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The impact of shallow cover on stability when tunnelling in soft soils

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ABSTRACT

Reducing the cover of shallow (metro) tunnels can lower construction costs by lowering cost of the station boxes, increase safety and lower operational cost in the long-term. For bored tunnels there are normally minimal depth requirements stemming from design and construction. The aim of this paper is to investigate the effects of the cover-to-diameter ratio C/D to the stability of tunnelling process. Several models to analyse the tunnel stability were investigated and were applied for a case study in a typical Dutch soil profile with soft Holocene soil layers. The range of the support pressures in TBM machines, especially in EPB, when tunnelling in soft soil is derived for varied C/D ratio in different soil conditions. Based on the analysis results, some designing optimizations are proposed for shallow tunnels in soft soil.

1. Introduction

Shield tunnelling is used widely in constructing underground infrastructure in cities due to the ability to limit settlements and damage to existing buildings. However, in an environment with soft overburden and buildings on pile foundations, the tunnel is often designed well below the pile tip level. There are two reasons for doing this: to reduce interaction between tunnelling process and piles, and to avoid having to drive through old abandoned piles that are still present below the streets. This results in deep station boxes. When the tunnels would be located at a more shallow level above the pile tip level, this would largely eliminate the impact on pile bearing capacity as well as reduce the required depth of the station boxes and the construction cost. That tunnel construction with shallow cover is technically feasible is shown for example by construction of the Oi Area Tunnel, Japan (Miki et al., 2009), the Zimmerberg Base Tunnel, Switzerland (Matter and Portner, 2004), or microtunnelling and pipejacking in soft ground, see Stein (2005). Moreover, other benefits are the low operational cost in the long-term and shorter travelling time from the surface to the platforms. This is possible only if there are no or very limited obstacles in the subsurface of the streets. Numerous authors have looked into stability of tunnel in soft soil such as Broms and Bennermark (1967), Atkinson and Potts (1977), Davis et al. (1980), Kimura and Mair (1981), Leca and Dormieux (1990), Anagnostou and Kovári (1994), Jancsecz and Steiner (1994), Chambon and Cort (1994), Broere (2001) and Mollon et al. (2009). However, they have not explicitly investigated the stability of very shallow tunnelling. This paper looks into several aspects of shallow overburden tunnelling and seeks the limits on the cover-to-diameter ratio C/D when tunnelling in soft Holocene layers. Various geotechnical influences on the tunnel will be studied and the effect of low C/D ratio will be modelled. The analysis is carried out with a number of ideal soil profiles 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 coefficient of lateral earth pressure and c is cohesion.

2. Geotechnical analysis of tunnel stability

2.1. Uplift

In tunnelling design, failure by uplift should be assessed as a permanent stability assessment. Uplift of bored tunnels is indicated in several studies such as Bakker (2000) and NEN-EN 1997-1 (1997). In offshore industry, there are models of uplift stability for oil and gas pipeline are proposed by Trautmann et al. (1985), Ng and Springman (1994), and White et al. (2001) which present various sliding blocks and inclined failure surfaces. In this paper, the model with vertical slip surface (Fig. 1) which has a diameter D soil volume above the circle tunnel is proposed for analysis. Below the ground water level, the tunnel is loaded by the following vertical forces: the weight of overlaying soil layers G_1, the weight

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Table 1
Soil parameters.

Soil type	γ (kN/m^3)	φ (°)	K (–)	c (kN/m^2)
Sand	17.9	35	0.4	2
Clay	16.5	33	0.5	7
Soft clay	15.5	20	0.65	5
Peat	10.5	20	0.65	5

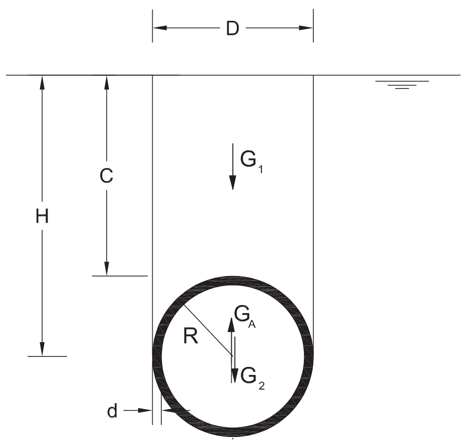

Fig. 1. Uplift calculation.

