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NUMERICAL ANALYSIS OF THE STRESS-STRAIN STATE OF METRO TUNNELS UNDER DIFFICULT CONDITIONS USING MIDAS GTS NX

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ЧИСЕЛЬНИЙ АНАЛІЗ НАПРУЖЕНО-ДЕФОРМОВАНОГО СТАНУ ТУНЕЛІВ МЕТРОПОЛІТЕНУ В СКЛАДНИХ УМОВАХ З ВИКОРИСТАННЯМ MIDAS GTS NX

Purpose. Assessment and predictive analysis of the stress-strain state (SSS) of the soil mass and structural elements of the subway tunnel lining, constructed in difficult hydrogeological conditions of Kyiv (in particular, in water-saturated clay soils), to minimize operational risks and ensure the stability of underground structures.

Methodology. The calculation is carried out using numerical modeling based on the finite element method (FEM) in the Midas GTS NX software package. The study used a nonlinear elastic-plastic Mohr-Coulomb model to describe the soil behavior, taking into account the physical and mechanical characteristics of the mass. The modeling involves taking into account the staged nature of the shield tunneling, the hydrostatic pressure of pore water, and the interaction of the soil-lining system through contact elements. To eliminate edge effects, a calculation area significantly exceeding the tunnel diameter was set.

Results. The nature of the redistribution of stresses and deformations around the tunnel with a diameter of 6 m at a depth of 20 m has been established. The values of the maximum vertical displacements in the tunnel invert (heave) are +10.9 cm, while the settlement of the ground surface above the vault is recorded at 4.8 mm. The maximum total vertical stresses in the lower part of the model reach 1762 kPa. The analysis of the pore pressure showed its linear increase to 353 kN/m², while the tunnel lining acts as a waterproof barrier. Plastic zones have been determined, concentrated in the tunnel haunches and the invert, which indicates the transition of the soil to a failure state in these zones.

Scientific novelty. The dependencies between hydrogeological conditions and the nature of the formation of plastic deformation zones for shallow tunnels under the conditions of Kyiv have been established. It has been proven that in water-saturated clay masses the main deformation factor is the intensive heaving of the invert due to unloading, which prevails over surface settlement. New data have been obtained on the configuration of the plastic zones of the mass under the conditions of the simultaneous influence of the soil self-weight and hydrostatic pressure.

Practical significance. The obtained calculation data allow at the design stage to reasonably choose the reinforcement parameters of the reinforced concrete lining and make decisions on the need for additional strengthening of the surrounding soil mass. This allows to significantly reduce the risks of flooding and uneven deformations of tunnels, ensuring reliable functioning of the city's transport infrastructure under conditions of increasing loads.

Keywords: *subway tunnel, stress-strain state, numerical modeling, finite element method, Midas GTS NX, Mohr-Coulomb model, settlement trough, geomechanical safety.*

Introduction. In many countries, economic policy is paying increasing attention to the modernization and development of transport infrastructure as a measure of macroeconomic stimulation.

However, in addition to macroeconomic and technological problems, for the successful and long-term functioning of transport infrastructure with increasing loads, it is necessary to address the issue of reducing operational risks, the occurrence of which can neutralize the macroeconomic and logistical effect. This issue should be resolved at the design stage.

For example, the construction of the metro in Kyiv is based on the city's needs for the development of transport infrastructure, reducing congestion and ensuring the accessibility of residential areas, this involves improving the quality of life of residents, creating new jobs and reducing the load on ground transport.

The Kyiv metro has over 70 km of tunnels, which form the basis of a network of three lines. The depth of the tunnels of the Kyiv Metro varies considerably: Arsenalna station is the deepest, located at 105.5 meters, while Zoloti Vorota station is at a depth of 96.5 meters. Other stations, such as Dnipro, are located near the surface, and the tunnels under the Dnipro were built at a depth of 28-32 meters. Accordingly, the construction of tunnels in difficult geological conditions, such as the Lybid River area, leads to lower depths (8-10 meters), which, in turn, can cause problems with flooding.

