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City utility tunneling designing method based on "support-rock mass" interaction

Abstract: At present time, megacities face acute problem of construction of tunnels for laying utility networks. In such large cities as Hanoi, Ho Chi Minh, there are still no solutions to the problem of laying technical pipelines, power cables, water supply and sewage pipelines. The paper describes the method of utility tunnel designing, taking into account rock composition and restricted construction conditions for each specific area, based on the use of small tunnel-boring machine. Using this method ensures reliability and economic feasibility of the tunnel construction project.
Keywords: rock (soil) mass, tunneling, stress, calculation, theory of elasticity, microtunnelling.

1. Introduction

Microtunnelling is a special case of pipe jacking [5], where remote control of an automated microtunnel boring machine (MTBM) is employed. Excavated soil is removed from the face of the pipe jacking shield or MTBM and transferred to the surface for disposal while the shield or MTBM and the product pipes to be installed are driven through the ground using the force developed by the tunneling facility.

Pipe jacking method is widely used for new pipeline installations [7]. Application areas involve oil & gas, water supply, sewage, communication and electricity pipelines, and pipe-roof projects [9]. Usually jacked pipes are glass Fibre Reinforced plastic Mortar Pipes (FRMP), concrete pipes, clay pipes, cast ductile iron pipes, and steel pipes.

For shallow burial jacked pipes, jacking load will control the cross-sectional design of the pipe, and the soil (loose rock) pressure may be insignificant. However, for deep burial projects, high soil pressure may lead to buckling of pipes [15], then the soil pressure becomes a crucial factor. Soil pressure on jacked pipes also invokes assessing the jacking force [11].

In current practices, soil pressure on jacked pipes is estimated using soil pressure models recommended by Japan Microtunnelling Association (JMA), German standard ATV A 161 (ATV A 161), UK 'Pipe Jacking Association' (PJA), ASCE 27, and Chinese standard GB 50332 (GB 50332) [2, 6, 8, 12, 14].

These soil pressure models are modified from one of Terzaghi arching models (termed Arching model I) [13].

2. Basic theoretical principles

When designing technical tunnels (microtunnels) in zones of loose low-strength soils (rocks) (in the initial stress domain), stress (load) on the tunnel is produced by gravity force, hydrostatic pressure, etc.:

$$\sigma_x^{(0)(0)} = \sigma_y^{(0)(0)} = -\gamma H,\ \sigma_{xy}^{(0)(0)} = 0, \quad (1)$$

where γ – average bulk density of soil, H – depth of burial; "–" sign is adopted in accordance with the rule of elasticity theory, according to which compressive stresses are considered to be negative.

Application of superposition principle allows modeling total stress in rock mass in the vicinity of tunnel as the sum of the initial $\sigma^{(0)(0)}$ and additional $\sigma^{(1)(0)}$ (caused by tunneling) stresses. Thus, we can write:

$$\sigma^{(0)} = \sigma^{(0)(0)} + \sigma^{(1)(0)}, \quad (2)$$

where σ denotes all components of the stress tensor.

The concept of "removable stresses" was introduced by Prof. Dr. I.V. Rodin [20] for simulating tunneling in a prestressed massive rock mass. Using this concept, it is easy to imagine that, when tunneling, the tunnel contour must be free of total normal and tangential stresses. This can be achieved using principle of superimposing stresses in the initial field around the tunneling zone by generation of the same magnitude additional stresses, but opposite in sign (in relation to the pre-existing stresses). Physically, this means that the initial stresses acting on the future tunnel contour must be "lifted".

After installing the tunnel support (lining), which has certain rigidity, redistribution of stresses happens in the surrounding rock mass. At the same time, in the case when the support is installed directly into the face immediately after opening the section of the mine, the removed (additional) stresses acting on the rock mass are completely transferred to it as load (radial pressure). The support acts as an integral part of the unified deformable system "support-rock mass". Figure 1 illustrates the calculation for determining permissible displacements of the rock mass and stresses in the support.

