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Analysis of the settlement of an existing tunnel induced by shield tunneling underneath

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ABSTRACT

Due to the increasing development of infrastructure systems in urban areas of China, many subway tunnels are being constructed near existing subway lines. Shield tunneling underneath will inevitably cause settlement of the existing tunnel and may even threaten the safety of running trains. In this study, based on an analysis of extensive monitoring data from massive undercrossing construction cases in the Shenzhen metro area, the deformation characteristics of existing tunnels and the ground caused by the construction of a new shield tunnel underneath were analyzed. The additional longitudinal stress of the existing tunnel, which is the main reason for water seepage and structural damage of the tunnel lining, was studied. The difference between the settlement of greenfield and the existing tunnel was analyzed. Several key influence factors of existing tunnel settlement, such as the spatial position, support pressure and tunnel stiffness, were discussed and an empirical equation for estimating the settlement of existing tunnels induced by the excavation of a new shield tunnel was proposed. The settlement profiles obtained using the new prediction equation were compared with the real monitoring records, and good agreement was observed. The new equation derived from the cases studied in this paper can provide a reference for predicting and controlling the deformation of existing tunnels induced by shield tunneling.

1. Introduction

During the past few decades, an unprecedented subway construction boom has occurred in China, with a significant and continuing demand for infrastructure and transportation systems. In urban regions, the shield method is widely used to build new tunnels, and new tunnel excavation to fulfill or extend the planned subway network scheme will inevitably cause negative impacts on existing subsurface structures. Shield tunneling beneath existing tunnels is very common in China, and at least one hundred undercrossing projects have been conducted in the past 6 years. The construction of a new tunnel is a more delicate process due to the impact of ground deformation and the further influence of soil movement on the surrounding structures. To guarantee the safety, stability and durability of existing lines, the ability to accurately predict the displacement of existing tunnels is crucial.

Many studies have investigated tunneling-induced ground movements, and many empirical expressions have been presented in the past few years. The ground deformation caused by tunneling can be depicted as a transverse settlement trough, which is often estimated using empirical methods and approximated using a Gaussian distribution (Peck, 1969; Schmidt, 1969; Attewell and Woodman, 1982; Ocak, 2014; Ye et al., 2015). Many authors have adopted physical modeling methods to analyze tunneling-induced ground movement (Mair, 1979; Taylor, 1984; Grant, 1998). In addition, some proposals based on analytical methods have been published (Loganathan and Poulos, 1998; Sagaseta, 1987; Verruijt and Booker, 1996).

The behaviors of existing tunnels induced by adjacent tunneling have also been extensively studied. Some examples are as follows: Yamaguchi et al. (1998) presented a case of four shield tunnels built near each other and assessed the behavior of the tunnels and surrounding ground based on measured data. The response characteristics of the preceding tunnel to the thrust from the succeeding tunnel were studied, and an analytical expression was proposed to describe the ground movement mechanism near the running tunnels. Cooper et al. (2002) presented extensive monitoring records obtained by monitoring the interior of two existing tunnels with a diameter of 3.8 m during the construction of three running tunnels with an external diameter of 9.1 m located underneath in London clay (on the Heathrow Express). Li and Yuan (2012) studied the deformation of an existing double-decked tunnel during the construction of two shield-driven tunnels with an external diameter of 6 m located underneath. Fang et al. (2015) presented a case of closely spaced twin tunnels excavated beneath other closely spaced existing twin tunnels. A superposition method was adopted to describe the settlement profiles of both existing tunnels and

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Nomenclature

S_t, S_s	settlement of soil and existing tunnel
V	percentage of ground loss
K, K_t	empirical trough width parameter of soil and tunnel settlement
i	distance from tunnel centerline to inflection point of the settlement
N	stability number
γ_n	natural unit weight of the soil
z_0	tunnel-axis depth
σ_s	total surcharge pressure
EI_{eq}	equivalent longitudinal stiffness of tunnel
α	intersection angle of new and existing tunnels
σ_T	EPBM/SBM face pressure
C_u	undrained cohesion of soil
E_s	soil elastic modulus
k	coefficient of the subgrade reaction.
$\bar{\lambda}$	adjustment coefficient
V_t, V_s	ground loss obtained from soil and tunnel settlement curve
D	external diameter of tunnel
η	modified stiffness coefficient
A_s	section area of existing tunnel
C	structure-soil stiffness ratio

ground surfaces induced by the construction of the twin tunnels below.

However, knowledge of the response of existing tunnels to later shield tunneling beneath remains limited, and even less data are available on the relationship between existing tunnel settlement and ground settlement. In this paper, 18 cases of shield tunneling beneath existing tunnels in Shenzhen, China, are presented. The deformation characteristics of the existing tunnels and the ground caused by the construction of a new shield tunnel underneath were analyzed. In addition, a new method that includes several factors, such as the spatial position, tunnel stiffness and support pressure, is proposed to estimate the settlement of an existing tunnel due to shield tunnel construction below.