Subway tunnels of various depths are an important component of the transport infrastructure. Their use allows for a significant reduction in travel distances, which seriously increases passenger transportation, and therefore the risks of the consequences of unforeseen loads.

Thus, despite the high cost, complexity and laboriousness of the work, the tendency to increase the volume of tunnels and underground structures is common to all developed countries of the world.

Tunnels are one of the main structures that make up transport systems. When designing such facilities, it is necessary to take into account the deformation characteristics of the soil mass, the depth of its occurrence and the transverse dimensions of the tunnels, as well as various types of loads and influences that affect the stress state of underground structures [1]. Disturbance of the soil mass integrity changes the stress-strain state of the mass, including along the tunnel contour. This leads to the appearance of tensile stresses in some places and compressive stresses in others [2]. Loads and influences on tunnels are classified as permanent and temporary. Rock pressure refers to permanent loads and effects on tunnels. Temporary loads include moving loads from vehicles moving both inside and outside the tunnel. For shallow tunnels, such loads are of particular importance.

Materials and Methods. To calculate the stress-strain state (SS) of soil masses around subway tunnels, numerical models based on continuum mechanics are used, taking into account the elastic, plastic and rheological properties of soils. The most common are the finite element method (FEM), the boundary element method (BEM) and hybrid approaches.

The purpose of phenomenological models in continuum mechanics (CMM) is to describe the behavior of soil as a continuum under the action of loads. These include the theory of elasticity for small, reversible deformations; the theory of plasticity for irreversible deformations (e.g., during failure or shear); and the theory of creep for long-term deformation processes typical of clayey rocks.

The boundary element method (BEM) is a numerical approach to solving problems of continuum mechanics, which is based on discretization of only the boundaries (boundaries) of the computational domain, and not the entire volume, as in the finite element method (FEM). When using the BEM, the main disadvantage is the complexity of modeling inhomogeneous or multilayer soils. In works [3–7] it is shown that the combination of both methods allows achieving an optimal balance between accuracy and computational efficiency.

The boundary element method (BEM) is less common in commercial software than the finite element method (FEM), but it is indispensable in problems where the medium is infinite (geomechanics, acoustics) or where ultra-high accuracy on the surface is important (cracks, stress concentration).

For the practical implementation of numerical analysis of the NDS around subway tunnels, the Midas software package, which is based on the finite element method (FEM), is often used. The program allows taking into account the complex geometry of the lining and the staged construction of the tunnel. To minimize the influence of boundary conditions and simulate a semi-infinite mass of soil, special infinite elements (Infinite Elements) are used in Midas, which allows combining the advantages of volumetric modeling of MSE with the accuracy of boundary solutions [3–5].

In the Midas software package, determining the SST around a tunnel requires the correct specification of three groups of parameters. The initial data include the geometric dimensions of the computational domain (the width and depth of which must exceed 3–5 tunnel diameters to eliminate edge effects), physical and mechanical characteristics of the soil (Mohr-Coulomb model with parameters E, v, γ, c, φ) and stiffness parameters of reinforced concrete lining with mandatory introduction of contact interface elements.

The mathematical formulation of the dimensions of the calculation area to exclude the influence of edge effects is based on the outer diameter of the tunnel D:

$$L \geq (3\ldots5) \cdot D; \quad \underline{H_b} \geq (3\ldots5) \cdot D \tag{1}$$

where L is the distance from the tunnel axis to the side faces of the model, m; H_b – distance from the tunnel invert to the lower boundary of the soil mass, m.

The soil mass behavior is described through elastic and plastic components. The elastic component (Hooke's Law) describes the relationship between stresses and strains, and the plastic component (Mohr-Coulomb criterion) is determined by the cohesion parameter c and the angle of internal friction φ.