The described problem has an analytical solution presented, for example, in [16, 17]. Following the provisions set out in the above-mentioned papers, let us change the way to solve this problem. Assuming that tunnel is not supported, then the contour L_0 rock mass (area S_0) boundary condition for the total stress in the polar coordinate system, taking into account the expressions (1), (2) can be written as:

$$\sigma_r^{(0)} = \sigma_r^{(1)(0)} + \sigma_r^{(0)(0)} = \sigma_r^{(1)(0)} - \gamma H = 0. \quad (3)$$

Here, the shear stresses are not considered due to the axial symmetry of the problem. Then, for the additional stress acting on unsupported contour, we obtain the following relation:

$$\sigma_r^{(1)(0)} = \gamma H . \quad (4)$$

Further, considering the case when the support is installed, it should be noted that it prevents free deformation of the tunnel contour L_0, creating stress resistance. For convenience, we consider the interaction of elements of a single "support-rock mass" system, as shown in Figure 2. Condition (4), taking into account the support resistance, takes the form:

$$\sigma_r^{(1)(0)} = \gamma H - q_0 = -(q_0 - \gamma H), \quad (5)$$

where q_0 – support resistance.

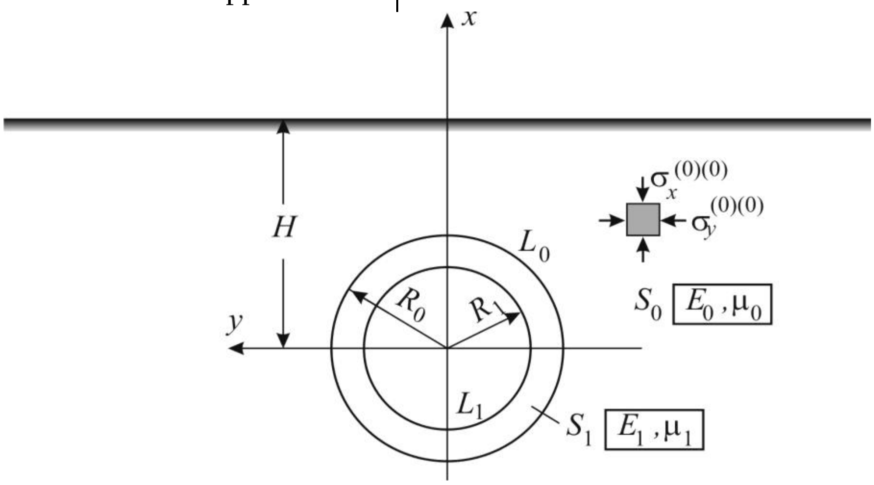

Fig. 1. Illustration of support behaviour

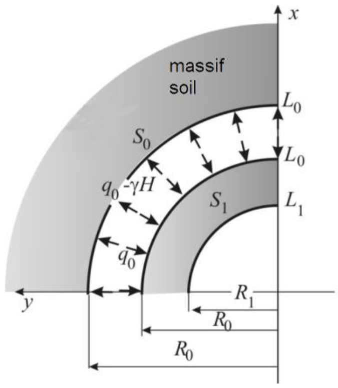

Fig. 2. Illustration of loading single deformable system "support-rock mass"

It is accepted in (5) that normal stresses acting towards zone S_0 (contour L_0) are compressive, i.e. negative. The outer tunnel support (ring S_1) accepting radial compressive stress q_0 simulates surrounding rock pressure on tunnel (area S_0). Neglecting the initial stresses on the support, we can write the expression for the additional radial stresses in the ring S_1 at line L_0 of the contact with rock mass:

$$\sigma_r^{(1)(1)} = -q_0. \qquad (6)$$

In the case where support is mounted directly in the face, the solution of the problem can be obtained using known solution of the Lame problem [21]: taking into account the plane strain thick-walled pipe, simulating support, the condition that the radial displacement at the line of contact between the support and the rock mass due to the action of additional stresses (5), (6), we obtain the equation connecting the resistance value with q_0. Taking into account the direction of stresses, and by applying outer tube radius $r_0 = \infty$ to the Lame formula, at the external pressure $p_0 = 0$, we write the expression for the movement of infinite medium (rock mass) at the contour line L_0:

$$u_0 = -\frac{R_0}{2G_0}(q_0 - \gamma H), \qquad (7)$$

where the notation: "–" sign in the expression (7) indicates that the condition $q_0 \leq \gamma H$ shows that rock mass, located outside the contour line L_0, shifts into the tunnel.

