Societies. CRC Press, pp. 464-467.
4. T.R. Rashidov, Vibrations of structures interacting with soil (Tashkent, Fan, 1975).
5. T.R. Rashidov, Uzb. J. Problems of mechanics pp. 62-66. (2017).
6. I.V. Baklashov, K.V. Ruppeneit, Strength of unsupported mine workings (Moscow, Nedra, 1965).
7. I.V. Baklashov, B.A. Kartozia, Mechanics of rocks (Moscow, Nedra, 1975).
8. I.V. Baklashov, B.A. Kartozia, Mechanics of underground structures and support structures (Moscow, Nedra, 1984).
9. A.N. Drankovsky, A.B. Fadeev, Underground structures in industrial construction (Kazan University, 1993).
10. V.G. Khrapov, E.A. Demeshko, S.N. Naumov et al., Tunnels and subways (Moscow, Transport, 1989).
11. M.M. Protodyakonov, Rock pressure on mine support (GONGS, 1933).
12. P.M. Tsimbarevich, Mining Journal **9** (1933).
13. L.D. Shevyakov, Mining Journal **7** (1931).
14. Guide to subway design. State Design and Survey Institute "Metrogiprotrans", "Transstroy" (Moscow, 1992).
15. Scientific and technical report. Full-scale studies of the static operation of load-bearing structures of the Mayakovskaya metro station (CNIIS, Leningrad, 1968).

16. P.V. Stepanov, S.G. Mandrikov, G.A. Skobennikov, Conduct research on the design of a column-mounted metro station in Leningrad from prefabricated reinforced concrete with compression of elements onto rock and develop recommendations. Scientific and technical report of CNIIS, L.-M., 1984.
17. F. Ketter, Bestimmung des Druckes an gekrümmten Gleitflachen, eine Aufgabe aus der Zehre von Erddruck, Berlin, 1903.
18. W. Ritter, Die Statik der Tunnelgewolde. Berlin, 1879.
19. R. Fenner, Research of rock pressure. Issues of the theory of rock pressure (Moscow, Gosgortekhizdat, 1961).
20. SP 120.13330.2011. Subways. Updated version of SNiP 02/32/2003. Moscow, Ministry of Regional Development of Russia, 2011.
21. SP 122.13330.2012. Railway and road tunnels. Updated edition of SNiP 32-04-97. Moscow, Ministry of Regional Development of Russia, 2012.