Limit state condition:

$$f = \sigma_m \sin\varphi + \sigma^{-}\left(\cos\theta - \frac{1}{\sqrt{3}}\sin\theta\sin\varphi\right) - c\cos\varphi = 0 \tag{2}$$

The reinforced concrete lining is modeled as an elastic element with axial stiffness and bending stiffness characteristics. To account for the slippage of the lining relative to the soil, interface elements are introduced, the stiffness of which is (k_n – normal, k_t – tangential) determined by the characteristics of the surrounding soil:

$$k_n = \frac{E_{soil}(1 - v_{soil})}{\mathrm{d_v}(1 + v_{soil})(1 - 2v_{soil})}, \; k_t = \frac{G_{soil}}{d_v}, \tag{3}$$

where d_v – virtual interface thickness.

This mathematical formulation allows us to implement in Midas GTS NX a finite element scheme for solving a system of linear algebraic equations of the form:

$$f[K]\{U\} = \{F\} \tag{4}$$

where $[K]$ – global stiffness matrix, taking into account the nonlinearity of the soil, U – nodal displacements, F – vector of external loads (self-weight and man-made pressure).

Initial conditions form the natural geostatic state before the start of work: the self-weight of the soil is generated, the lateral expansion stress is generated due to the coefficient of lateral pressure at rest K_0 and the initial hydrostatic pressure of pore water for water-saturated clay.

Boundary conditions are imposed according to the kinematic principle: rigid fixation of the lower boundary along all axes $Tx = Ty = Tz = 0$, roller supports on the side boundaries (prohibition of horizontal movements) $Tx = 0$, $Ty/Tz \neq 0$ and stress-free ground surface.

Engineering and geological conditions typical for the territory of Kyiv are represented by modern (Holocene) alluvial sandy loams and sands, covered with a fill layer up to 5.0 m thick and underlain by marly clays of the Eocene age of the Paleogene system (Kyiv horizon), lower in the section - by sands of the Buchach horizon of the Middle Eocene of the Paleogene system.

Geometric characteristics of tunnel lining - the internal diameter of the right running tunnel on the site is 5400 mm, the external 6000 mm. The thickness of the blocks (frame elements) is 300 mm, their width is 1000 mm. The length of the blocks is variable and is 451 mm, 939 mm, 1453 mm, 1855 mm, the size of the block for the bottom is 2724 x 1000 x 595 (mm). Reinforcement of the blocks – A400, A240, concrete B40. A typical cross-section of the tunnel is shown in Fig. 1.

