Prediction of land surface subsidence in the U2 metro tunnel in Vienna

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ABSTRACT

This paper discusses the numerical and probabilistic modelling of the tunnel construction concerning some not yet completed sections of the Vienna U2 metro line. It presents the algorithm and results analysis of numerical simulation for the step-by-step tunnel construction using the New Austrian Tunnel Method (NATM) in a dense urban environment. The nature and magnitude of subsidence of the earth surface depending on a number of factors involved in the calculation scheme are determined, and all parameters of the stress-strain state of the system "tunnel - ground mass" are obtained. A methodology is proposed for determining reliability by the criterion of additional vertical subsidence of the ground surface which accompanies underground construction. By comparing the results of numerical modelling, empirical calculation and geotechnical monitoring of the metro construction site, it is shown that they correlate well enough with each other. The results of this study can be used to predict the level of the ground settlement during tunnelling works in areas of dense urban development.

Keywords: numerical modelling; geotechnical monitoring; surface subsidence; system "tunnel – ground mass"; reliability

1. Introduction

The steady increase in the number of urban residents and the growing urbanisation are an incentive for additional transport routes and the expansion of the metro network. Often new metro lines are located in areas with an existing dense urban development and there are certain risks that need to be minimised during the designing and tunnelling phases. One of the many risks arising during tunnelling and tunnel operation in compact urban areas is the ground deformation, that is, the subsidence of the ground surface. Here are some examples from recent years that vividly illustrate the problem.

1.1. Lessons from accidents during tunnel construction

1.1.1. Heathrow Express Metro, 1994

By the time of the October 1994 accident, work on the Heathrow Express Underground had already been underway for a year. Tunnellers had begun drilling two tunnels beneath London Airport, spraying liquid concrete on the tunnel walls as a supporting lining. In the morning of 21 October 1994, large cracks appeared in one of the tunnels, pieces of concrete fell out and the tunnel collapsed; fortunately, no human life was lost. During the next three days the tunnel continued collapsing under the entire airport, which resulted in ground failures on the airfield and between the runways, as well as in a damage to several buildings (Fig. 1). It was decided to make a concrete "plug" as a spur-of-the-moment solution, in order to prevent the spread of destruction. During the trial it was established that in the absence of these measures, the collapse could have completely destroyed the Piccadilly line, which would have resulted in human casualties. The damage repair works totalled £ 150 million, which is three times the cost of the project budgeted for the Heathrow Express underground section.

Figure 1. Tunnel roof collapse in Heathrow central area during the construction of the rail link to Paddington (See Airnews copy)

1.1.2. Munich, 1994

On the evening of 20 September 1994, a sinkhole of approx. 10 x 10 m was suddenly formed in front of the Trudering High-Speed Railway station; into this sinkhole a passenger bus fell, killing three people and injuring 34 (Fig. 2). During the construction of metro line 2, as the expert report would state later, "unrecognised and undetected sand cracks in the impermeable marl layer" appeared, through which groundwater penetrated through the clay layer into the underground tunnel.

Figure 2. A sinkhole swallowing a bus (source: picture-alliance/dpa)

1.1.3. Cologne Archives, 2009

On 3 March 2009, the Historical Archive of the City of Cologne collapsed (Fig. 3), killing two people who were residents of neighbouring houses. At a depth of 28 m, the construction of the underground shaft of the new Cologne North-South railway was underway. It is assumed that the ground failure was caused by a groundwater breakthrough into the tunnel under construction, located under the archive and causing the suffosion. The total damage is estimated at € 1.33 billion.

Figure 3. Collapse of the archive building in Cologne (https://www1.wdr.de/radio/wdr5/sendungen/morgenecho/stadtarchiv-112)

1.1.4. Warsaw Metro, 2012

On 14 August 2012, a tunnelling machine hit a quicksand near the Vistula River, a horizon not marked on geological maps. The groundwater rushed into the tunnel and filled the entire space with mud, up to the future Powiśle metro station (Fig. 4). All construction work was immediately stopped, fortunately no one was injured.

