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Structural design model for tunnels in soft soils: From construction stages to the long-term

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ABSTRACT

In bored tunnel design, most recent structural design models for tunnel linings concentrate on the behaviour of the tunnel lining in the long-term. The load on the tunnel lining in these models is derived from the original soil stresses, often simplified for a single homogeneous layer. Field observations show that higher loads may occur in the initial hours after the assembly, that might effect the tunnel lining and that soil layers with different stiffnesses may have a negative impact on the internal forces of the tunnel lining. This paper proposes a new model for these early construction stages and also includes a more accurate model which explicitly models the impact of multilayered soils. The change of internal forces in the tunnel lining from the initial construction time to the long-term is investigated with this model. Validations with field observations and other analysis results at time of construction and the long-term confirm that the new structural analysis models can accurately predict internal forces in the tunnel lining. The analysis results also show that internal forces in the tunnel lining have an increasing trend in time and become stable in the long-term and accord with field observations.

1. Introduction

The increased use of Tunnel Boring Machines (TBMs) in constructing (urban) underground space (Broere, 2016) combined with the fact that most of the world urban population resides in coastal and delta areas, with often soft soil conditions, means that increasingly tunnels are bored in soft layered soils and with decreasing cover. Besides assessment of face stability, surface settlement and resulting damage to buildings (Vu, 2016), structural analysis of the tunnel lining remains an important issue in tunnelling design. However, the common simplifications of homogeneous soil conditions and homogeneous stress conditions in most structural design methods are less applicable for tunnels with limited cover in soft soils. There is a number of structural design models for the tunnel lining commonly used encompassing both analytical models and numerical models. The first analysis method for an elastic continuum was proposed by Schmid (1926). Morgan (1961) introduced an analytical continuum model, which considers the elliptical deformation of the tunnel lining. Then, Schulze and Duddeck (1964) produced a bedded ring model for analysing the case of shallow tunnels. Windels (1966) further developed the model proposed by Schulze and Duddeck (1964) by taking into account the second order of the series expansion of the analytical solution and the deformation of the tunnel lining in the construction stage. A design model for a circular tunnel in an elastic continuum with geometrical nonlinearity was presented by Windels (1967). The model proposed by Morgan (1961) was corrected by Muir Wood (1975) by taking into account the tangential stresses; however, the radial deformations of the tunnel lining due to these stresses were neglected. In 1976, Muir Wood (1976) solved this problem. The basis for the common tunnel design models used in practice and guidelines (ITA-WG2, 2000) were introduced by Duddeck and Erdmann (1985), including a bedded-beam model without a reduction of ground pressure at the crown and a continuum model. In the bedded-beam model, the interaction between tunnel lining and the surrounding soil is presented by bedding springs. In the continuum model, this interaction is included automatically. Blom (2002) extended a beam model to take into account the effects of longitudinal joints and soil reactions to estimate the deformation of the tunnel lining. Oreste (2007) applied a hyperstatic reaction method to derive the internal forces in the tunnel lining with a finite element method (FEM) framework for the case of tunnelling in rock. Even though the interaction between tunnel lining and surrounding medium through Winkler springs is simulated in this model, only radial pressures are considered. A further model, which includes the tangential pressures, was developed by Do et al. (2014). Recently, an adaptation of Do et al. (2014) model has been proposed by Vu et al. (2017) for shallow tunnels in soft soils. The comparison of the analytical results derived from this

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model and Duddeck and Erdmann (1985) in Vu et al. (2017) with various depths of the tunnel shows that the new model is not only applicable in the case of shallow tunnelling and aligns closer to field observations, but also can be applied in the cases of tunnelling with moderate and deep depth.

Although many models have been proposed for tunnel design since 1926, most of these models focus on the long-term behaviours and include assumptions of the actual loading on the tunnel lining and interactions between the lining and surrounding soils that are valid for long-term loading conditions (Duddeck and Erdmann, 1985; Vu et al., 2017). In the long-term, when the grout in the tail void hardened, the tunnel lining is often considered supported directly by the soil with stresses in the soil dependent on the stress state prior to tunnelling. In practice, at the start of segment assembly, the lining is surrounded by injected grout just behind the TBM. Field data show that high pressures on the tunnel lining and large strain development occur in initial hours after assembly of tunnel segments (van Oosterhout, 2003; Bezuijen and Talmon, 2004; Talmon and Bezuijen, 2009). The other problem is that even most recent models only investigate the behaviour of the tunnel lining in a homogeneous soil and with load conditions relevant for the long-term stage. An effort to analyze the case of a tunnel in a multi-layered soil was carried by Bakker (2000). However, this analysis was carried out with an approximate method by modifying the multilayered soil parameters to an approximate homogeneous soil. This might lead to inaccurate predictions of deformation and internal forces when the tunnel is in different soil layers. A numerical simulation using a 3D FE model, where the advance process of tunnelling in a two layered soil condition including the TBM advancement steps, ring-wise assembly, grout hardening process and also consolidation is modelled, will yield more detailed and more accurate results. For example, Ninić and Meschke (2017) show that such an approach is possible, but also that for engineering practice it is still less applicable due to high computational load and the required large number of input parameters. As such a simpler framework is still preferred for design purposes.

In order to prevent any damage on the tunnel lining, a careful assessment of the tunnel lining deformations and loads is needed from the time of construction to the long-term. This paper looks into a method to calculate internal forces in the long-term for a situation with multi-layered soil conditions and in the tunnel lining in various construction stages as well as investigates the change of these internal forces in time.