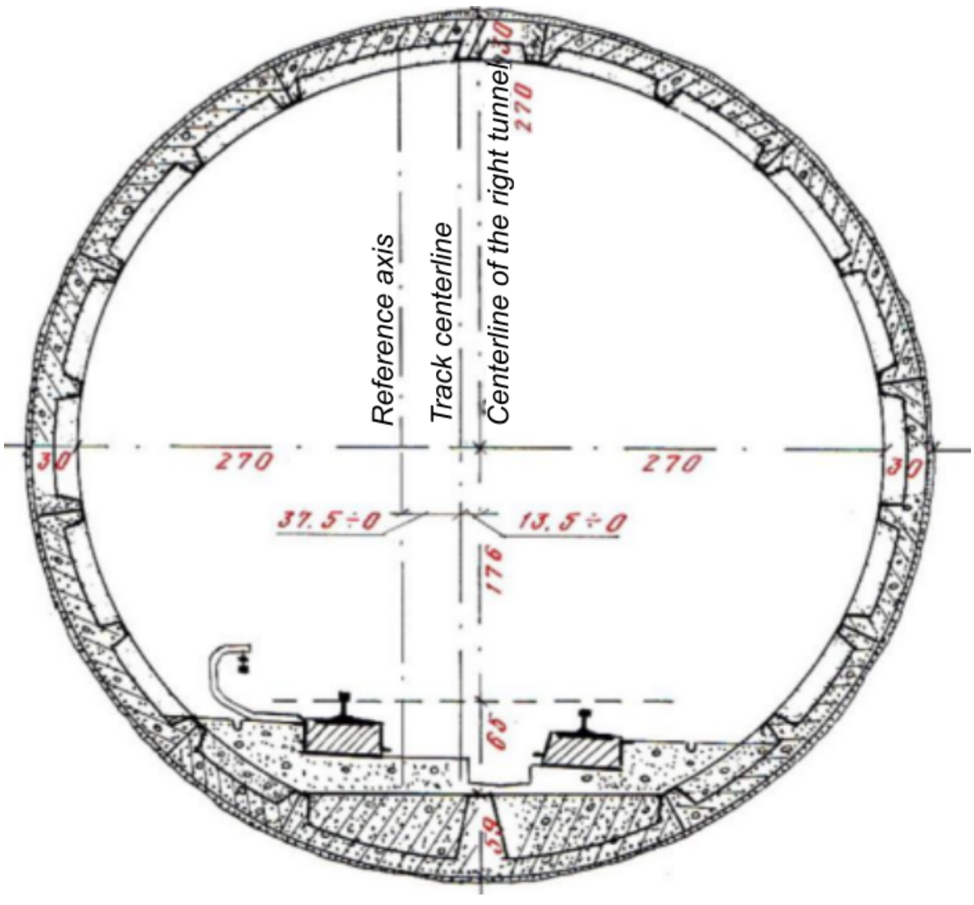

Fig. 1 Typical tunnel cross-section

In order to give an example of the calculation, we will simulate a specific scenario in Midas for a tunnel with a diameter of 6 m at a depth of 20 m in water-saturated clay.

The initial data for modeling the stress-strain state of soils during shield tunneling are given in Tables 1–2.

Table 1

Physical and mechanical characteristics of tunnel construction materials

Material Name	Model Type	Elastic Modulus E, kN/m²	Poisson's Ratio ν	Unit Weight γ, N/m³	Damping Coefficients
Reinforced concrete tunnel lining	Elastic	$3.6 \cdot 10^7$	0.2	25	0.05
Steel structure of the tunnel shield	Elastic	$2.1 \cdot 10^8$	0.25	77	0.05
Cement-sand grout injected behind the lining	Elastic	$1.5 \cdot 10^6$	0.3	22	0.05

Table 2

Design parameters of the soil mass

Material Name	Model Type	Elastic Modulus E, kN/m²	Cohesion C, kN/m²	Angle of internal friction φ, deg.	Unit Weight γ, kN/m³	Coefficient of lateral earth pressure K_0
Soft rock (Weak rock)	Mohr-Coulomb	30 000	30	36	20	0.691

Hydrogeological and filtration parameters are given by the permeability coefficient (x, y, z) $k = 1 \cdot 10^{-5}$, Poisson's ratio $\nu = 0.495$

Below is a description of the results produced by the program, with numerical values and interpretation of physical processes (Fig. 2–6).

Fig. 2 shows the contour plots of vertical displacements (T_Z) where a characteristic distribution of displacements around the excavated space of the tunnel is observed. The contours of the blue zones above the tunnel indicate the settlement of the soil mass, the maximum value of which is about 4.8 mm. And the red zone under the tunnel demonstrates the heaving of the invert due to unloading of the soil mass, the maximum value of the invert heave is 10.9 cm ($+ 1.09 \cdot 10^{-1}$ m).

Analysis of the data obtained from the displacement graph shows that the maximum displacement values are recorded in the central part, where the zone of the greatest load influence is formed. Towards the edges of the calculation area, the displacement values gradually decrease, which indicates the symmetrical nature of the deformation process.