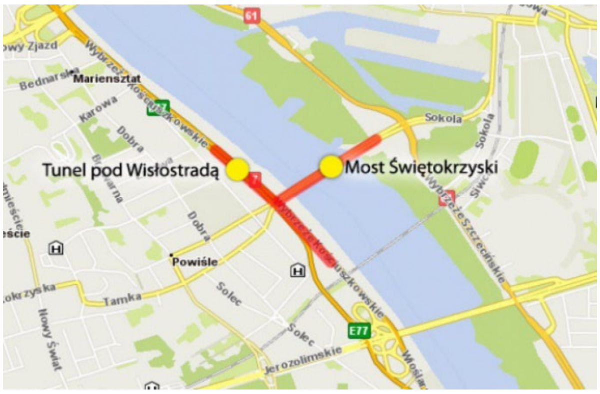

Figure 4. The site of tunnel collapse during the construction of metro line 2 in Warsaw (https://tvn24.pl/polska/zawal-na-budowie-metra-w-stolicy-zamkniety-most-i-wislostrada-ra270906-ls3504256)

1.1.5. Berlin – Alexanderplatz, 2022

At the Alexanderplatz, Covivio company is constructing a complex of multipurpose buildings (Fig. 5), including twin towers 130 m high; the 17 m deep foundation pit for them is only two metres away from the U2 metro tunnel (Fig. 6). Since the beginning of October 2022, the underground station sensors have detected a subsidence of the tunnel by more than 3 cm; a part of the route has been closed and a rail replacement bus service organized. The barrier wall between the metro tunnel and the excavation is one metre thick, but it had been deformed under the groundwater pressure (Fig. 7). As a result, the metro tunnel, too, underwent deformation – cracks in the tunnel were discovered in July 2022, and in September 2022, groundwater penetrated the tunnel floor. After the subsidence reached 3.5 cm, the decision was made to disconnect the Pankow route from the network because the stability of the tunnel was threatened and there was a risk of metro trains derailing. In February 2023, subsidence had already reached 3.8 cm. Stabilising works began in March 2023; it was decided to stabilise the tunnel by injecting cement slurry into the ground through d = 6.5 cm pipes, with a total of 60 injectors installed in the area under the tunnel (Fig. 8).

Figure 5. A visualization of the construction project https://www.covivio.immo/office/alexanderplatz/#expose

Figure 6. The construction site (https://www.rbb24.de/panorama/beitrag/2023/02/u2-covivio-alexanderplatz-beton-lanzen-tunnel-baugrube.html)

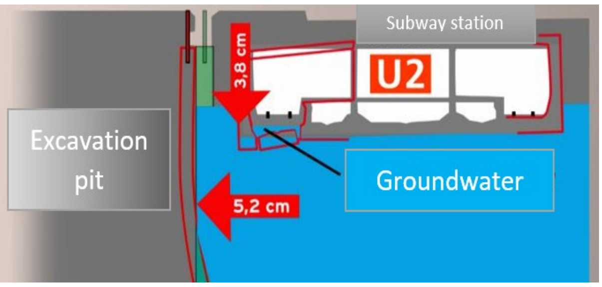

Figure 7. Deformation diagram of an underground tunnel (https://www.rbb24.de/panorama/beitrag/2023/02/u2-covivio-alexanderplatz-beton-lanzen-tunnel-baugrube.html)

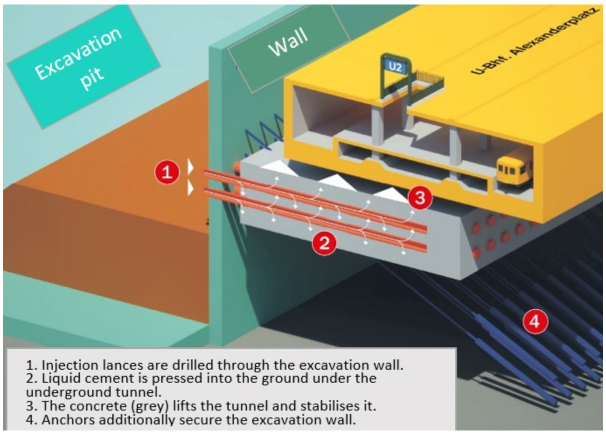

Figure 8. Visualisation of the ongoing metro tunnel rehabilitation (© graph: Tsp/Klöpfel. source: Covivio, February 2023)

1.1.6. Kyiv Metro, 2023

In December 2023, six metro stations in Kiev were closed due to tunnel leaking and water flooding the tracks. The official wording is as follows: uneven decompaction, vibrocreep ground deformation and overloading led to the threat of tunnel collapse. Significant leaks and cracks appeared on the tracks and tubes of the tunnel frame as early as November 2023 (Fig. 9). Large cracks in the asphalt and ground failures were noticed on the surface after the closure of the stations (Fig. 10). The tunnels between the stations "Demiivska" and "Lybidska" were constructed in the floodplain of the Lybid River in 2008 –2010 through underground boring under difficult geotechnical conditions, with artificial drainage. According to preliminary estimates, the restoration works will last about 6 months, and possibly more.

