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PREDICTION OF GROUND SUBSIDENCE BY MODELING TUNNEL BORING MACHINE TECHNOLOGY

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Abstract: Tunneling using tunnel boring machines (TBMs) induces ground settlement, leading to deformations in buildings and structures located within the zone of influence of the planned construction. Settlement prediction based on volume loss is one of the established practices in Russia; however, this method relies on construction experience and does not directly account for the primary technological parameters affecting soil settlement. In some cases, to obtain accurate deformation predictions using this method, it is necessary to perform a back-calculation of volume loss, considering the tunnel section already excavated. This study examines the modeling of all technological factors involved in TBM tunneling for forecasting settlement resulting from metro tunnel construction. A division of the TBM excavation process into sequential phases is introduced for finite element analysis. Two calculation schemes were formulated for different sections of the Rublyovo–Arkhangelskaya line of the Moscow Metro, in two variants: (1) using volume loss, and (2) accounting for all technological factors. In the latter variant, parameters such as shield face pressure, grout injection pressure, TBM weight, structural stiffness of the TBM, shield tail gap, and others were incorporated. Displacement plots for adjacent buildings were generated from the resulting calculations and compared with measured displacements. A comparative analysis between the volume loss method and the alternative comprehensive technological-factor method was conducted. The comprehensive method demonstrated a high degree of quantitative and qualitative agreement with measured values and allows assessment of the soil's mechanical behavior over time during TBM excavation.

Keywords: TBM, volume loss, face pressure, grouting pressure, subsidence, FEA, tunneling, Plaxis 3D

ПРОГНОЗ ОСАДКИ ЗЕМНОЙ ПОВЕРХНОСТИ ПРИ МОДЕЛИРОВАНИИ ТЕХНОЛОГИИ ПРОХОДКИ ТОННЕЛЯ МЕХАНИЗИРОВАННЫМ КОМПЛЕКСОМ

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Аннотация: Проходка тоннелей при помощи тоннелепроходческих механизированных комплексов (ТПМК) вызывает осадку поверхности, что приводит к деформациям зданий и сооружений, находящихся в зоне влияния проектируемого строительства. Прогноз осадок с использованием коэффициента перебора является одним из устоявшихся в российской практике, однако этот метод основан на опыте строительства и не учитывает напрямую основные технологические параметры, влияющие на осадку грунта. В ряде случаев для получения корректных деформаций этим методом необходимо делать обратный пересчет коэффициента перебора с учетом уже пройденного участка трассы. В представленном исследовании рассматривается моделирование всех технологических факторов проходки ТПМК для прогноза осадки от строительства тоннелей метрополитена. Предложено разбиение на этапы проходки ТПМК для моделирования МКЭ. Разработаны 2 расчетные схемы разных участков Рублево-Архангельской линии Московского метрополитена в двух вариантах: с использованием коэффициента перебора, с учетом всех технологических факторов. При моделировании альтернативным методом учитывались такие технологические параметры как: давление пригруза, давление нагнетания, вес ТПМК, жесткость конструкций ТПМК, кольцевой зазор и др. В результате расчета построены графики перемещений зданий окружающей застройки. Произведен анализ полученных результатов и их сравнение со значениями фактических перемещений. Проведено сравнение расчета с использованием коэффициента перебора и альтернативного метода расчета. Предложенный альтернативный метод расчета показывает высокую количественную и качественную сходимость с фактическими значениями, а также позволяет оценить механику поведения грунта при проходке ТПМК во времени.

Ключевые слова: ТПМК, коэффициент перебора, давление пригруза, давление нагнетания, осадка, МКЭ, тоннелирование, Plaxis 3D

1. INTRODUCTION

Tunnel boring machine (TBM) is a high-technology equipment used for the construction of underground tunnels. It was first employed in 1825 during the excavation of the Thames Tunnel in London, marking a major milestone in the history of tunneling. Today, TBMs are widely used for the construction of metro lines, road tunnels, water supply conduits, and other underground structures, especially in densely populated urban areas where usual excavation methods may be impractical. In international practice, TBMs with active face support have been successfully applied in challenging geological conditions for tunnel construction [1–2].

Despite their advantages, tunnel excavation using TBMs involves certain risks and adverse consequences. One of these consequences is the movement of the ground mass [1], which depends not only on the geotechnical conditions of the excavation site but also on the operational parameters of the shield-driven excavation [3]. These settlements can lead to deformations and damage of buildings, structures, and infrastructure located above or adjacent to the tunnel [4–6]. This factor necessitates rigorous monitoring to minimize environmental impact and ensure safety.

Surface settlements induced by tunnel excavation has been investigated by numerous foreign researchers, including R. B. Peck [7], N. Loganathan [8], M. O'Reilly and B. New [9], P. B. Attewell [10], and many others. Among domestic scholars, notable contributions have been made by Protosenya A.G. [11], Tupikov M.M. [12], Mazein S.V. [13], and others.

To minimize these risks and ensure construction safety, monitoring of ground surface settlement is employed. This method involves observing soil movements and deformations of buildings within the TBM tunneling influence zone. Geotechnical monitoring encompasses visual and instrumental inspections as well as automated systems that collect real-time data. Monitoring enables timely detection of critical changes such as cracks, tilts, or excess settlement and the implementation of corrective measures to prevent emergency situations [14–16].