$G_0 = \dfrac{E_0}{2(1+\mu_0)}$ – mechanical characterization of rock, called shear modulus (8).

Equation (7) can be represented in the form $q_0 = f(u_0)$, that is,

$$q_0 = \gamma H\left(1 - \frac{u_0}{R_0}\frac{2G_0}{\gamma H}\right) = \gamma H - u_0 \operatorname{tg}\beta \qquad (9)$$

where $\operatorname{tg}\beta = \dfrac{2G_0}{R_0}$.

In mechanics of underground structures, this formula (9) is called “equation of equilibrium states of rock mass, since it implies that each value of support resistance (pressure on the support) corresponds to certain amount of movement of the tunnel section. Further, applying to the Lame equation outer tube radius $r_0 = R_0$, external pressure $p_0 = q_0$, inner radius $r_1 = R_1$, and internal pressure $p_1 = 0$, we arrive to the second problem of the support plane deformation. We write the expression for the radial displacements of the points on the outer support contour line:

$$u_0 = \frac{R_0}{2G_1}\frac{q_0}{R_0^2 - R_1^2}\left[(1-2\mu_1)R_0^2 + R_1^2\right] \qquad (10)$$

The sign in expression (10) is selected for the same reasons as in formula (7), i.e. if $q_0 > 0$, points of the contour line L_0 move inside the ring (support).

Equation (10) may also be presented in the form $q_0 = f(u_0)$, and, as easily seen, this relationship is linear. We represent it in the form of:

$$q_0 = u_0 \operatorname{tg}\alpha, \qquad (11)$$

where

$$\operatorname{tg}\alpha = \frac{2G_1}{R_0}\frac{R_0^2 - R_1^2}{\left[(1-2\mu_1)R_0^2 + R_1^2\right]} \quad (12)$$

Equations (9) and (11) connect the same parameters q_0 and u_0. We have built the corresponding graphic representations of these dependences together in one drawing, as shown in Figure 3 [1, 10].

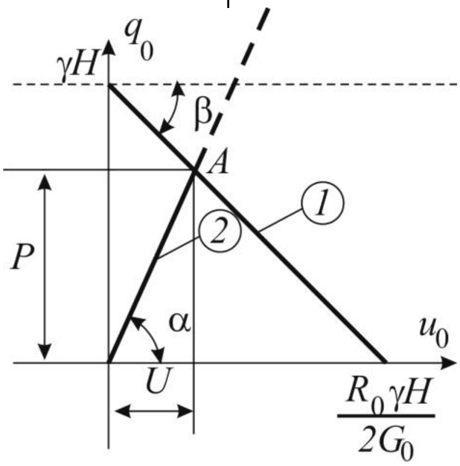

Fig. 3. Determination of equilibrium state of a single deformable system "support-rock mass"

Line *1* in Figure 3 presents the rock mass equilibrium state, line *2* presents the support equilibrium state. Obviously, point *A* at intersection of these two lines will correspond to the equilibrium state of the single deformable "support-rock mass" system. The value of $q_0 = P$ corresponds to the pressure load on the support (support resistance) with given geometrical and mechanical parameters directly at the face, and the value of $u_0 = U$ – displacement rock mass in these geological conditions. The position of point *A* is determined using the combined solution of equations (9) and (11):

$$q_0 = \gamma H \frac{1}{1 + \frac{\operatorname{tg}\beta}{\operatorname{tg}\alpha}} = \gamma H \frac{1 - \frac{R_1^2}{R_0^2}}{\left(1 - \frac{R_1^2}{R_0^2}\right) + \frac{G_0}{G_1}\left[(1-2\mu) + \frac{R_1^2}{R_0^2}\right]}. \quad (13)$$

Additional stresses in the support (equal to the total stresses at the same time due to no initial stresses) are determined by the formulas:

– at the outer contour of the support L_0:

$$\sigma_r^{(1)} = -q_0;\ \sigma_\theta^{(ex)} = -\frac{1 + \frac{R_1^2}{R_0^2}}{1 - \frac{R_1^2}{R_0^2}} q_0; \quad (14)$$

– at the inner contour of the support L_1:

$$\sigma_\theta^{(in)} = -\frac{2}{1 - \frac{R_1^2}{R_0^2}} q_0. \quad (15)$$

The internal forces and bending moments in the support of radial tunnel sections are calculated from structural mechanics formulas [18]:

$$N = -\frac{\sigma_\theta^{(in)} + \sigma_\theta^{(ex)}}{2} b\Delta; \qquad M = -\frac{\sigma_\theta^{(in)} - \sigma_\theta^{(ex)}}{12} b\Delta^2, \quad (16)$$

where $\Delta = R_0 - R_1 = R_0\left(1 - \frac{R_1}{R_0}\right)$ – thickness of the support, $b = 1$ m.

