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Analytical methods in predicting excess pore water pressure in front of slurry shield in saturated sandy ground

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ABSTRACT

During mechanized tunnelling with a slurry shield in saturated sand, excess pore water pressures will generate in front of slurry shield. These excess pore water pressures can influence the stability of the tunnel face. The magnitude of the excess pore water pressures thus becomes of importance for the stability of the tunnel face. In this paper, two theories solutions are presented to describe the development and decline of the excess pore water pressures in front of slurry shield in saturated sandy ground. The first theory considers transient flow in a semi-confined aquifer with elastic storage, while the second one assumes different conditions of unconfined steady-state flow governed by the penetration of slurry into the soil in front of the tunnel face. Both methods are tested at different positions around the tunnel, during both drilling and standstill and compared with measurements performed at the Green Hart Tunnel and North/South line in the Netherlands. It is shown that both analytical theories can predict the excess pore water pressures in front of slurry shield. The second one seems more appropriate because it reflects the effect of slurry penetration. Furthermore, the measurements seem to indicate that the influence of elastic storage is not so big as assumed in the first theory.

1. Introduction

Mechanized tunnelling with a slurry shield has been widely used for the tunnel projects in saturated sandy ground. A significant number of studies (e.g. Anagnostou and Kovári, 1994) have been conducted to investigate the process at the slurry supported tunnel face during slurry shield tunnelling. Anagnostou and Kovári (1994) first developed a comprehensive understanding of the mechanism of face failure during slurry shield tunnelling. In their study, various parameters, including ground properties, slurry suspension parameters, slurry pressure, tunnel geometry, etc., are considered. The effect of slurry penetration into the ground in front of the tunnel was also discussed. It was found that the effectiveness of slurry support essentially depends on the slurry penetration distance into the ground. A larger penetration distance leads to a lower effective support pressure, and hence a more unstable condition of the tunnel face. In addition, they argued that because infiltration takes place gradually over time, the safety factor is time-dependent. Larger penetration distance will decrease the effective support force, as well as stability of tunnel face (as expressed by the safety factor). The experience on shield tunnelling, however, started in the Netherlands only two decades ago (Bezuijen and Bakker, 2008). Because of the limited knowledge of shield tunnelling in permeable saturated sandy ground, a series of in-situ measurements and studies have been carried out during the construction of early shield tunnels in Netherlands. One of the findings of that research was that there is an excess pore pressure present in the soil in front of the TBM during drilling, which influences the stability of the tunnel face (Bezuijen et al., 2006; Broere, 2003; Broere and van Tol, 2000, 2001). This excess pore pressure disappears during standstill of the TBM, see Fig. 1. In Netherlands, excess pore pressures were measured during the tunnel driving at various projects, such as the Second Heinenoord Tunnel, the Botlek Tunnel, Green Hart Tunnel (GHT) and North/South Metro Line (N/S Line) (Bezuijen et al., 2006, 2016; Broere, 2003; Kaalberg et al., 2014).

To predict the excess pore water pressure in front of a TBM, some analytical methods have been developed (Bezuijen et al., 2006, 2016; Broere and van Tol, 2001). Broere and van Tol (2001) proposed a model to predict these excess pore water pressures assuming that drilling takes place in a semi-confined aquifer and that equilibrium is not achieved immediately due to elastic storage, so that transient conditions must be considered. The excavation is modelled thorough its discharge, which is estimated considering that the amount of water displaced by the penetrating slurry is roughly equal to the pore volume in the excavated soil. Bezuijen et al. (2006, 2016) developed a model where the pore water pressure variations are governed by the properties of the slurry

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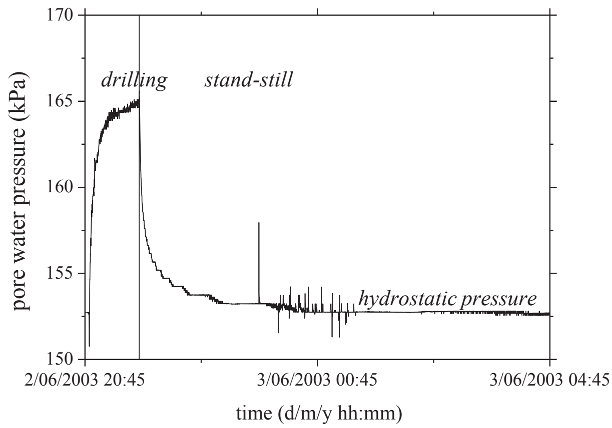

Fig. 1. Pore water pressure during the process of tunnelling at Green Hart Tunnel: drilling and standstill.

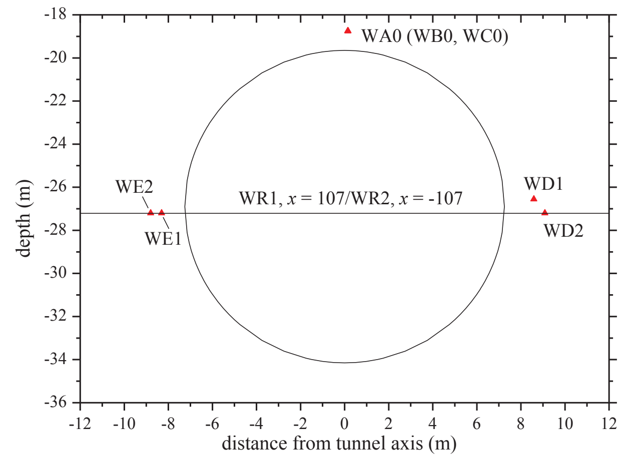

Fig. 2. Sketch of pore pressure transducers at ring 2117 at GHT.