To assess potential displacements of the soil mass and surface structures within the projected construction influence zone geotechnical calculations are used.

In practice, ground surface settlement due to TBM excavation is calculated using the volume loss, defined as the ratio of the excavation cross-section to the designed lining cross-section according to SP 249.1325800.2016 "Underground utilities. Design and construction by closed and cut-and-cover methods". It should be noted that this standard applies to trenchless installation of utilities by microtunnel boring machine (MTBM) and horizontal directional drilling (HDD) machine and does not directly cover metro tunneling with TBMs, it is used in the absence of more specific regulations. This approach represents an established practice in Russia and has been described by domestic authors such as Ter-Martirosyan A.Z., Isaev I.O., et al. [17–20], Isaev O.N., Sharafutdinov R.F. [21], and others. Despite its widespread use, the volume loss is grounded in construction experience rather than the physical processes driving soil settlement [1]. It permits determination of settlement on the moment of tunnel completion, and finite-element modeling based on this ratio is further complicated by the lack of a regulated methodology for defining the boundaries of the calculation model – parameter that can significantly affect the accuracy of results [22].

Ground settlement induced by TBM-driven excavation is caused by factors such as shield tail gap, taper of shield, and backfill shrinkage [23]. In addition, parameters influencing settlement include excavation gradient, tunnel overburden depth, face pressure and grouting pressure [24]. In the study by H. Liu, J. Shi, J. Li, and C. Liu [25], a method was presented for calculating surface settlement that accounts for various technological parameters: face pressure, grouting pressure, shield dimensions, shield tail gap, changes in shield alignment, and hardening of the cement slurry in the annular space behind lining.

The aim of the present work is to establish an optimal methodology for predicting surface settlement induced by TBM excavation, as well as to evaluate an alternative calculation approach that omits the volume loss while incorporating all technological factors affecting settlement, or to refine the existing limits of the volume loss for given excavation conditions.

2. MATERIALS AND METHODS

The article models several TBM tunneling cases along different sections of the Rublyovo-Arkhangelskaya line of the Moscow Metro: from *Bulvar Karbysheva* station to *Zhivopisnaya* station, and from *Zvenigorodskaya* station to *Karamyshevskaya* station.

For the alignment section considered in the first case, the geological structure is as follows: anthropogenic soils (tQIV); alluvial deposits of the 3rd (Khodynka) terrace above the floodplain of the Moscow River (aQIII); fluvioglacial deposits of the Oka-Dnieper horizon (fQII O-D). Beneath these lie Carboniferous rocks of the upper part of the Kasimovian stage: Upper Carboniferous deposits of the Voskresensk subformation (C3vs), and Upper Carboniferous deposits of the Suvorovo subformation (C3sv), represented by fractured limestones degraded to boulders, gravel, and rubble, as well as stiff clays.

For the alignment section in the second case, the geological profile is as follows: anthropogenic soils (t–QIV), alluvial deposits of the 3rd terrace above the floodplain of the Moscow River (a–QIII), moraine deposits of the Dnieper glaciation (g-QIId), fluvioglacial deposits of the Oka–Dnieper horizon (f–QIIo-d), Oxfordian deposits of the upper part of the Jurassic system (J3ox), Bathonian–Callovian deposits of the upper and middle parts of the Jurassic system (J2+3bt–k), Voskresensk subformation (C3vs), and Suvorovo subformation (C3sv) of the upper Carboniferous system.

Geological profiles along the proposed tunnel alignments are shown in Fig. 1 and Fig. 2.

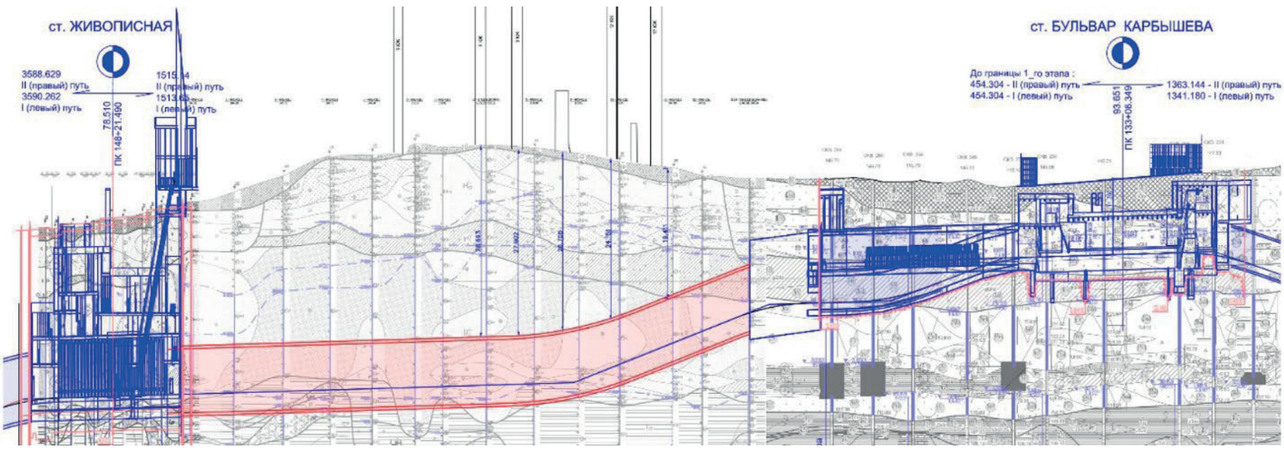


Figure 1. Longitudinal profile of the section from Bulvar Karbysheva station to Zhivopisnaya station