In order to clarify the reasons that led to the failure state of the tunnel, boreholes will be drilled to carry out geological and geodesic works, as well as to check adherence to compliance requirements during the construction.

Figure 9. Cracks in the emergency section of the metro https://telegraf.com.ua/kiev/2023-12-08/5821615-pidtoplennya-i-zakrittya-metro-u-kievi-shcho-vidbuvaetsya-naspravdi-i-skilki-trivatime-transportniy-kolaps

Figure 10. Cracked asphalt and dips above the emergency tunnel (https://telegraf.com.ua/kiev/2023-12-17/5822993-nad-zatoplenoyu-stantsieyu-metro-u-kievi-nova-np-eksklyuzivni-foto-ta-video)

2. Modelling and calculation

2.1. Projected section of the U2 metro in Vienna

The Vienna Metro is in constant development, with five expansion phases in the last 45 years. One of the latest stages is the construction of the U5 line and the modification and extension of the U2 line. This project has the highest difficulty level, being carried out in the context of dense above- and underground urban development. Depending on the ground conditions along the construction route, tunnelling is carried out by open-cut tunnelling (where possible), or, on the difficult sections, by boring using a TBM (Tunnelling Boring Machine), as well as the New Austrian Tunnelling Method (NATM).

The geomorphological structure of the Vienna area is a complex of Quaternary abrasion-erosion terraces descending progressively towards the Danube and intersected by stream and river valleys. The Vienna terraces do not consist of pure gravel; they also include sand, silt and clay, most often in the form of thin lenses that extend sideways for several hundred meters. The geological structure of the city territory includes sedimentary rocks of Quaternary and Neogene age. The metro structures are located in Miocene sedimentary layers. The lower ground layers of the construction site are Neogene sediments of Miocene age, overlain by Quaternary Danube sediments of Pleistocene age; therefore, the following rock types were identified: 1) Danube quartz gravel (QG3); 2) Miocene sediments (MZ). In turn, the MZ layer is subdivided into the following engineering geological elements (Figure 11):

– MZa (siltstone with higher clay content);
– MZb (siltstone with lower clay content);
– MZc (dusty fine and medium sand);
– MZd (coarse sand and gravel).

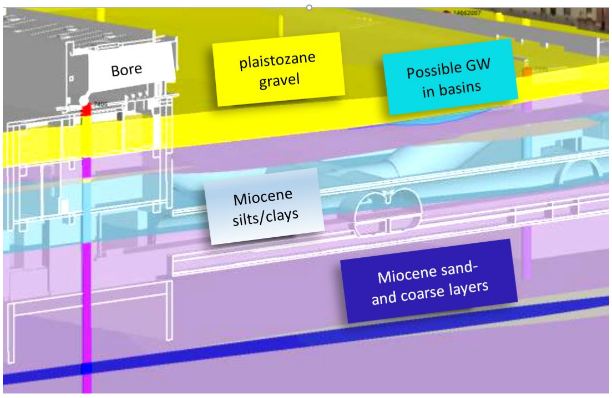

Figure 11. A subterranean model of the station Matzleinsdorfer Platz (Krepper et al, 2022)

2.2. Soil model and calculation schemes

The construction site under consideration was calculated using the PLAXIS 2D software introducing soil and structures loads, as well as additional uniformly distributed strip load of 20 kN/l.m, generated by vehicles and pedestrians on the–surface. For our simulation we choosed a variant using the Hardening Soil (HS) model, which is an advanced elastic-plastic model with isotropic stiffening. The actual stiffness characteristics are calculated by PLAXIS software on the basis of obtained stresses, taking into account the state of the soil according to a hyperbolic stress-strain relationship.