2. Empirical-analytical methods for ground movement caused by shield tunneling

The ground movements induced by tunneling are presumed to fit a Gaussian distribution, as established by Peck (1969) (Eq. (1)) and verified by a substantial number of field measurements and laboratory tests (Mair et al., 1993):

$$S = S_{\max} e^{\left(-\frac{x^2}{2i^2}\right)} = \frac{AV}{i\sqrt{2\pi}} e^{\left(-\frac{x^2}{2i^2}\right)} \tag{1}$$

where S is the theoretical settlement at a given horizontal level, S_{max} is the maximum settlement at the tunnel centerline, x is the lateral distance from the tunnel centerline, i is the distance from the tunnel centerline to the inflection point of the settlement, V is the percentage of ground loss, and A is the sectional area of the tunnel.

To estimate the value of i, Eq. (2) was proposed by O'Reilly and New (1982):

$$i = K \times z_0 \tag{2}$$

where z_0 is the new tunnel-axis depth, and K is an empirical trough width parameter with a value based on the soil condition: 0.2–0.3 in granular soils above the water level; 0.4–0.5 in hard, fractured clay; 0.5–0.6 in glacial deposits; and 0.6–0.7 in soft clay.

To describe the settlement curve in the subsurface, Eqs. (3) and (4) were proposed by Mair et al. (1982, 1993) to calculate the settlement trough width parameter after investigating available field measurements and centrifuge model test data. Han et al. (2007) suggested an empirical equation (Eq. (5)) for the excavation of a tunnel in different types of soils:

$$i = K(z) \times (z_0 - z) \tag{3}$$

$$K(z) = \frac{0.175 + 0.325(1 - z/z_0)}{1 - z/z_0} \tag{4}$$

$$K(z) = \frac{1 - a(z/z_0)}{1 - z/z_0} \cdot K \tag{5}$$

where z is the depth at a certain position in the ground, and the value of a is 0.65 for clayey soils and 0.5 for sandy soils.

The ground loss ratio V at the surface and subsurface can usually be considered equivalent, and many methods have been presented to calculate V induced by tunneling. Peck (1969) suggested a formula for estimating the ground loss, presented as Eq. (6). Based on field data, Arioglu (1992) found a positive relationship between V and N (the stability ratio) for face-pressurized TBM cases, as shown in Eq. (7) (Ercelebi et al., 2011):

$$V = \frac{3.192 \times i \times S_{\max}}{D^2} \tag{6}$$

$$V = 0.87e^{0.26N} = 0.87e^{0.26\cdot\left(\frac{\gamma_n \cdot z_0 + \sigma_s - \sigma_T}{C_u}\right)} \tag{7}$$

In Arioglu's equation, N is the stability number, γ_n is the natural unit weight of the soil (kN/m^3), z_0 is the tunnel-axis depth (m), σ_s is the total surcharge pressure (kPa), σ_T is the EPBM/SBM face pressure (kPa), and C_u is the undrained cohesion of the soil (kPa), which can be estimated using the following relation:

$$E_s = 600C_u \tag{8}$$

where E_s is the elastic modulus of soil.

3. Case study

3.1. Monitoring scheme in the Shenzhen metro area

During the construction of new tunnels, the deformation of the existing tunnels was monitored. The in-tunnel automatic high-performance remote-reading method was adopted in nearly all tunnel-crossing construction cases in Shenzhen. The layout of the monitoring points and the measured frequency were determined for specific circumstances. Generally, the monitoring width covered the range of the estimated deformation trough of the existing tunnels, and the monitoring data were recorded every 15–30 min during the undercrossing period. The general measuring point (MP) layout is presented in Fig. 1; five monitoring points were installed at the roof, sidewall and track of the existing tunnel. As shown in Fig. 2, the Leica TCA2003 total station measuring system was utilized, which consists of the total station, a wireless transmission module, built-in GeoMos software, reflecting prisms and an antenna.

Among the undercrossing projects, the greenfield settlement and the existing tunnel lining stress were selectively monitored. The monitoring section of the greenfield settlement was located at the same depth as the bottom of the existing tunnel, usually 2–5 m from the point of intersection. As shown in Fig. 3, single-point extensometer monitoring, which can indicate displacement using frequency variations, was used to measure the settlement of the surrounding soil near the existing tunnel. Generally, the distance between the monitoring points was 4–6 m, and at least one monitoring section was connected to the existing tunnels. In this way, the settlement of the nearby soil at the same depth as the bottom of the existing tunnel could be obtained, and the interaction between the soil and existing tunnels could be learned. The