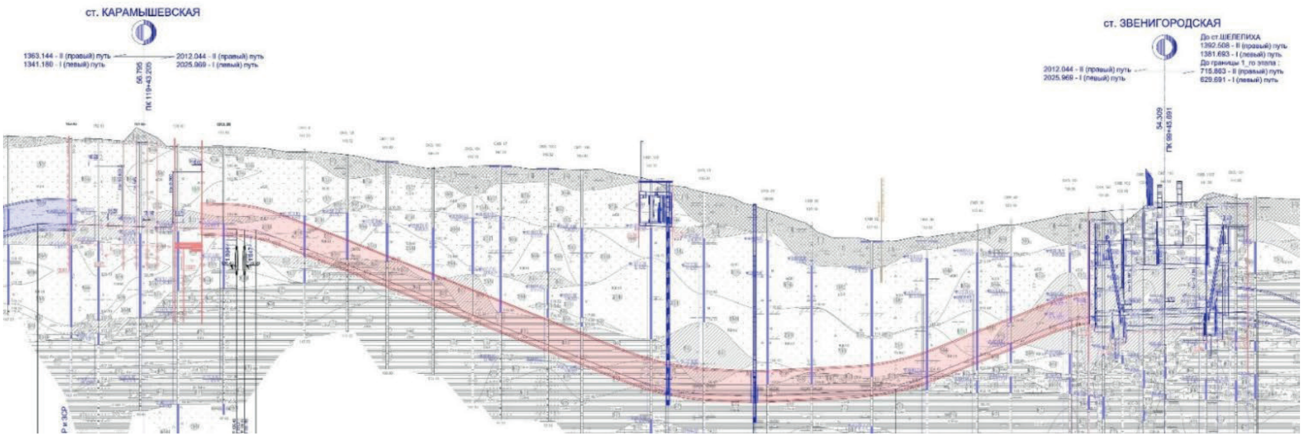


Figure 2. Longitudinal profile of the section from Zvenigorodskaya station to Karamyshevskaya station

Finite Element Model

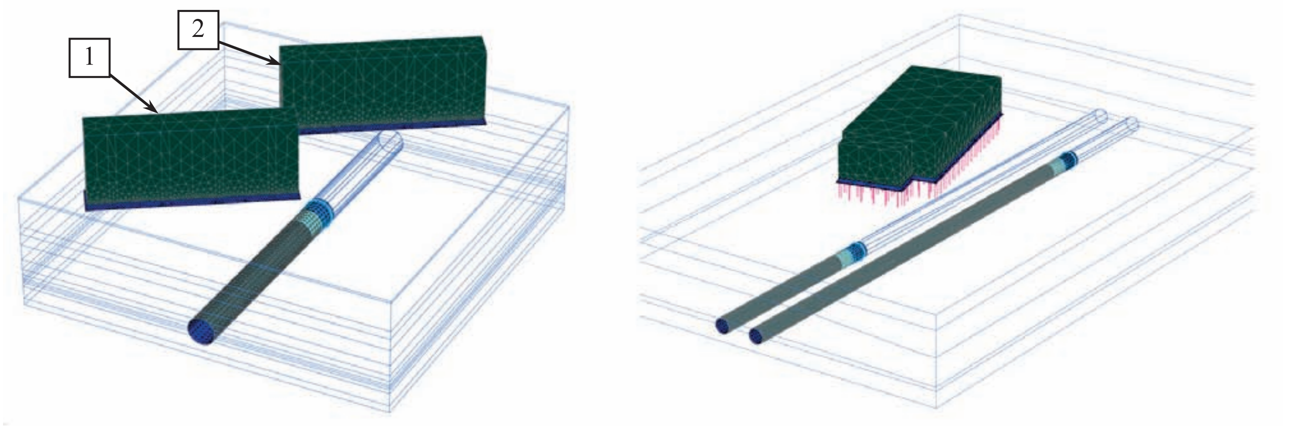
The first model represents a section measuring 145 × 151.2 m with a depth of 45.4 m; tunneling is carried out at a depth of 40 m. Within the calculation scheme, the total tunneling length is 151.2 m, with the excavation length per phase equal to the ring width – 1.8 m.

The calculation scheme for this section is shown in Fig. 3a.

The model includes two buildings (Fig. 3a) located at: Moscow, Marshala Tukhachevskogo Street, 56 bldg. 3 (1) and Moscow, Parshina Street, 21 bldg. 2 (2), both at a distance of 0 m from the tunnel axis.

The second model has dimensions of 163.9 × 248 × 35.4 m. In the model, tunneling is carried out at a depth of 32.4 m, and its length within the calculation scheme is 248 m for both the right and left tunnels. The excavation length per phase is equal to the installed ring width – 1.2 m.