In order to understand the impact of the tunnel on the magnitude of vertical displacements on the ground surface, calculations have been carried out for stage of construction of the underground railway and the station Matzleinsdorferplatz in the area under consideration: a single 10.4 m diameter tunnel at a depth of 20 m. Dimensions of the soil massif for our calculation models are x x y =100 x 40 m. Parameters of soils put in the calculation model are shown in Table 1; 15-node finite elements are taken as the basic type. The upper soil layer is bulk soil with thickness 5 - 9 m, the second layer is clay soil with thickness 31 - 35 m. It should be noted that all soil parameters were established by testing in a specialised geotechnical laboratory, and all physical-mechanical and hydraulic characteristics for the calculation are based on the test results. The tunnel was bored using the New Austrian Tunnelling Method (NATM); the main properties of the tunnel lining are: longitudinal stiffness EA1 = $6 \cdot 10^6$ kN/m, flexural stiffness EI = $20 \cdot 10^3$ kNm^2/m, Poisson's ratio v = 0.15.

After carrying out all the necessary procedures, all the parameters of the stress-strain state of the soil massif and tunnel structure were obtained (Fig. 13). The parameter of vertical deformations of the ground surface (subsidence cavity) u_y will be considered below.

Table 1. Soil characteristics used in the calculations (the data are based on the Wiener Linen report, see References)

Name	Units	Designation	Bulk landfill	Silt, clay
Weight density of soil	kN/m^3	γ_{unsat}	19	20
Weight density of water-saturated soil	kN/m^3	γ_{sat}	20	21
Secant stiffness modulus	kPa	E_{50}^{ref}	$2 \cdot 10^4$	$3.2 \cdot 10^4$
Oedometer tangent stiffness modulus	kPa	E_{oed}^{ref}	$1.8 \cdot 10^4$	$2.5 \cdot 10^4$
Young´s modulus for unloading and reloading	kPa	E_{ur}^{ref}	$7.5 \cdot 10^4$	$14 \cdot 10^4$
Poisson's ratio	-	v'_{ur}	0.3	0.3
Degree index	-	m	0.5	0.5
Specific cohesion	kN/m^2	c'_{ref}	0	30
Internal friction angle	°	φ'	25	22.5
Dilatancy angle	°	ψ	0	0
Pressure	kN/m^2	p_{ref}	100	100
Drainage type			drained	drained

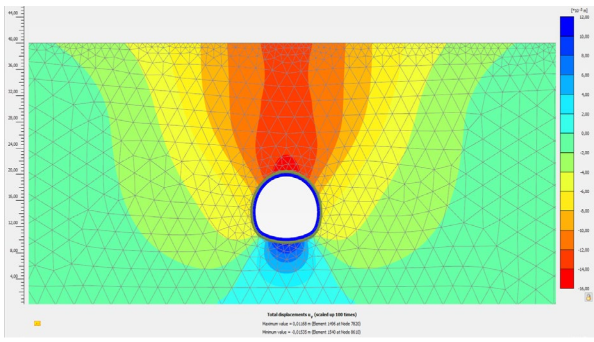

Figure 12. Results of FEM calculations – distribution of vertical deformations in the soil mass

2.3. Proposed algorithm for estimating the probability of exceedance for vertical deformations

This study proposes a methodology to assess the probability of exceedance for vertical ground surface deformations during tunnelling and the impact of the deformations on surrounding buildings. In designing and constructing underground structures in dense urban areas, the safety of surrounding buildings is one of the main requirements, while on the other hand the amount of additional deformations allowed for existing buildings is always smaller than for a new construction. Therefore, when designing any underground structures, design decisions are necessary that ensure the reliability not only of the underground structure itself, but also of neighboring buildings. To quantify the risk for existing buildings, a limit for additional building settlement the can be used as a criterion; it should not exceed a certain permissible level. A viable option here is the method of statistical tests (Monte Carlo method) using the calculations results obtained with the PLAXIS programme. The Monte Carlo method allows taking into account the stochastic nature of loads and impacts, as well as soil properties, which is impossible in a deterministic calculation. The Monte Carlo method of statistical tests (Monte Carlo simulation) is a numerical method of solving mathematical problems based on modelling random variables and conducting a significant number of statistical tests using random variables according to known distribution laws. The method of statistical tests is quite universal and is used to assess the reliability and safety of building structures and foundations when solving problems of reliability theory based on modelling random variables and constructing statistical estimates, as well as to develop possible (probable) processes or results using various methods of random number generation, which are then applied to a real model. The Monte Carlo method is successfully implemented in geotechnical design practice (Pereia, 2011; Misra, 2009; Chalermyanont and Benson, 2004; Baecher and Christian, 2003 etc.).