that penetrate into the soil during drilling and the excavation is set in a homogenous unconfined thick soil layer where steady-state conditions can be considered. The dissipation of the hydraulic head through the penetrated zone defines the piezometric head in front of the tunnel, which sets the new pore water pressure distribution based on an assumption of radial flow. Unlike the model of Broere, Bezuijen assumed that the influence of elastic storage is negligible, thus there must be no phase shift between pore pressure responses taken at different positions. As explained by Talmon et al. (2013), the penetration of slurry can be distinguished into two processes: mud spurt and filter cake formation. When mud spurt starts, the slurry (water with bentonite particles) penetrates into the soil. After some time, the bentonite particles are blocked in the soil pores and only water from the slurry flow into the soil, the filter cake thus forms at the surface of soil. Because the cutter wheel will constantly remove the filter cake, however, there will be no formation of filter cake at the tunnel face during drilling of TBM. Hence, the effect of filter cake is not taken into account. The piezometric head drops because of penetration during standstill of TBM, and rises due to removal of the penetrated soil by cutter wheel during drilling.

In this paper, the two analytical models are introduced and the differences between them is interpreted. The calculated results from these two models are compared with the pore pressures measured around the construction of the GHT and the N/S Line in the Netherlands.

2. Field measurements at GHT and N/S Line

The GHT is located approximately 20 km southwest of Amsterdam. The tunnel is constructed through a saturated sand layer, where the overburden consists of peat and clay. Hydro-geotechnical conditions are described by Aime et al. (2004) and tunnel design by Aristaghes et al. (2002). Eight pore pressure transducers (PPTs) were installed in the soil around the tunnel in advance of the TBM passing. The location of PPTs around the tunnel cross-section can be seen in Fig. 2. The two instruments further away from the TBM are just marked at their depth (WR1, WR2). All instruments were placed in the ground along the tunnel chainage 4219 m, with the exception of WB0 (4221 m) and WC0 (4223 m). This layout allows the pore pressures to be evaluated both in time and space. The records of the shield position are only available after the face of the TBM had passed the instrumentations section. However, other cases (Kaalberg et al., 2014; Aime et al., 2004) reveal that the increments of pore pressure are more or less symmetric regarding the distance between the face and the instruments. Therefore, the distances after the section can be considered equivalent to distances before the section.

The tunnels of N/S Line in Amsterdam is constructed through a saturated sand layer, where the overburden consists of very soft soil. It is a challenging project in an unfavourable urban environment with over 1000 historical buildings founded on wooden piles close to the tunnels (Kaalberg et al., 2014). The diameter of the two bored tunnels is 6.5 m. At a historic bridge "Bridge 404", both tunnels pass the bridge at a different depth (see Fig. 3). The East tunnel is constructed only 1.5 m below the pile tips of the bridge. Unfortunately, details of instruments at N/S Line are unavailable.

3. Transient model

In the Netherlands, the sandy layers used for tunnelling are normally overlain with soft soil layers of peat and clay with a low permeability. In such a situation, the pressure distribution in the soil can be evaluated as a semi-confined aquifer. A time-dependent method for build-up and dissipation of excess pore pressures in a semi-confined aquifer was proposed by Broere and van Tol (2001). While the TBM is drilling, the excess pore pressure is given by:

$$\phi-\phi_\infty = \frac{Q\lambda}{4k_sH}\left[erfc\left(\frac{xu}{2\sqrt{t}}+\frac{\sqrt{t}}{u\lambda}\right)\exp\left(\frac{x}{\lambda}\right)-erfc\left(\frac{xu}{2\sqrt{t}}-\frac{\sqrt{t}}{u\lambda}\right)\exp\left(-\frac{x}{\lambda}\right)\right] \tag{1}$$

where ϕ (m) is the piezometric head at a distance x (m) in front of the tunnel face; ϕ_∞ (m) the piezometric head far from the tunnel in the soil; Q (m^3/s) the discharge at $x = 0$; the leakage length $\lambda = \sqrt{k_sH\dot{c}}$ (m), with $\dot{c}$ (s) the hydraulic resistance of the confining layer; k_s (m/s) the permeability of aquifer; H (m) the height of aquifer; $u = \sqrt{S_s/k_s}$, with S_s (m^{-1}) the coefficient of specific storage; t (s) the time.

According to Mohkam and Bouyat (1985), volume of the water discharged by the penetrating slurry is roughly equal to total pore volume of the excavated material. Hence, the discharge per unit area of tunnel could be estimated by:

$$q = -nv_{TBM} \tag{2}$$

with n (–) the porosity of the excavated soil and v_{TBM} (m/s) the advance rate of the TBM.

The value of Q then can be determined by Eq. (2). This is in fact a one dimensional transient model assuming flow from the tunnel front into a semi confined aquifer. Because the model has not taken into account the influence of slurry penetration, it is only valid when no plastering occurs. The pore pressure build up at the beginning of drilling is according to this model governed by elastic storage, as well as the pressure decay. Eq. (1) is only valid when the drilling velocity is smaller than the penetration velocity of the slurry. When the drilling velocity is larger than the penetration velocity the piezometric head will be overestimated by Eq. (1). In this case, therefore, Eq. (1) needs to be rewritten as: