Modeling of tunneling-induced ground surface movement

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Abstract. Surface subsidence control during tunnelling in urban areas is required to minimize any disturbance to nearby buildings and services. The main focus of previous studies concerning surface subsidence was basically on empirical solutions derived from field research. The number of analytical studies which have been carried out is not enough because they do not consider complex ground conditions; hence, it is required to model the effect of surface subsidence caused by tunnelling based on comprehensive analytical solution combined with numerical modelling. This paper presents the results of tunneling-induced surface settlement modelling based on finite element method. The effect of the overconsolidation ratio of soils expressed in terms of the coefficient of earth pressure at rest ($K_0$) on surface subsidence caused by tunnelling is investigated. It is demonstrated that value of $K_0$ is an essential factor affecting accuracy of consolidation settlement estimate in overconsolidated clays.

1. Introduction

Ground surface settlement caused by tunnelling in soft ground is one of the basic issues in all aspects of tunnel design. Surface subsidence can be caused by several factors such as ground loss at the tunnel face, behind the tail of the shield and through the tunnel support or linings. Other factors include the consolidation of the soil due to reduction of ground water level. Besides surface settlement, tunnelling also produces lateral deformation of the ground and longitudinal movement of the ground ahead of the tunnel face. To control the effects of deformation various field studies and experiences are carried out, empirical and numeric methods are developed.

The semi-empirical approach described in many references [8] is applied to estimate the size and shape of the settlement trough at a “greenfield site”. These solutions are based on field observations. The equations used are based on the assumption that the settlement profile above a single tunnel is of normal probability or Gaussian form. To define the surface settlement trough the following parameters are used: volume loss $V_s$, point of inflexion $i$ and maximum surface settlement $S_{max}$ the settlement profile. The Gaussian curve is used at all levels in the ground above the tunnel. For the combined effect of multiple tunnels, the movements induced by each tunnel are simply added.

Numerical methods have become much more common in the analysis of excavations in urban environments. Such a numerical technique as FEM analyzes heterogeneous ground layers by means of more sophisticated constitutive models, involving time dependent effects, initial and boundary conditions which are similar to the actual field ones. They are particularly effective for the study of tunnels excavated in grounds that can be modelled as continuous media, with due account of non-linear behaviours, as well as complex staging and geometrical conditions.

It is important to select constitutive models for the soil capable of reproducing field behaviour. Linear elastic soil models usually give trough widths that are too wide. Most of the numerical
Simulations of tunnel excavation are performed with linear-elastic perfectly-plastic models without the influence of the nonlinear ground behaviour. Stiffness is the same for loading, unloading and reloading. Models based on hardening plasticity involving kinematic hardening are appropriate, as they can model the nonlinear elasticity at small strains, the effect of stress history and cyclic loading [9]. One of the great benefits of numerical analysis is that it enables to incorporate adjacent influences of surface structures or tunnels. It is also possible to model the effects of compensation grouting to protect surface structures during tunnelling projects. Numerical modelling is often carried out to get back-analysis of experimental data, to verify the input parameters that had been assumed in the planning and design stages.

As it is known, accurate determination of the in situ stress is an important issue in the numerical simulation of many geotechnical problems. The determination of lateral earth pressures is absolutely necessary at engineering structure design and application stages. Particularly, the knowledge of lateral earth pressure is required in tunnel, retaining structure and deep foundation design. Tunneling in soft ground always leads to stress rearrangements and deformations in the surrounding soil. The deformations can be detected by the evaluation of measurement data available from previous tunnel project sites – the empirical prediction – or by numerical calculations. Vertical stresses can be easily determined on the basis of depths, densities and groundwater data, but to calculate horizontal stresses is more complicated task. They usually have to be estimated using empirical values of $K_0$, the coefficient of earth pressure ‘at rest’ under conditions of zero lateral strain. The coefficient $K_0$ is the ratio of in situ horizontal effective stress to the vertical effective stress. $K_0$ is determined by field or laboratory tests. The coefficient of earth pressure at rest ($K_0$) changes depending on the relative density, stress history, over-consolidation ratio, plasticity index and similar soil properties. The over consolidated soil is a normal soil which has suffered a bigger load in the ancient geological time than nowadays.

According to the theory of elasticity, $K_0 = \nu/(1-\nu)$, where parameter $\nu$ – Poisson ratio, taken often according to recommendations. When applying the method of final elements with using of elastoplastic models, the calculations should be based on the laboratory and computing experiments data concerning model input parameters.