1. Let us consider the theoretical background. In 1969, Peck suggested that the ground settlement follows the law of normal distribution; he came to this assumption by summarising the measured data of ground settlement caused by a large number of tunnel construction at that time (fig. 13). The following formula can be used to calculate surface settlement:

$$S_{(x)} = S_{max} e^{-\frac{x^2}{2i_x^2}}, \quad (1)$$

$$S_{max} = \sqrt{\frac{\pi}{2}} \cdot \frac{V_L D^2}{4 i_x}, \quad (2)$$

Where $S_{(x)}$ is the settlement at the offset distance x from the tunnel centerline, that is, the maximum settlement above the tunnel centerline, and i is the width coefficient of the surface settlement trough, V_L is the volume of settled soil above the tunnel as a result of mining operations. O'Reilly and New (1982) proposed the following simple relationship for the value of i_x:

$$i_x = K \cdot z, \quad (3)$$

where z is the depth from the ground surface to the tunnel centre line, and K is a dimensionless constant depending on the type of soil (K = 0.5 for clay soils and 0.35 for sandy soils).

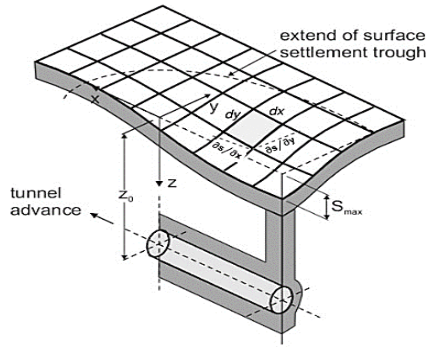

Figure 13. Surface subsidence geometry caused by tunnelling (Franzius, 2003)

There is also a modification of Peck's formula as outlined in Xiaowu (2021).

2. Considering formulas (1) and (2), we note that the quantities included are mostly random, especially S_{max}, which directly depends on the soil properties and external influences, or, in other words, $S_{max} = f(\gamma, \varphi, E, q)$. Therefore, when performing numerical calculations (PLAXIS), coefficients of variation were set for some soil and load characteristics, which were adopted in combination with the laws of random distribution according to the JCSS Probabilistic Modelling Code, according to the studies of Phoon and Kulhawy, 1999, Griffiths and Fenton, 2007 and displayed in Table 2. The range of values was pre-defined in Mathcad Prime software environment.

Table 2. Coefficients of variation and distributive laws adopted for the calculation

Parameters	Designation	Distribution	COV
Weight density	γ	normal	0.05
Angle of friction	φ	normal	0.1
Secant stiffness modulus	E_{50}^{ref}	lognormal	0.2
Oedometer tangent stiffness modulus	E_{oed}^{ref}	lognormal	0.2
Young´s modulus for unloading and reloading	E_{ur}^{ref}	lognormal	0.2
Strip load	q	normal	0.1

3. As a result, the values of vertical deformations S_{max} were obtained, with the coefficient of variation v = 0.24 (24%); the value of S_{max} is distributed according to the normal law.

4. To assess the reliability by the criterion of limit exceedance for the additional vertical settlement of the ground surface resulting from the tunnel construction, we will use the Monte Carlo statistical test method.

The following laws of distribution for input parameters are assumed:

1) distribution of the random variable $P_{S_{add}} = P_{S_{add}}(S_{add})$ of maximum permissible additional vertical deformations, following the normal distribution law with mean value $m_{S_{add}}$ = 0.03 m and standard deviation $\sigma_{S_{add}}$ = 0.003 m. The adopted mean value is

based on the monitoring of buildings and structures, as well as the analysis of some current studies and regulatory documents.

2) The distribution of the random variable $P_{S(x)} = P_{S(x)}(S(x))$ of design deformations, assumed following normal distribution law with $m_{S(x)}$ and $\sigma_{S(x)}$, respectively. The values of this parameter are determined first for S_{max}, and then calculated by formula (1).

5. According to the Monte Carlo method, N statistical tests (in our case N = 10^5) are conducted according to the following algorithm.