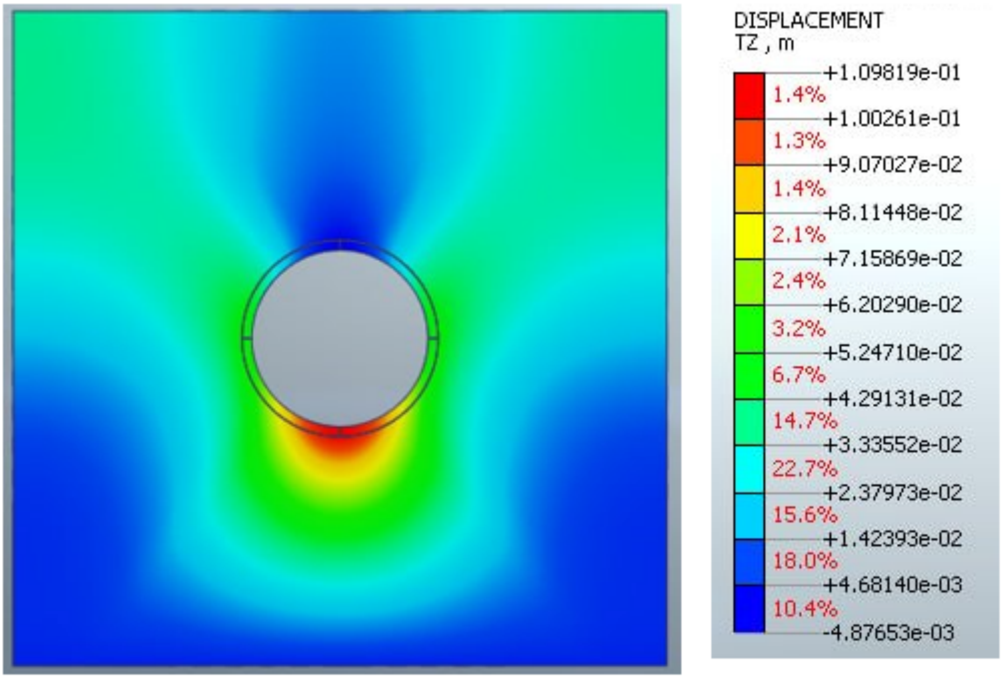

Fig. 2. Vertical movements at the final stage of tunnel construction

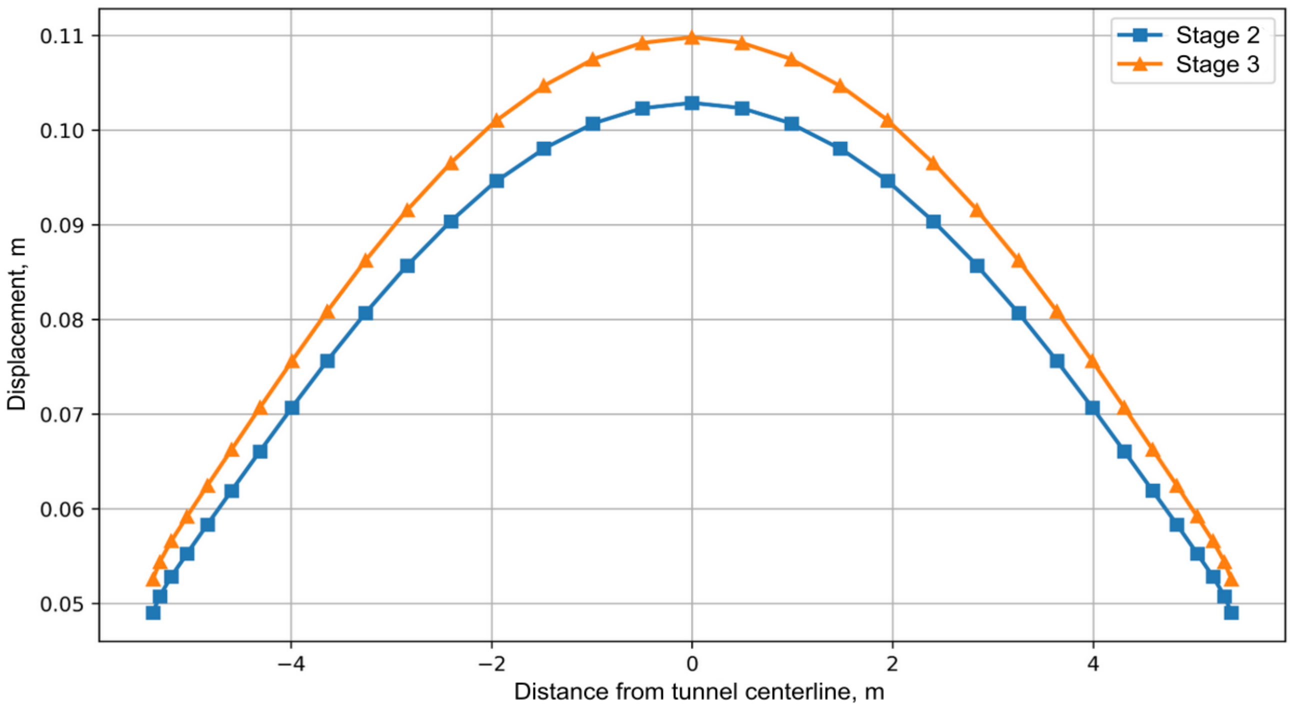

Fig. 3. Displacement graph (TZ) in the tunnel invert (stage 2 – the excavation stage (unloading the soil mass), stage 3 – reinforced concrete lining)

Fig. 4 shows the characteristic total vertical stresses (σ_{zz}) kN/m².

A classical geostatic distribution is observed in which stresses increase with depth (from red at the top to blue at the bottom), stress concentrations are noticeable on the sides, and above and below the tunnel the stresses