Taking into account expressions (14)–(15), expression (16) can be shaped into:

$$N = -\frac{q_0 R_0}{2} b\left(\frac{3 + \frac{R_1^2}{R_0^2}}{1 + \frac{R_1}{R_0}}\right); \qquad M = -\frac{q_0 R_0^2}{12} b\left(1 - \frac{R_1}{R_0}\right)^2. \quad (17)$$

Expression (17) allows to test the support strength.

Suppose, for example, the support is made of concrete, whose compressive strength is characterized by calculated resistance R_b. The condition of enough bolting strength can be written in the form [18]:

$$|N| \le NS, \qquad (18)$$

where N – calculated normal force, which is determined from the first equation (17), NS – limiting bearing capacity of the support radial section, which is determined by equation $NS = kR_b \Delta b\left(1 - \frac{2e_0}{\Delta}\right)$, in which $k = 1$, $e_0 = \left|\frac{M}{N}\right|$ – eccentricity of the longitudinal force application.

Using expressions (17), we obtain

$$NS = kR_b \Delta b\left(1 - \frac{2e_0}{\Delta}\right) =$$

$$= kR_b R_0\left(1 - \frac{R_1}{R_0}\right)\frac{4\left(2 + \frac{R_1^2}{R_0^2}\right)}{3\left(3 + \frac{R_1^2}{R_0^2}\right)}. \qquad (19)$$

Then the condition of the support strength (18) is shaped into:

$$P_u = q_0 \le = \frac{8kR_b}{3b}\frac{\left(1 - \frac{R_1^2}{R_0^2}\right)\left(2 + \frac{R_1^2}{R_0^2}\right)}{\left(3 + \frac{R_1^2}{R_0^2}\right)^2} \qquad (20)$$

where: P_u – limiting pressure that can be withstood by the considered bolting without loss of bearing capacity.

Thus, line *2* in Figure 3 characterizes the equilibrium state of the concrete support; it should be limited to the point having ordinate $q_0 = P_u$. This means that the support strength will be ensured by the condition:

$$P \le P_u \qquad (21)$$

Significant improving performance of the static support can be achieved by installing it with some shift l_0 from the tunnel bottom. In this case, diagram for determining the equilibrium state of "support-rock mass" deformable system shall be as shown in Figure 4.

Here, as before, line *1* – the rock mass equilibrium state, line *2* – support equilibrium state, for the support erected at a distance of l_0 from the tunnel bottom. *A* point at the intersection of these two lines will correspond to the equilibrium state of the single deformable "support-rock mass" system. For comparison, line *3* (dotted line) presents equilibrium state of support being built directly at the bottom and point *B* – the corresponding equilibrium point. Shift of support $u(l_0)$, implemented in the support construction to support face with l_0 distance (from bottom), is characterized by the segment 00^*.

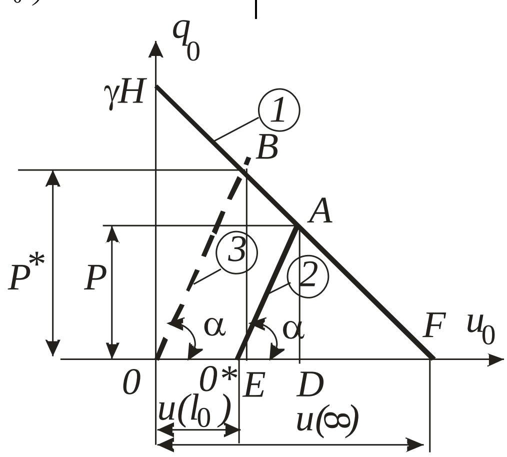

Fig. 4. Determination of equilibrium state of geotechnical system "support-rock mass" in case of support positioning at distance l_0 from tunnel bottom

If the tunnel is unsupported for a long time (the support is installed at considerable distance from the bottom when $l_0 \to \infty$), all possible displacement of the rock mass at this time and actual *0F* interval corresponds to shift of unsupported tunnel, that is, $u(\infty) = \gamma H R_0 / 2G_0$.