2. Structural analysis for tunnel linings in the long-term

Recent models in tunnelling design, e.g Duddeck and Erdmann (1985), ITA-WG2 (2000), Do et al. (2014) and Vu et al. (2017), often assume a tunnel in homogeneous soil conditions. Especially, in soft soils with variable soil stiffnesses, this obviously leads to imprecise predictions for internal forces in the tunnel lining due to the inaccurate values of the interactions between the tunnel lining and the surrounding soils and the soil loading at particular points of the tunnel lining. To that end, we extend the structural model proposed in Vu et al. (2017) (Figs. 1 and 2) to the case of a multilayered soil as can be seen in Fig. 3. The lining is represented by a frame work based on the finite element (FE) model described by Do et al. (2014) and Vu et al. (2017) which is used to derive internal forces in the tunnel lining (Fig. 2).

In this model, the load at each node on the tunnel lining frame depends on the depth of the calculated ith node and which soil layer it is located. In detail, the depth of the ith node z_i is given by:

$$z_i = (H + R\cos\theta_i) \tag{1}$$

The vertical soil pressures at the ith node on the tunnel lining in the jth layer can be estimated as:

$$\sigma_{v,i} = \sum_{m=1}^{j-1} \gamma_m H_m + (z_i - H_j)\gamma_j \tag{2}$$

where H_m and γ_m are the depth and the weight unit of the mth layer (see Fig. 3).

The horizontal soil pressure at the ith node on the tunnel lining $\sigma_{h,i}$ is given by:

$$\sigma_{h,i} = K_j \sigma_{v,i} \tag{3}$$

where K_j is the coefficient of horizontal effective stress at rest of the jth layer. Adapting to the method indicated in Vu et al. (2017), the initial radial ground reaction stiffness of the jth layer $\eta_{r,j,0}$ is estimated as:

$$\eta_{r,j,0} = \beta \frac{1}{1+\nu_j}\frac{E_j}{R} \tag{4}$$

where E_j and ν_j is Young's modulus and Poisson's ratio of the jth layer and in accordance with Do et al. (2014) $\beta = 2$ is used here.

The relationship between tangential spring stiffness η_s and normal spring stiffness η_n is (Vu et al., 2017):

$$\eta_s = \frac{1}{3}\eta_n \tag{5}$$

The maximum radial reaction pressure $p_{n,j,lim}$ of the jth layer can be calculated as:

$$p_{n,j,lim} = \frac{2c_j\cos\varphi_j}{1-\sin\varphi_j} + \frac{1+\sin\varphi_j}{1-\sin\varphi_j}\Delta\sigma_{j,conf} \tag{6}$$

where c_j,φ_j are cohesion, the friction angle of the jth layer.

The confining pressure on the tunnel perimeter $\Delta\sigma_{j,conf}$ is estimated as:

$$\Delta\sigma_{j,conf} = \frac{\sigma_{h,i} + \sigma_{v,i}}{2}\frac{\nu_j}{1-\nu_j} \tag{7}$$

and the stiffness of the radial springs $k_{n,i}$ and tangential springs $k_{s,i}$ of the ith node of the frame is:

$$k_{n,i} = \eta^*_{n,j,i}\left[\frac{L_{i-1}+L_i}{2}\right] = \frac{p_{n,j,lim}}{\delta_{n,j,i}}\left(1-\frac{p_{n,j,lim}}{p_{n,j,lim}+\eta_{n,0}\delta_{n,j,i}}\right)\frac{L_{i-1}+L_i}{2} \tag{8}$$

$$k_{s,i} = \eta^*_{s,j,i}\left[\frac{L_{i-1}+L_i}{2}\right] = \frac{p_{s,j,lim}}{\delta_{s,j,i}}\left(1-\frac{p_{s,j,lim}}{p_{s,j,lim}+\eta_{s,0}\delta_{s,j,i}}\right)\frac{L_{i-1}+L_i}{2} \tag{9}$$

where $\delta_{n,j,i}$ and $\delta_{s,j,i}$ are the radial and tangential deformations of the ith node in the jth layer.

Similar to Do et al. (2014) and Vu et al. (2017), in this multilayered soil model, the analysis frame used consists of 360 elements representing a 1° segment. The condition that the radial springs are only active in the compression condition is still applied.

3. Structural design model for tunnel linings in construction stages

During TBM tunnelling, when precast segments are placed, the advance of the shield creates an annular cavity between the segments and the surrounding soil. This is due to the TBM's shape and the overcut. In order to minimize the movement of surrounding soil into the gap, grout is injected rapidly at the tail of the TBM. The injected grout induces pressures on the tunnel lining and the soil around. This grout pressure changes in different construction stages as shown in field data in, for example, Groene Hart Tunnel, Sophia Rail Tunnel and Botlek Railway Tunnel, in the Netherlands (van Oosterhout, 2003; Bezuijen and Talmon, 2004; Talmon and Bezuijen, 2009). Field data show that the peak value of grout pressures and the development of strains often occur in initial hours after the assembly of segments. This might lead to potential high internal forces in the tunnel lining and result in damages of the tunnel lining. A structural assessment for the tunnel lining in construction stages, therefore, can not be neglected. This part of the paper introduces a structural design model for construction stages of the tunnel lining.