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Volume loss in shallow tunnelling

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ABSTRACT

Although volume loss has an important effect in estimating the ground movements due to tunnelling in the design stage, this parameter is often determined by experience. This makes it difficult to estimate the impact on volume loss when changing project parameters like soil conditions, depth of the tunnel or sensitivity of the surroundings. This paper investigates the relationship between volume loss and cover-to-diameter C/D ratio in shallow tunnelling. Based on a number of (empirical) relations from literature, such as the stability number method and an analysis of the bentonite and grout flows, volume loss at the face, along the shield and at the tail is determined. Long-term volume loss behind the shield is also estimated by means of consolidation. In this way a band width of achievable volume loss for future projects is derived.

1. Introduction

Tunnelling often leads to settlements of the soil surface due to over-excavation, soil relaxation and inefficient tail void filling. The magnitude of volume loss is influenced by tunnelling management, characteristics of the tunnel boring machines (TBM), and the geotechnical conditions. In predictions of surface settlement (Peck, 1969) and subsurface settlement (Mair et al., 1993), the volume loss is often determined by engineering experience and data from previous cases. This makes it difficult to correctly assess the volume loss for a future project under radically different conditions like a shallow depth of the tunnel and/or very different soil parameters. A ground movement analysis in Vu et al. (2015a) shows the important role of volume loss for settlement calculations and in predicting the effects on existing buildings induced by tunnelling. Especially for (very) shallow tunnels near building foundations, the impact of changes in volume loss is large. Most previous studies on volume loss start from a given volume loss and establish deformation patterns from that or correlate surface observations to volume loss at the tunnel for specific projects. Mair et al. (1982), Attewell et al. (1986), Macklin (1999) and Dimmock and Mair (2007) studied the volume loss with a summary of projects in overconsolidated clay relating to the volume loss at the tunnelling face. Verruijt and Booker (1996), Verruijt (1997), and Strack (2002) applied analytical methods for predicting the ground loss around the tunnel. Loganathan (2011) proposed volume loss calculations but only approximated volume loss along the shield with the worst case, and does not take the consolidation into account. Meanwhile, Bezuijen and Talmon (2008) showed the effect of grouting pressure on the volume loss around the TBM but none of these includes a detailed method to estimate volume loss along the TBM. This paper aims to estimate the volume loss when tunnelling with limited C/D ratios (i.e. less than 1) in various soils with a focus on slurry shield tunnelling.

On the basis of the studies by Attewell and Farmer (1974), Cording and Hansmire (1975) and Mair and Taylor (1999), the volume loss in the tunnelling progress can be estimated by the sum of the following components as shown in Fig. 1:

- Volume loss at the tunnelling face: soil movement towards the excavation chamber as a result of movement and relaxation ahead of the face, depending on the applied support pressures at the tunnelling face;
- Volume loss along the shield: the radial ground loss around the tunnel shield due to the moving soil into the gap between the shield and surrounding soil, which can be caused by overcutting and shield shape. The bentonite used in the tunnelling face flows into the gap, while the grout used in the shield tail also flows in the opposite direction. Due to the drop of bentonite and grout flow pressures in a constrained gap, soil can still move into the cavity when the soil pressure is larger than the bentonite pressure or grout pressure;

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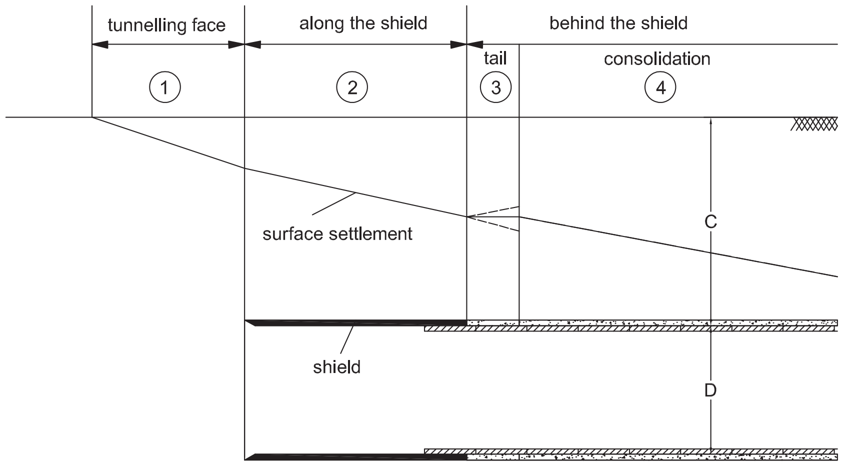

Fig. 1. Volume loss components.