are somewhat lower due to the unloading of the soil mass. The maximum compressive stresses at the bottom are about 1762 kPa (1.76 MPa).

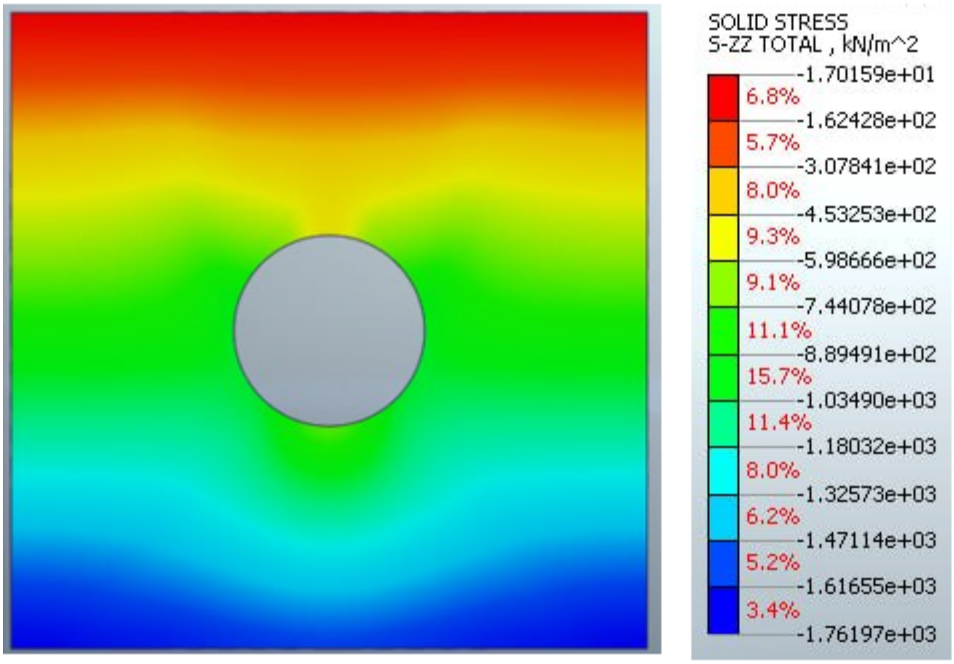

Fig. 4. Distribution of total vertical stresses

Figure 5 shows the hydrostatic pressure, which increases linearly with depth, the pressure varies from 0 at the surface to approximately 353 kN/m² at the lower boundary of the calculation area. The contours of the isofields are slightly curved around the tunnel lining, which indicates that the lining acts as a waterproof barrier, changing the natural filtration flow.