2. Experimental design

To provide a tool to improve the state-of-the-practice of controlling ground movements associated with tunnelling, this paper presents a numerical procedure for tunnelling that effectively and efficiently updates design predictions of surface settlements during construction. Several authors have studied the influence of $K_0$ in numerical tunnel analysis on surface settlements, e.g. Gunn (1993), Addenbrooke (1997), Franzius (2005), and stated that the settlements trough becomes wider and less deep with increasing $K_0$-value. In the present studies the non-linear elastoplastic Hardening Soil Model was applied to investigate the influence of $K_0$ on surface settlement and find parameters $K_0$ and OCR for modelling deformations around tunnels in Munich.

The city of Munich forms the core of a fast growing urban region in the Alpine foreland of Bavaria, in southern Germany. The subsurface is composed of Neogene and Quaternary formations made up of loose alluvial, fine- to coarse-grained sediments. Today’s landscape is known as the Munich gravel plain, which comprises sand terraces formed during the Pleistocene glacial periods, as well as the modern floodplain of the river Isar. Due to major unconformity and hiatus, these rather young gravel terraces overlie Neogene Molasse deposits of the Alpine foreland basin, which, in contrast, involve fine-grained fluviatile and lacustrine facies.

The subsoil in Munich has the following geological characteristics: uppermost Quaternary sediments comprise thick and extremely thick interglacial layers of various ages. Their thickness ranges from 4 to 7 m, whereas, in the zone of superimposed lower and upper terraces, it ranges from 8 to 25 m. These sediments are underlied by soft late-Tertiary sandstone known as "flinz". The layer composition varies significantly: thick fine to medium micaceous sand layers, practically impermeable clay marl layers with high variable monaxial compressibility of about 50 kN/m$^2$ for plastic lime-free clays and up to 6000 kN/m$^2$ for marl. The average compressibility value is about500 kN/m$^2$. In the
multi-Tertiary layers there is often a regular sandwich system within the soil layers. The layers have a wavy profile and sudden indentations can be observed in erosion trenches.

The Munich U-Bahn began operation in 1971. The Munich U-Bahn system currently comprises eight lines, serving 96 stations, and encompassing 95 kilometres of routes. The inverse analysis is carried out using data from a tunnel U-8 in Munich. The tunnel is approximately 14 m below the ground surface, with diameter being 7 m. The settlement parameters are modelled by 15-noded triangular elements in a plane strain mesh (Plaxis, Brinkgreve et al. 2004). Deformations of the tunnel and its overburden were studied using two material models: the hardening soil model (HS) and the hardening soil model with small-strain stiffness (HSS initial stress conditions were geostatic). Inclinometer data are used as observations in the inverse analysis that calibrates the numerical model in an objective way, the engineering judgments being made during the construction of the tunnel.

In this study, effects of the coefficient earth pressure at rest and over-consolidation ratio were investigated. There are different methods to determine the preconsolidation stress, \( \sigma'_p \), from laboratory oedometer data. Casagrande (1936) developed the most commonly used method and it is that method that was used. The table 1 shows the results of OCR estimation based on compression tests for neogene molasse deposits.

| Sample No. | Sampling depth, \([m]\) | \(\sigma'_z\), \([kPa]\) | Preconsolidation Stress, \(\sigma_{cm}'\), \([kPa]\) | OCR, [-] | Calculated mark of a surface, \([m]\) |
|------------|-------------------------|-------------------|---------------------------------|---------|-------------------------------|
| 01-60221   | 24,8                    | 356,8             | 1850                            | 5,2     | 124                           |
| 02-60189   | 28                      | 392               | 1675                            | 4,2     | 107                           |
| 03-60193   | 32,6                    | 442,6             | 2000                            | 4,5     | 130                           |
| 04-a-60194 | 33,4                    | 451,4             | 2075                            | 4,6     | 135                           |
| 04-b-60194 | 33,4                    | 451,4             | 2375                            | 5,3     | 160                           |
| 05-60195   | 35,5                    | 474,5             | 1925                            | 4,1     | 121                           |
| 06-60477   | 23,5                    | 342,5             | 1200                            | 3,5     | 71                            |
| 07-60479   | 32,1                    | 437,1             | 2100                            | 4,8     | 139                           |
| 08-60482   | 35,8                    | 478,4             | 1975                            | 4,1     | 125                           |
| 09-60494   | 28,5                    | 356,7             | 2325                            | 6,6     | 166                           |
| 10-62151   | 19,5                    | 250,5             | 1650                            | 6,6     | 117                           |
| 16-61741   | 25,8                    | 319,8             | 1750                            | 5,5     | 119                           |
| 17-62170   | 20                      | 292               | 2400                            | 8,22    | 179                           |
| 19-a-60052 | 33,6                    | 411,6             | 1850                            | 4,5     | 120                           |
| 19-b-60052 | 33,6                    | 411,6             | 2075                            | 5       | 139                           |
| 20-60146   | 23                      | 373               | 1875                            | 5       | 125                           |

In general, the data confirm the statements of geologists about an erosion of deposits in the Quaternary Period through the reconstruction of the alpine paleorelief.