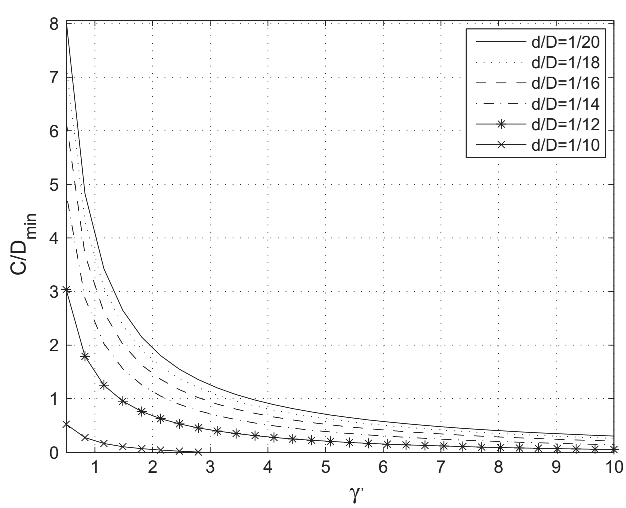

Fig. 2. Relation between unit weight of soil and the minimum required ratio C/D.

of the tunnel G_2 and the uplift force G_A. The uplift force of the tunnel can be estimated according to the Archimedes's principle as:

$$G_A = \gamma_w \frac{\pi}{4} D^2 \tag{1}$$

where γ_w is the volumetric weight of water and D is the diameter of the tunnel.

The weight of the tunnel lining follows from:

$$G_2 \approx \pi \gamma_T D d \tag{2}$$

where is d is the thickness of the tunnel lining and γ_T is the weight unit of the tunnel lining (concrete).

The weight of the soil layers above the tunnel is given by:

$$G_1 \geqslant DH\gamma'_g - \frac{\pi}{8} D^2 \gamma'_g \tag{3}$$

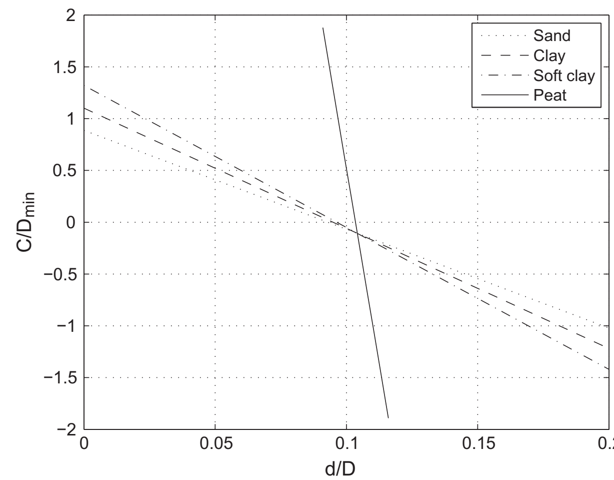

Fig. 3. Relation between ratio of d/D and the minimum required ratio C/D.

Table 2
Minimum required d/D.

Soil type	γ (kN/m^3)	d/D
Sand	17.9	0.093
Clay	16.5	0.095
Soft clay	15.5	0.096
Peat	10.5	0.103

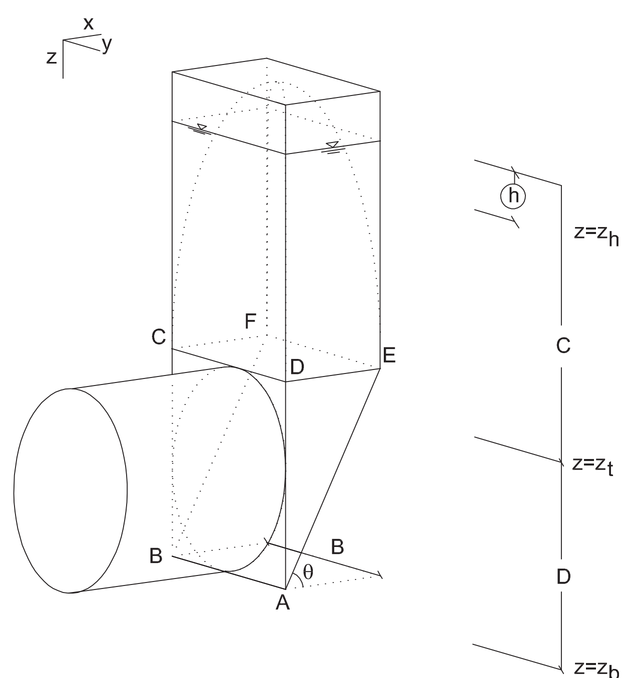

Fig. 4. Wedge loaded by soil silo (Broere, 2001).

where γ'_g is the effective volumetric weight of soil.

In the construction phase, it is assumed that friction between the lining and surrounding ground is not included in the vertical equilibrium (lower boundaries). If the uplift force G_A is smaller than the total of tunnel weight and the upper soil layers weight, there will be no uplift of the tunnel (although safety factors have not been included here):