1) Determination of random values of vertical deformations S_{max} (FEM).

2) Calculation of the surface settlement $S_{(x)}$ using formula (1).

3) In each case, the condition is checked $S(x) \leq S_{add}$.

4) After all N tests have been performed, the probability of the limit exceedance for additional deformations during the design life is calculated as the ratio of the number of tests in which S_{add} - $S(x) < 0$ to the number of all tests.

5) A similar calculation is also performed for an operating period of 50 years (*S(x)T*).

Figure 14 shows the cumulative distribution function of random variables of ground vertical deformations: *S(x), S(x)T*, S_{add}.

6. According to EN0, the value of the probability of reaching the limit state (the probability of structural failure) P_f can be obtained from the formula:

$$P_f = \Phi(-\beta), \qquad (4)$$

where Φ is the cumulative distribution function of the standardised normal distribution, β – reliability index (or safety characteristic).

7. Basically, the surrounding development is mainly in the CC2 impact class, for which the minimum value β is 2.9 for a time period of 1 year and β = 1.5 for a time period of 50 years (Table C2 of EN1990:2002) – Table 3.

Table 3. Results of probabilistic calculations of exceeding for additional ultimate deformation of the base (above the line are values for a period of 1 year, below the line are values for a period of 50 years)

The name of the values	Distance from the longitudinal tunnel axis		
	0 m	10 m	20 m
The probability of exceeding the additional limit deformation of the base	2 x 10^{-3}	0	0
	5 x 10^{-3}	0	0
Recommended EN minimum values (SLS) (normal level)	3.2×10^{-3}		
	8.1×10^{-2}		
Reliability index β	3	5.2	5.2
	2.75	5.2	5.2
Recommended minimum values (SLS) (normal level)	2.9		
	1.5		

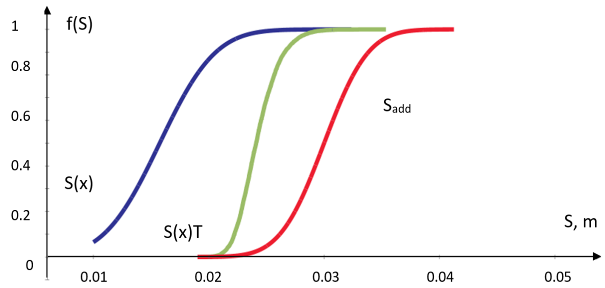

Figure 14. Cumulative distribution function of random variables of ground vertical deformations: blue curve - additional settlements for a year *S(x)*, green curve – additional settlements for 50 years *S(x)T*, red curve – maximum permissible additional vertical deformations S_{add}.

3. Geotechnical monitoring carried out during the construction of the U2 metro line in Vienna

During the construction of the tunnels and the Matzleinsdorferplatz station of the U2 underground line extension from 2018 onwards, careful geotechnical monitoring of various objects on the construction site is being carried out. Inclinometers are used to perform the monitoring, which measure the deformation of the ground and structures in real time. The construction site is quite extensive, let us consider and analyse the deformations that occur during the tunnelling of the U2 underground tunnel in a small section of the building. The measurement points 180131-180133 are located above the deep support ÖBB and above the future tunnel T2 (fig. 15); values of vertical deformations – fig. 16.

Monitoring data shows that subsidence of the ground surface above the tunnels ranges from 1 to 25 mm, with maximum deformation values of 25 mm for the southern shaft and 15 mm for the northern shaft.

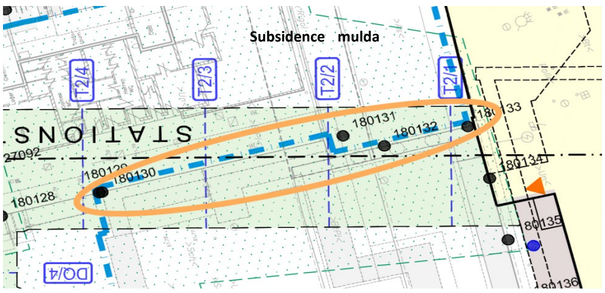

Figure 15. Diagram of inclinometers installation on the section (black points) (Report «Bericht über die Interpretation der bisherigen Setzungsmessungen U2/18», 2021)