The calculation scheme for this section is shown in Fig. 3b.



a) b)

Figure 3. Calculation schemes:
a) section from Bulvar Karbysheva station to Zhivopisnaya station,
b) section from Zvenigorodskaya station to Karamyshevskaya station

On the ground surface above the model, there is a building located at 2nd Silikatny Passage, 9 bldg. 2, at a distance of 0 m from the tunnel axis.

For settlement calculations using volume loss, a value of **0.78%** was adopted in accordance with [17] for the section from *Bulvar Karbysheva* to *Zhivopisnaya* station, and a value of **1.14%** in accordance with [20] for the section from *Zvenigorodskaya* to *Karamyshevskaya* station.

In developing an alternative calculation method, the approach proposed by H. Liu, J. Shi, J. Li, and C. Liu [25] was used as the basis. In their work, the authors accounted for face support pressure, grouting pressure, TBM shield stiffness, tunnel lining stiffness, the shield tail gap, and grout hardening. In the present study, in addition to these parameters, the TBM shield weight was considered; stiffening ribs were added inside the shield to ensure TBM rigidity; tail grouting pressure in the over-lining space was modeled as hydrostatic pressure instead of a surface-applied load to account for soil pressure and uniform compression of the lining; a sealed TBM brush element was included, enabling simulation of tunnel lining installation under shield protection; and stiffness values for certain materials were corrected.

The tunneling from *Bulvar Karbysheva* to *Zhivopisnaya* station is carried out using an Earth Pressure Balance (EPB) TBM with a diameter D = 10.87 m, length 9.38 m, and weight of about 1,000 t. The external lining diameter is d = 10.5 m, ring width is 1.8 m, and lining thickness is 0.45 m. The face support pressure is set according to the actual pressure during excavation of each ring.

In the area near the buildings, the face support pressure is 2.8 atm, and the tail grouting pressure is 4 atm.

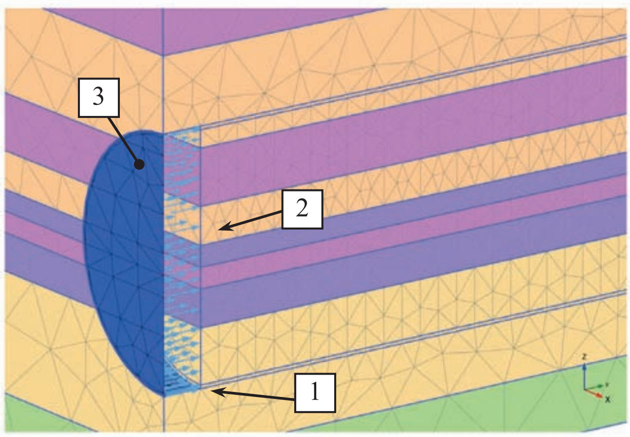
The tunnel section from *Zvenigorodskaya* to *Karamyshevskaya* station of the Rublyovo–Arkhangelskaya line was constructed using two different imported EPB TBMs. The technical characteristics of both shields are similar: diameter – 6.56 m, length – 9.45 m, weight – about 225 t, external tunnel lining diameter – 6.3 m, ring width – 1.2 m, and lining thickness – 0.3 m. The face support pressure was set at 3 atm, and the tail grouting pressure at 4 atm.

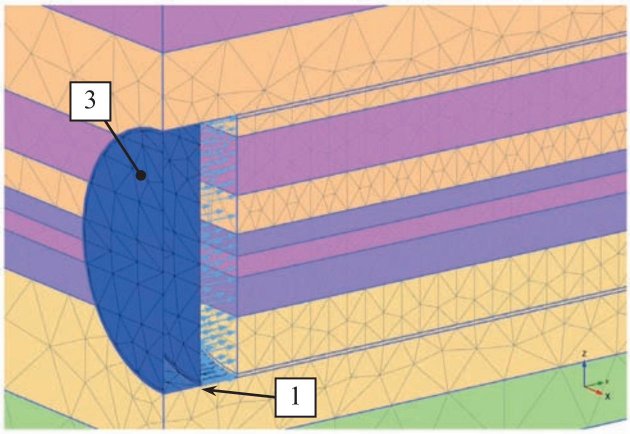
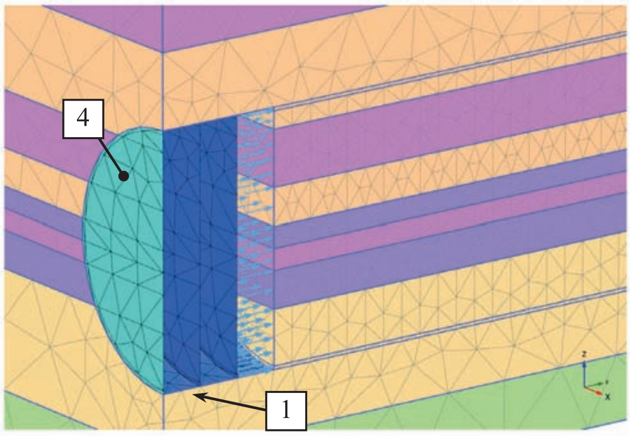
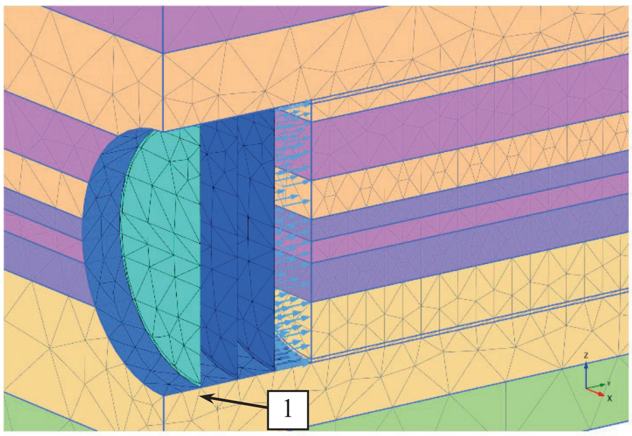
Tunneling sequence