Based on the various laboratory and field test results, many methods have been developed to determine the coefficient at earth pressure at rest of different soil types. The determination of earth pressure coefficient at rest by using laboratory methods has been preferred because of the simplicity of the laboratory tests compared to field ones.

Currently, \( K_0 \)-consolidation tests can be performed using an oedometer or a triaxial apparatus. Since the oedometer test has the disadvantages of unknown side friction as well as difficulties in measuring the radial stress, the triaxial apparatus more fully satisfies the \( K_0 \)-condition. Recent advances in control techniques and equipment encourage the easy use of triaxial \( K_0 \)-consolidation test.

Triaxial tests have been performed to determine the model parameters and provide data for validation of the model to predict the behaviour of soil. The results of estimation of coefficient of earth
pressure at rest ($K_0$) at $K_0$ consolidation for 20 samples of neogene molasse clay soil are given in fig. 1, 2. $K_0$ values that were obtained from empirical equations and values obtained from laboratory tests were compared.

![Figure 1. Variation of $K_0$ with OCR on unloading](image1)

![Figure 2. The confidence bands for OCR vs depth with confidence levels of 90% and 95%](image2)

Figure 3 shows the transverse settlement curves to demonstrate the influence of $K_0$ on the shape. The different surface settlement curves have been obtained for $K_0=0.6$, 0.8 and 1.0 respectively. A tremendous influence of $K_0$ on the magnitude of the maximum steady-state settlement is observed. The trough becomes shallower and wider with increasing horizontal initial stress. $K_0=1$ surface heave curves show that heave is primarily concentrated over the tunnel center line.
The construction of twin tunnels is a common requirement for underground railways and there are equations to make preliminary ground movement predictions. In practice, the tunnels will rarely be driven simultaneously and one tunnel is likely to be excavated before the other. In some cases this will give rise to an asymmetry which is not modelled by the equations.

It is generally assumed that the predicted ground movements for each tunnel can be superimposed. In relation to twin tunnels with reduced transversal distance between the axes, this assumption may be unconservative. The disturbance caused by the first tunnel drive can be simulated by assuming a greater volume loss for the second bore and superimposing the resulting ground movements.

Figure 4 shows side-by-side tunnels, the superimposed curve almost exactly matches the numerical results.

Figure 3. Influence of the $K_0$ value of soil on the calculated surface settlement

Figure 4. Predicted and observed ground settlement profile for side by side tunnels
$K_0$ coefficient variations are as follows: for sand 0.426; 0.6 and 0.8; for clay 0.576; 0.8 and 1.0. Vertical movements data concerning each settlement model from the PLAXIS program were taken to compare a calculated subsidence with different $K_0$ values and measured surface subsidence. As illustrated in fig. 2, the best coincidence with measured subsidence have settlement curve computed with $K_0$-OC-1 (for molasse sand – 0.6, for clay – 0.8).

**Summary**

The surface settlements appear to be sensitive to the $K_0$-value. For geotechnical calculations, $K_0$-value for overconsolidated soils - 0.6 (sand) and 0.8 (clay) are proposed.

1) The variation of horizontal stresses is linear. The vertical stresses are greater than pre-consolidation pressure in loading stage. Therefore, $K_0$ values are considered as constant for vertical stresses which are greater than pre-consolidation pressure.

2) It was observed that experimental $K_0$ values are greater than $K_0$ values calculated from empirical equations for normally consolidated state. However, experimental $K_0$ values are lower than calculated $K_0$ values for over consolidated state. $K_0$ values calculated for low over consolidation ratio values are close to test results.

3) In the unloading stage, horizontal stresses decreased slower than vertical stress, that’s why $K_0$ values increased. Therefore, $K_0$ values are considerably affected by over consolidation ratio in unloading stage. Since over consolidation ratio increases, coefficient of earth pressure at rest also increases.

Thus, the application of the special software allows engineers not only to reduce labor input but also significantly increase the reliability of the obtained results, which in its turn could contribute to design efficiency increase.

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