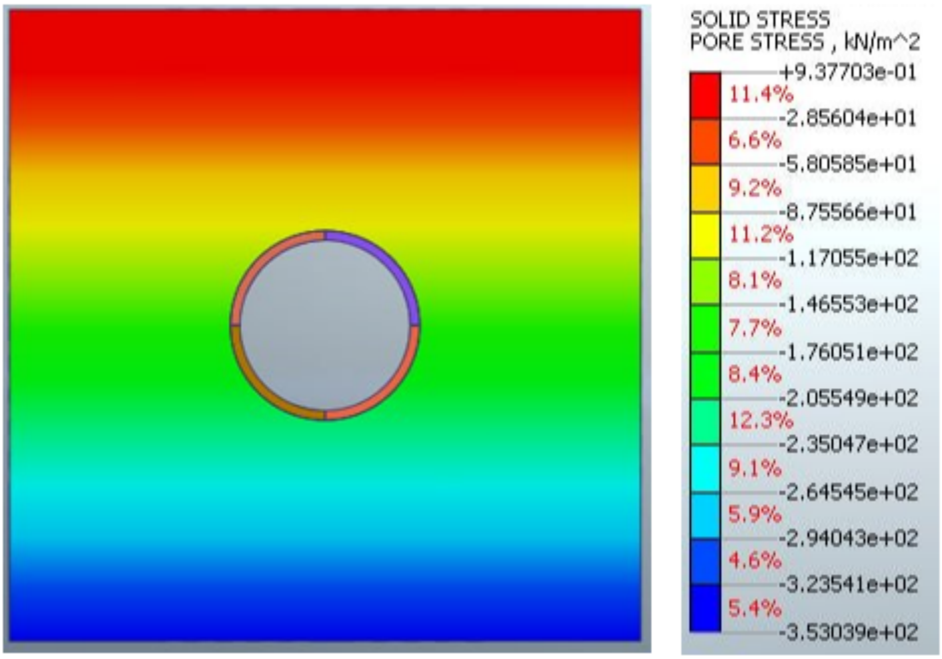

Fig.5 Hydrostatic pressure

Critically important for assessing stability is the determination of plasticity zones (Fig. 6). Red color shows the zones where the strength limit according to the Mohr-

Coulomb criterion is reasonably reached and passes into a plastic state (destruction). They are concentrated in the "shoulders" of the tunnel and in the tunnel invert. This explains the large value of the heaving in Fig. 2. Blue color indicates the unloading zones of the soil mass.

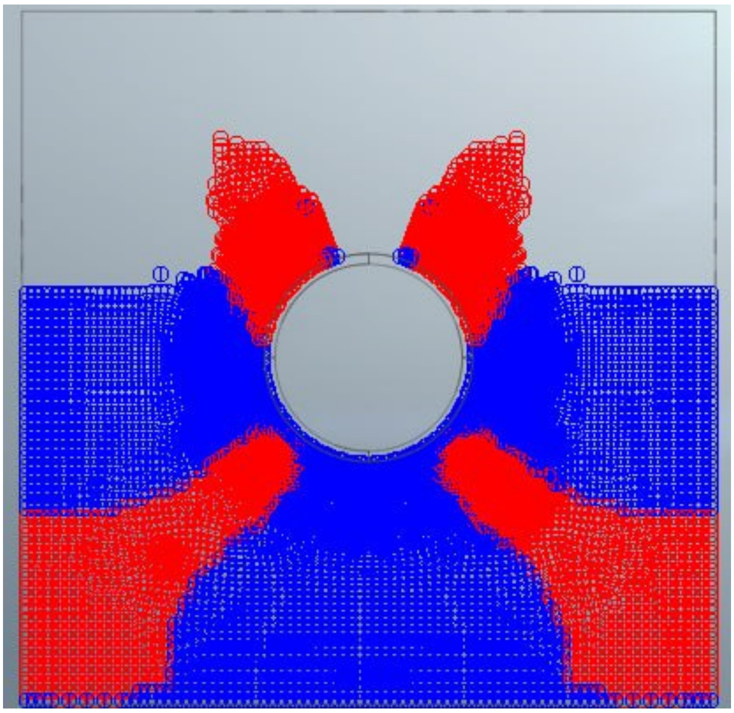

Fig. 6. Plastic zones around the tunnel

The calculation shows that the main problem for this type of lining and given conditions is the instability of the invert (bottom) and the formation of significant zones of plastic deformation in the lateral parts. The selected reinforced concrete lining takes up a significant portion of the load, but the surrounding soil mass requires additional strengthening measures to minimize the zones of base failure.