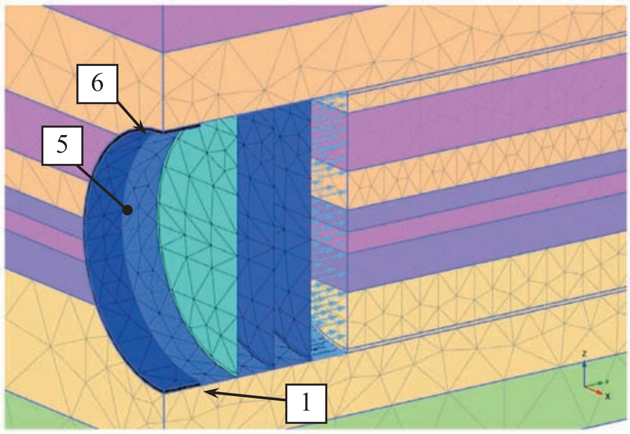
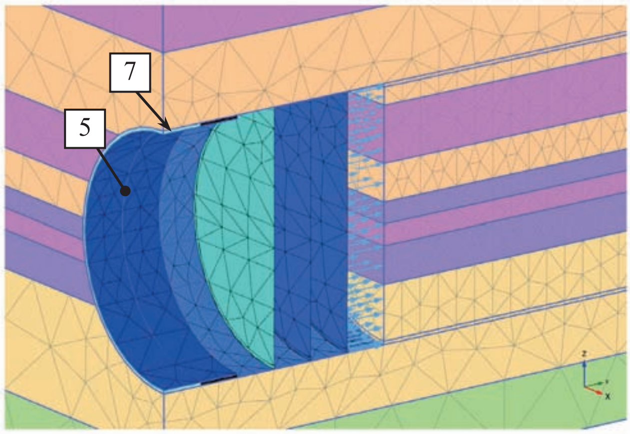
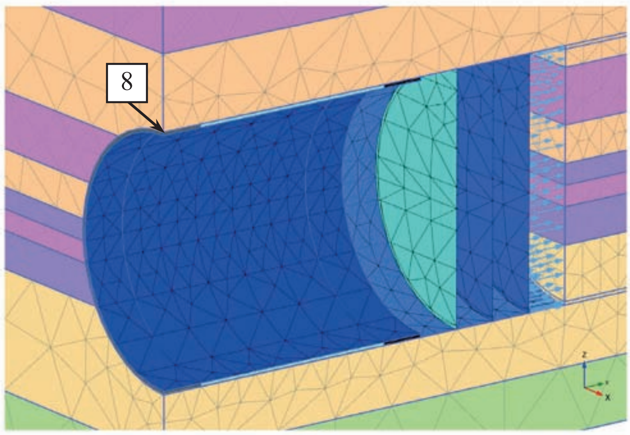
The tunneling on the section from *Bulvar Karbysheva* to *Zhivopisnaya* station is modeled according to the scheme presented in Table 1.

A similar tunneling simulation is performed in the calculation model for the section from *Zvenigorodskaya* to *Karamyshevskaya* station, taking into account the ring width and the shield length used during the construction of that section.

Table 1. Staged modeling of excavation using the alternative method

Tunneling phase No.	Changes at the stage	Model
1	The first shield shell segment (1) is activated; face support pressure (2) is applied at the front plane; at the rear plane of the first section, a bulkhead (3) is activated.	