– Volume loss at the tail: when precast segments are placed, the advance of the shield results an annular cavity between the segments and surrounding soil. Grout is used in order to prevent surrounding soil moving into the gap. Volume loss at the tail depends on applied grouting pressure at the tail and proper volume control, where high grout volume and pressure may lead to local heave and low volume to increase settlements as indicated in Fig. 1;
– Volume loss behind the shield tail due to consolidation: in this void along the tunnel lining, grout consolidates and forms a grout cake, and the stress changes induced in the soil may lead to long-term consolidation settlements in soil volume above the tunnel. Other causes of volume loss are shrinkage of grout and long-term lining deformations. However, their contributions to the total volume loss are small comparing to the above factors.

The total volume loss V_L in tunnelling progress can be given as:

$$V_L = V_{L,f} + V_{L,s} + V_{L,t} + V_{L,c} \tag{1}$$

where $V_{L,f}$ is volume loss at the tunnelling face, $V_{L,s}$ is volume loss along the shield, $V_{L,t}$ is volume loss at the tail, and $V_{L,c}$ is volume loss due to consolidation.

To illustrate the impact of the different contributions in different soil conditions, estimates are made for a number of ideal soil profiles which are derived from Amsterdam North-South metro line project (Gemeente-Amsterdam, 2009), consisting of a single soil type with most important properties as defined in Table 1, where γ is volumetric weight, φ is the friction angle, K is the initial coefficient of lateral earth pressure, c is cohesion, C_s is compression constant, C_{swel} is swelling constant, ν is Poisson's ratio and E_s is the stiffness modulus of the ground.

Table 1
Soil parameters used in design of Amsterdam North-South metro line project (Bosch and Broere, 2009; Gemeente-Amsterdam, 2009).

Soil type	γ (kN/m^3)	φ (°)	K (–)	c (kN/m^2)	C_s (–)	C_{swel} (–)	ν (–)	E_s (kN/m^2)
Sand	20	35	0.5	–	–	–	0.2	20,000
Clayey sand	17.9	35	0.4	2	–	–	0.2	12,000
Clay	16.5	33	0.5	7	100	1000	0.15	10,000
Organic clay	15.5	20	0.65	5	80	800	0.15	5000
Peat	10.5	20	0.65	5	25	250	0.15	2000

2. Volume loss at the tunnelling face

When tunnelling, the soil ahead of the excavation chamber generally has the trend to move into the cavity which is created by the tunnelling machine. The soil volume moving towards the face depends on applied support pressures and can be controlled by adjusting the support pressures. In stability analysis for tunnelling, the stability number N proposed by Broms and Bennermark (1967) is widely used. By studying the relationship between this stability number and volume loss at tunnelling face, Attewell et al. (1986), Mair et al. (1982), Mair (1989), Macklin (1999) and Dimmock and Mair (2007) presented a method to determine the expected volume loss based on observed data.

The stability number N is given by:

$$N = \frac{\gamma(C + D/2) - s}{c_u} \tag{2}$$

where s is the support pressure at the tunnelling face and c_u is undrained shear strength of the soil.

In shallow tunnelling, the support pressure at the tunnelling face should be high enough to avoid the collapse to the excavation chamber but also limited to prevent blow-out and fracturing. Firstly, the required support pressure must be higher than or at least equal to the total of water pressure and horizontal effective soil pressure taking into account three dimensional arching effects. The wedge model, which was studied by Anagnostou and Kovári (1994), Jancsecz and Steiner (1994) and Broere (2001), is commonly applied to determine the minimum support pressure $s_{\min}$. In the case of shallow tunnelling, the minimum support pressure $s_{\min}$ can be derived from the wedge model, as follows:

$$s_{\min} = \sigma'_h + p = \sigma'_v K_{A3} + p = \gamma' z K_{A3} + p \tag{3}$$

where p is pore pressure and K_{A3} is the three dimensional earth pressure coefficient determined in Jancsecz and Steiner (1994).

Secondly, the maximum support pressures are often estimated as to avoid blow-out and fracturing. According to Vu et al. (2015b), the maximum support pressures in the case of blow-out are given by:

$$s_{0,t,\max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + \frac{1}{2}HK_y\gamma' \tan\varphi\right) - \frac{aD}{4} \tag{4}$$

$$s_{0,b,\max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + \frac{1}{2}HK_y\gamma' \tan\varphi\right) + \gamma_T \pi d + \frac{aD}{4} \tag{5}$$