Conclusions. The use of the Midas GTS NX software package allows for high-accuracy prediction of changes in the stress-strain state of the soil mass, taking into account complex conditions (water saturation, nonlinear soil properties and staged work).

Modeling showed that when building tunnels in soft water-saturated soils of Kyiv, the main risk is not only surface settlement, but also intense heave of the tunnel invert (up to 10.9 cm), which is caused by unloading the soil mass and the transition of the soil under the tunnel to a plastic state.

It was established that plastic zones are concentrated in the tunnel haunches and the invert. This indicates that the reinforced concrete lining operates under conditions of significant uneven pressure, which requires careful calculation of its reinforcement.

Hydrostatic pressure significantly affects the distribution of stresses, and the tunnel lining acts as a barrier that changes natural filtration. This confirms the need to design reliable waterproofing and take into account additional water pressure on the lining.

To ensure the stability of tunnels in such conditions, it is recommended to provide for measures for proactive strengthening of the face or grouting of the surrounding soil mass, especially in the tunnel invert, to minimize plasticity zones and prevent excessive deformations.

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АНОТАЦІЯ

Мета. Оцінка та прогнозний аналіз напружено-деформованого стану (НДС) ґрунтового масиву і конструктивних елементів оправлення тунелю метрополітену, що споруджується у складних гідрогеологічних умовах Києва (зокрема, у водонасичених глинистих ґрунтах), для мінімізації експлуатаційних ризиків та забезпечення стійкості підземних споруд.

Методика. Розрахунок виконано методом чисельного моделювання на основі методу скінченних елементів (МСЕ) у програмному комплексі Midas GTS NX. У дослідженні використано нелінійну пружно-пластичну модель Мора–Кулона для опису поведінки ґрунту з урахуванням фізико-механічних характеристик масиву. Моделювання передбачає врахування поетапності щитової проходки, гідростатичного тиску порової води та взаємодії системи «ґрунт–оправлення» через контактні елементи. Для виключення крайових ефектів задано розрахункову область, що значно перевищує діаметр тунелю.

Результати. Встановлено характер перерозподілу напружень і деформацій навколо тунелю діаметром 6 м на глибині 20 м. Значення максимальних вертикальних переміщень у лотку тунелю (здимання) становлять +10,9 см, тоді як осідання земної поверхні над склепінням зафіксовано на рівні 4,8 мм. Максимальні сумарні вертикальні напруження в нижній частині моделі досягають 1762 кПа. Аналіз порового тиску показав його лінійне зростання до 353 кН/м², при цьому оправлення тунелю виконує функцію водонепроникного бар'єра. Визначено пластичні зони, сконцентровані в боках склепіння та лотку тунелю, що свідчить про перехід ґрунту в граничний стан у цих зонах.

Наукова новизна. Встановлено залежності між гідрогеологічними умовами та характером формування зон пластичних деформацій для тунелів мілкого закладення в умовах Києва. Доведено, що у водонасичених глинистих масивах основним деформаційним чинником є інтенсивне здимання лотка внаслідок розвантаження, яке переважає над осіданням поверхні. Отримано нові дані щодо конфігурації пластичних зон масиву за умов одночасного впливу власної ваги ґрунту та гідростатичного тиску.

Практична значимість. Отримані розрахункові дані дозволяють на стадії проєктування обґрунтовано обирати параметри армування залізобетонного оправлення та приймати рішення щодо необхідності додаткового закріплення навколишнього ґрунтового масиву. Це дає змогу суттєво знизити ризики підтоплення та нерівномірних деформацій тунелів, забезпечуючи надійне функціонування транспортної інфраструктури міста в умовах зростаючих навантажень.

Ключові слова: *тунель метрополітену, напружено-деформований стан, чисельне моделювання, метод скінченних елементів, Midas GTS NX, модель Мора–Кулона, мульда осідання, геомеханічна безпека.*

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