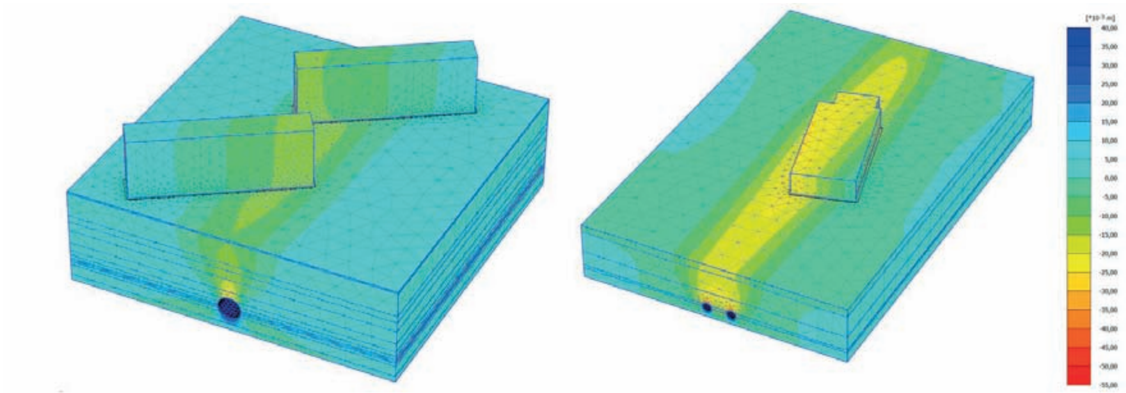
Tunneling phase No.	Changes at the stage	Model
2	The shield shell segment (1) is activated, and a bulkhead element (3) is activated at the rear plane of the section.	
3	The shield shell segment (1) is activated, and a bracing frame (4) is activated at the rear plane of the section.	
4	The shield shell segment (1) is activated.	

Tunneling phase No.	Changes at the stage	Model
5	The shield shell segment (1) and the tunnel lining (5) are activated; within the space between the shield shell (1) and the tunnel lining (5), the TBM tail brush-seal element (6) is activated, and interface elements are assigned at the shell–brush and lining–brush boundaries.	
6-10	The tunnel lining (5) is activated; the void between the cutting diameter and the lining diameter is represented by a grout material (7) with low stiffness, and its hydrostatic pressure is set equal to the grout-injection pressure.	
11	The cement slurry is replaced by hardened backfill grout (8) characterized by increased stiffness parameters.	

Properties of the modeled elements are provided in Tables 2 and 3, and the geological boreholes used in the models are listed in Table 4

3. RESULTS

Contours of vertical displacements for the first and second models are shown in Fig. 4. The results of the numerical models are presented as graphs of vertical deformation in Figs. 5 and 6.



a) b)

Figure 4. Contours of vertical displacements:
a) section from Bulvar Karbysheva station to Zhivopisnaya station,
b) section from Zvenigorodskaya station to Karamyshevskaya station

Table 2. Properties of soils and auxiliary ground elements (1st model)

Soil elements modeled using Linear Elastic и Mohr-Coulomb					
Soil element	γ / kN/m³	E / MPa	ν	c / kPa	φ' (phi)
Anthropogenic layer (1-40)	16	12	0,3	1	29
Medium sands (10-5-3-2)	17	27	0,3	2	34
Loams with grus and crushed stone (24-3-3)	21,4	19	0,35	54	27
Silty sands (25-5-5-1)	18,2	25	0,3	7	33
Fine sands (25-5-4-1)	18,7	40	0,3	5	35
Medium sands (25-5-3-1)	18	44	3	3	37
Clays with sand pockets (41-2-2)	17,8	22	0,35	120	18
Clays with faunal remains (43-2-2)	19,4	29	0,3	85	18
Materials modeled using Linear Elastic и Mohr-Coulomb					
Tail brush-seal	0,1	100	0,3	-	-
Backfill grout material	0,1	1	0,4	-	-
Hardened backfill grout	24,8	700	0,3	43,2	28
Materials modeled using Plate elements					
Material name	γ / kN/m³	E / MPa	ν	G / MPa	d / m
Tunnel lining	15	34000	0,2	14380	0,45
Shield shell	400	360000	0,3	138500	0,075
Bracing frame	0,1	210000	0,3	80770	0,3
Bulkhead	34,8	210000	0,3	80770	0,05

Table 3. Properties of soils and auxiliary ground elements (2nd model)

Soil elements modeled using Linear Elastic и Mohr-Coulomb					
Soil element	γ / kN/m³	E / MPa	ν	c / kPa	φ' (phi)
Anthropogenic layer (1-40)	17,8	14	0,3	2	32
Fine sands (15-5-4-2)	17,9	27	0,3	2	32
Fine sands (25-5-4-1)	20,1	34	0,3	3	36
Limestones (51-16-3)	22,7	50	0,25	-	-
Clays (52-2-2)	19,8	26	0,2	61	21
Materials modeled using Linear Elastic и Mohr-Coulomb					
Tail brush-seal	0,1	100	0,3	-	-
Backfill grout material	0,1	1	0,4	-	-
Tail brush-seal	24,1	700	0,3	43,2	28
Materials modeled using Plate elements					
Material name	γ / kN/m³	E / MPa	ν	G / MPa	d / m
Tunnel lining	25	34000	0,2	14380	0,3
Shield shell	7,35	360000	0,3	138500	0,04
Bracing frame	0,1	210000	0,3	80770	0,3
Bulkhead	355,2	210000	0,3	80770	0,05