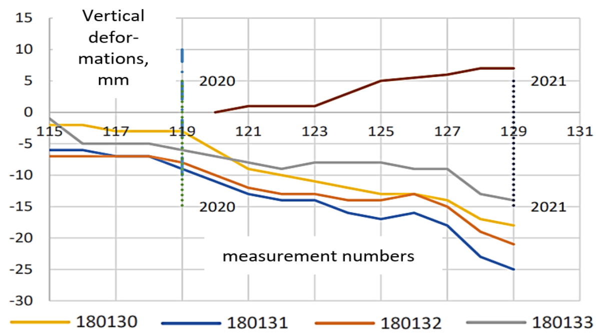

Figure 16. Results of vertical deformation monitoring at points 180131-180133 (Report «Bericht über die Interpretation der bisherigen Setzungsmessungen U2/18», 2021)

4. Resalts and discussion

The deformation pattern obtained by empirical calculation and numerical calculation (PLAXIS) are compared with the results of geotechnical monitoring (Fig. 17). Undoubtedly, the outline of the subsidence cavity and the maximum strain values have a similar character, while it is noted that the width of the subsidence cavity (transverse dimension) obtained from the FEM calculation taking into account the variability of initial data results is slightly smaller.

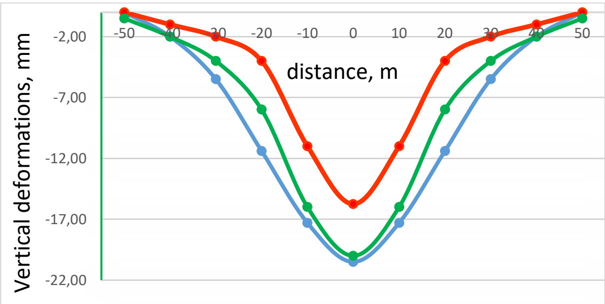

Figure 17. Graphs of vertical ground surface deformations above a tunnel: red curve – FEM calculation taking into account the variability of initial data; green – monitoring results; blue – empirical method

5. Results and discussion

The main results of this study can be summarised as follows:

1. One of the main causes for accidents during tunnelling is an insufficient study of the geotechnical conditions at the construction site.

2. The other cause is insufficient risks consideration. When designing tunnels, probability calculations for hazardous situations should be made; reliability should be determined numerically, using a parameter.

3. For structures and complex constructions of class CC3 of responsibility, a probabilistic calculation of reliability should be provided, in addition to the double parallel calculation (using different calculation schemes and computer programmes. The probability of failure for each specific possible failure event should also be calculated and ranked.

4. In order to understand the possible hazardous effects of earthworks on buildings and structures located in the subsidence mulch area, buildings should be grouped according to their sensitivity to ground surface movements. In this process the current technical condition of the building should be taken into account.

5. Another crucial step should be a predicting of the subsidence of the ground surface during underground works. In the FEM calculation, a range of values for vertical deformations of buildings during tunnel construction was obtained and reliability calculation by Monte Carlo numerical modelling using the additional settlement criterion was carried out. A calculation algorithm using the additional settlement criterion was proposed. This approach takes into account the stochastic nature of loads and impacts, as well as the heterogeneity of soils, whose physical and mechanical properties vary both in depth and along the strike.

The probability of failure was 0.002 (period 1 year) and 0.002 (period 50 years) and the reliability indexes equal $\beta_1 = 3$ and $\beta_{50} = 2.75$, which is an acceptable value for the SLS condition.

6. The ground surface subsidence along the tunnel is at most 25 mm (based on monitoring results), and corresponds well with the Gaussian function as well as with the FEM calculation results (taking into account the variability of initial data); the discrepancy between the results obtained does not exceed 20%.

7. In the numerical FEM calculation, the subsidence of the ground surface in the case of the tunnel ≈ 16 mm.

8. The width parameters of the subsidence cavity are i ≈ 18.0 m according to the FEM calculation and about 19 m according to the empirical calculation. The numerical calculation shows that the width of the subsidence cavity is smaller than in the monitoring results.

9. The proposed approach can be applied for preliminary assessment of the subsidence level of the ground surface during tunnelling – the most sensitive areas can be considered in a flat setting with a simplified load setting. Without doubt, in a complex geotechnical situation, the soil mass section with the designed underground structures should be modelled in 3D. However, such an approach is necessary to understand threats posed by underground structures to buildings on the surface.

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