As seen from Figure 4 that the support positioning at distance l_0 from tunnel bottom leads to decreasing pressure on the support, i.e. the condition $P \le P^*$ is in force. Mmechanics of underground structures [3] allows to take into account this effect using the equation:

$$P = P^* \alpha^* \text{ or } \alpha^* = P/P^* \qquad (22)$$

Considering the triangles AEF and 0BF, 0^*AD and *0BE* in Figure 4, one can obtain:

$$\alpha^* = \frac{P}{P^*} = \frac{AD}{BE} = \frac{O^*A}{OB} = \frac{OF - OO^*}{OF} = \frac{u(\infty) - u(l_0)}{u(\infty)} = 1 - \frac{u(l_0)}{u(\infty)} = 1 - f(l_0), \qquad (23)$$

where

$$f(l_0) = \frac{u(l_0)}{u(\infty)} = \frac{u(l_0)}{\left(\frac{R_0 \gamma H}{2G_0}\right)}, \quad u(\infty) = \frac{R_0 \gamma H}{2G_0}.$$

This can be obtained from decision of the respective plane problem for unsupported tunnel (microtunnel).

To determine the value of $u(l_0)$, three-dimensional picture of surface deformations arose near the microtunnel should be considered.

Studies related to determining the displacement contour, depending on the distance to the tunnel bottom, are described in large number of papers, among which are the works of N.A. Davydov [19] and M. Baudendistel [4]. When solving the proper task of the theory of elasticity in the preset volume using numerical methods such as finite element method, each of the authors offered their own formula for determining value of $f(l_0)$.

Using specific representation for $f(l_0)$ and applying it to equation (23) produces corresponding formulas for the calculation of correction factor α^*. For instance, based on data of Baudendistel M., Prof. Dr. N.S. Bulychev performed the correlation analysis between the values of ratio α^* and the relative distance l_0/R_0 and suggested applying the exponential dependence [17]

$$\alpha^* = 0{,}64\left(e^{-1{,}75 l_0 / R_0}\right). \qquad (24)$$

Then the corresponding components of the initial stress field in intact rock mass are multiplied by the value of α^*, hence the initial stresses are determined by formulas:

$$\sigma_x^{(0)(0)} = \sigma_y^{(0)(0)} = -\gamma H \alpha^*, \ \tau_{xy}^{(0)(0)} = 0. \qquad (25)$$

3. Conclusion

Some conclusions may be important from practical standpoint:

For example, formula (16) shows that decreasing thickness of the support $\Delta = R_0 - R_1 = R_0\left(1 - \frac{R_1}{R_0}\right)$ results in decreasing pressure on it, and at $R_1 \to R_0$ the pressure is absent at all, that is, $q_0 \to 0$. In very soft soil (loose rock), when the condition $G_1 >> G_0$ (G_1 – shear modulus of the material supported) is in force and $G_0/G_1 \to 0$, pressure q_0 on the support tends to achieve the value of the initial stress in the intact rock mass (in its natural state), e.g. $q_0 \to \gamma H$.

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Название:	**Метод расчета коммуникационных тоннелей в черте города на основе взаимодействия «футеровка-массив грунта»**
Аннотация:	В настоящее время перед мегаполисами остро стоит проблема строительства тоннелей для прокладки коммуникаций. В таких больших городах, как Ханой, Хошимин, до сих пор остаются без решения проблемы прокладки технических трубопроводов, силовых кабелей, водопровода, канализации. Статья описывает метод расчета прокладки коммуникационных тоннелей, учитывающий состав грунта и ограниченные условия строительства для каждой конкретной области, с помощью небольшой туннельной бурильной машины. Использование данного метода обеспечивает надежность и экономическую целесообразность проекта строительства.
Ключевые слова:	масса грунта, прокладка тоннеля, напряжение, расчет, теория упругости, микротоннелирование