Table 4. Geological boreholes used in the models

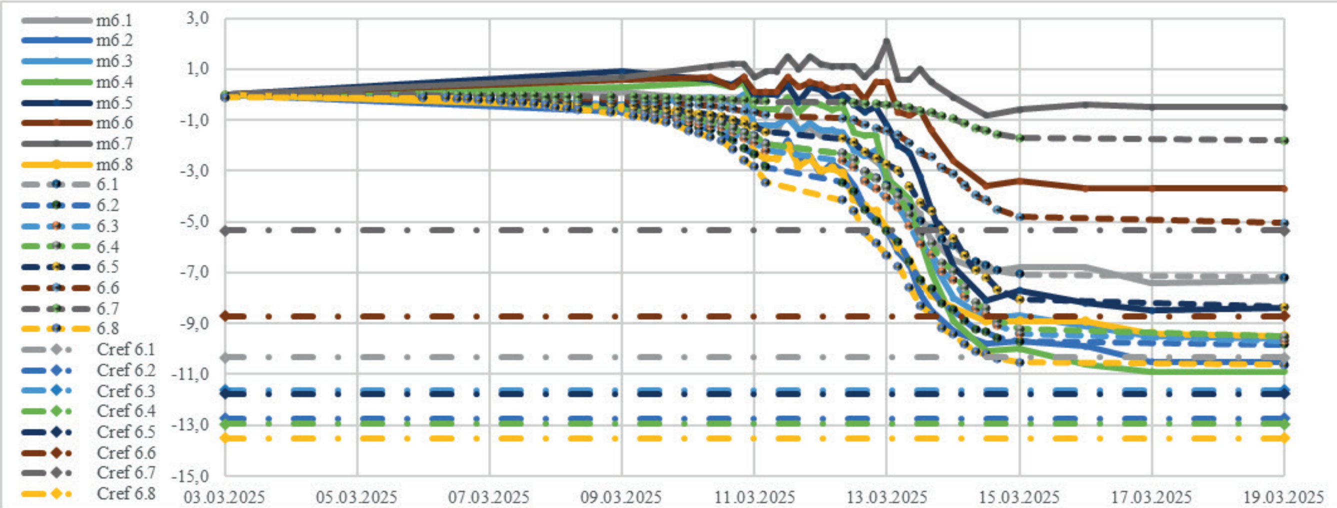
1st model borehole		2nd model borehole	
Soil elements	*h / m*	Soil elements	*h / m*
Anthropogenic layer (1-40)	0,8	Anthropogenic layer (1-40)	6,0
Medium sands (10-5-3-2)	11,3	Fine sands (15-5-4-2)	10,6
Loams with grus and crushed stone (24-3-3)	3,2	Fine sands (25-5-4-1)	11,0
Fine sands (25-5-4-1)	4,5	Limestones (51-16-3)	1,4
Silty sands (25-5-5-1)	6,2	Clays (52-2-2)	6,6
Fine sands (25-5-4-1)	4,1		
Silty sands (25-5-5-1)	2,4		
Fine sands (25-5-4-1)	1,6		
Medium sands (25-5-3-1)	0,9		
Silty sands (25-5-5-1)	0,9		
Medium sands (25-5-3-1)	1,6		
Clays with sand pockets (41-2-2)	5,1		
Clays with faunal remains (43-2-2)	2,8		

For the first computational scheme, the plot in Fig. 5 is given for the building at Parshina Street, 21 bldg. 2. The plot includes the measured ground-surface displacements. To generate the first plot, the timing of the actual installation of the tunnel lining rings was aligned with the corresponding installation phases in the numerical model, which enabled a proper comparison of the observed building deformations with the calculated time histories.

4. DISCUSSING

The plot in Fig. 5 shows that the alternative method provides sufficient agreement in determining the maximum displacement: the difference is about 0.27 mm, or 2.5% (throughout the article, comparisons are made relative to the maximum observed ground-surface displacement; for this plot it is 10.9 mm). At the same time, the maximum deviation at individual points is 1.4 mm, or 12.8%, at point 6.4, which corresponds to the permissible error of in-situ measurements of vertical displacements of 2 mm for Class II accuracy in accordance with [26], as well as to external factors that may affect the absolute measurement values, including temperature, vibration, etc. In addition, the plot shows that the values obtained from the calculation using the volume loss method are higher than the observed values at every monitoring point, with a maximum difference of 5.0 mm, or 45.9%, at point 6.7, which may lead to an incorrect assessment of the geotechnical situation and to unnecessary additional compensation measures. The difference between the maxima is 2.6 mm, or 23.9%. It should be noted that the calculation using the volume loss coefficient is much simpler and less computationally intensive and gives results with sufficient agreement for a preliminary analysis. However, the volume-loss approach does not explain the physical mechanisms that cause ground-surface settlement. Similar results were obtained in [1].

However, different patterns are observed for the plot in Fig. .6 The alternative calculation likewise agrees with the measured displacements, with a difference between the maxima of 3.1 mm (13.6%), and a maximum pointwise deviation of 4.3 mm (18.9%) at point 15.3. By contrast, the results obtained using the volume-loss approach are substantially lower than the observations: the difference between the maxima is 5.5 mm (24.1%), and the maximum pointwise deviation reaches 12.3 mm (53.9%) at point 15.3. To reconcile the measured displacements with those predicted by the volume-loss method in this case, the volume-loss coefficient must be increased to 2.85% (obtained by back-analysis). The discrepancy between the adopted coefficient and the back-calculated value may indicate the influence of additional unaccounted factors on the effective volume loss relative to the design assumption. It should be noted that an inaccurate assessment of the geotechnical conditions in such circumstances may lead to operational incidents during tunneling and necessitate complex and costly remedial measures.



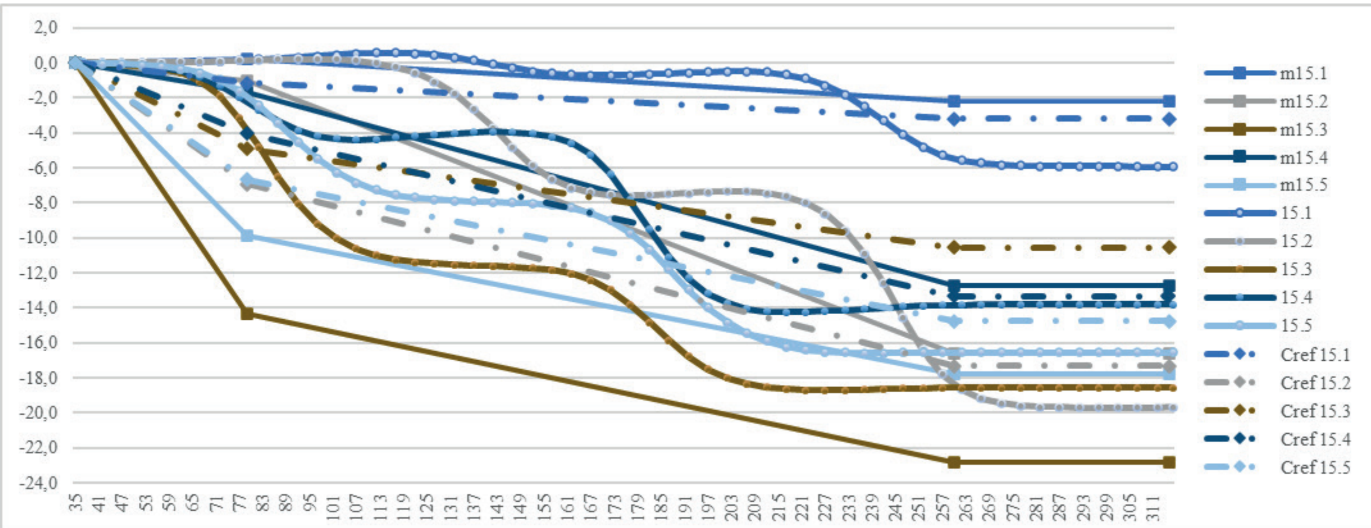
<u>Figure 5.</u> Displacement plot for the building shown in Fig. 3a

m6.1–6.8 — monitoring results

6.1-6.8 – staged modeling of all tunneling processes

Cref 6.1-6.8 – calculation using the volume-loss coefficient

Furthermore, comprehensive stage-by-stage modeling may reveal uplift of surface structures at certain points, caused by the structural stiffness of the buildings – as reported in [3, 5] – or to incorrectly specified face-support pressure or tail-void grouting pressure. This deformation mechanism is not reflected in calculations based on the volume loss coefficient.



<u>Figure 6.</u> Displacement plot for the building shown in Fig. 3b

m15.1-15.5 – monitoring results

15.1-15.5 – staged modeling of all tunneling processes

Cref 15.1-15.5 – calculation using the volume-loss coefficient

5. CONCLUSIONS

The proposed method exhibits a high degree of qualitative agreement with the monitoring results. Discrepancies in quantitative agreement may be attributable to measurement error in field observations, inaccuracies in the input data for the analysis, and other external factors. A more rigorous treatment of input parameters – such as geological conditions – together with proper modeling of structural behavior and higher-accuracy geotechnical monitoring, would enable closer quantitative agreement.

Consistent with Liu H., Shi J., Li J., and Liu C. [25], the calculation shows uplift of the ground surface as the shield passes the monitoring point and significant settlement after the shield has passed, which corresponds to the actual behavior of the ground during tunneling. The uplift effect may be caused by the stiffness of the building, as well as by improperly specified face support pressure when the TBM operates close to the surface. The volume-loss coefficient does not describe the mechanics of ground behavior during TBM excavation.

If additional, unaccounted factors influence the selected design value of volume loss, calculations based on that coefficient can differ substantially from the results of the alternative method presented in article.

It can also be observed (Fig. 5 – plots Cref 6.1–6.8; Fig. 6 – Cref 15.1–15.5) that the volume-loss approach allows the building-wide maximum displacement to be estimated with reasonable accuracy, whereas displacements at individual points may be entirely inconsistent with their measured values; by contrast, the alternative calculation yields closer agreement with the measured displacements at individual points on the structure.

Note that in [17–21], calculations using a prescribed volume-loss coefficient in accordance with the regulatory standards do not allow surface settlement to be predicted accurately. Accordingly, the only way to obtain an actual volume-loss coefficient is to adjust it by back-analysis using the already constructed section of the alignment.

Results obtained with the volume-loss coefficient differ from monitored values for the structural elements by 24.1%, which suggests that the coefficient is suitable for preliminary, approximate calculations.

For refinement under complex geotechnical conditions, it is recommended to verify results using the method presented in present article, which models all stages of the tunneling process.

Further research is required to refine the calculation methodology and to ensure the necessary accuracy in predicting ground-surface settlement and the response of adjacent structures.

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