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The influence of flow around a TBM machine

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ABSTRACT: The flow of grout and bentonite around a slurry shield TBM is investigated, using a 1-dimensional calculation model. The results show that the shield of the TBM is only partly in contact with the surrounding soil and that for a large extent it is in contact with the liquid grout and, depending on the amount of over cutting, also in contact with the bentonite that is injected at the tunnel face. These liquids also determine the forces on the TBM shield. The calculation model presents the pressure distribution on the shield and shows the influence of the soil properties, the overcutting and the properties of the grout and the bentonite. Assumptions in the model are checked with 2-D finite element calculations, plane strain and axi-symmetric, that show qualitative agreement, and a reasonable quantitative agreement. The model is useful to explain the phenomena and as a first estimation of the influence of flow around the TBM.

1 INTRODUCTION

Tunnelling through urban areas asks for a minimal ground loss. Modern TBM's can operate at an average ground loss of 0.5% or less (Bowers & Mos, 2005). To be able to calculate the ground loss that can be expected in this range it is necessary to improve the calculation methods. One decade ago it was sufficient to calculate whether the front pressure was sufficient to prevent a collapse of the bore front and not high enough to get a blow-out. Nowadays 4-D finite element modelling is used to estimate the ground loss and the settlement trough that is the result of it.

In most simulations on TBM tunnelling it is assumed that the non-excavated soil around the TBM slides over the shield skin of the TBM (Hoefsloot & Verweij, 2005; Kasper & Meschke, 2006). As a consequence the tapering of the TBM results directly into a volume loss and a settlement trough.

However, in reality the calculated settlement trough is too high, there may be a space between the shield skin of the TBM and the soil. This is because the cutting wheel of a TBM is in most cases a bit larger than the TBM itself. There is only a small difference (a few centimetres on a TBM with a diameter of 10 metres or more), but this is significant. It means that for a slurry shield bentonite can flow from the tunnel face back to the grout used to fill up the tail void. However, it is also possible that grout can flow from the tail void to the tunnel face.

A 1-dimensional model is developed to get information on the order of magnitudes of these flows, the pressure distribution and the soil deformation. The model assumes a given pressure distribution at the tunnel face (the face pressure) and at the tail of the TBM (the grout pressure) and incorporates the yield strength of both the grout and the bentonite. It takes into account the overcutting of the cutting wheel and the conical construction of the TBM (with a bit smaller diameter at the tail compared to the front). Linear elastic soil behaviour is assumed.

The paper will briefly describe the model, more detailed information is presented by Bezuijen (2007), some example calculations will be presented and the assumptions will be checked by using 2-D finite element calculations.

2 GEOMETRY OF A TBM

A TBM shield seems to be a tube with a constant diameter, sketched in a way as in Figure 1. However, looking more in detail the shield is part of a cone. The diameter at the front is larger than at the tail. The difference is only small, normally around 0.4% of the diameter of the TBM. For a TBM shield with a diameter of 10 m, this means that the diameter of the shield is 4 cm smaller at the tail compared to the diameter just after the cutter head. This small difference in diameter allows the TBM to manoeuvre in the soil.

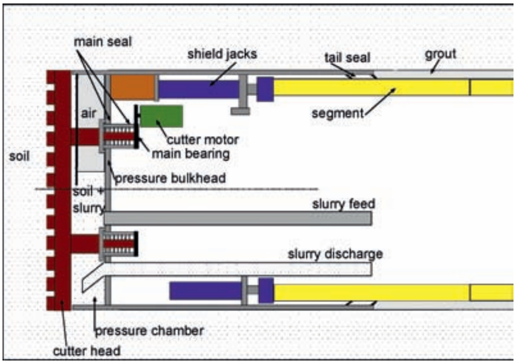

Figure 1. Sketch of a slurry shield TBM.

3 GROUT FLOW AROUND A TBM

3.1 *Theory*

Assume a TBM with a small change in diameter (a diameter decrease from front to back). The TBM is boring in soil that is assumed linear elastic with a shear modulus *G*.

Assume that the soil in contact with the shield at every location of the shield. The tapering of the shield will lead to a decrease in stresses in the soil around the TBM going from the tunnel face to the tail. A simple approach is to neglect the influence of gravity and assume a tunnel that is positioned perfectly symmetric in the bore hole. In such a situation the relation between deformation and stress reduction can be written as (Verruijt, 1993):

$$\Delta\sigma = 2\frac{\Delta r}{r}G \tag{1}$$

Where $\Delta\sigma$ is the change in pressure, Δr the change in radius, r the radius of the tunnel and the grout and G the shear modulus of the soil around the tunnel.

Calculating the pressure from front to tail of the TBM, without the influence of grout or bentonite, will lead to an ongoing pressure reduction.

However, bentonite is injected at the front of the TBM and grout in the tail. Normally the bentonite pressure is lower than the initial soil pressure and the grout pressure is higher. Using eq. (1) this means that there will be a diameter decrease at the front and a diameter increase at the tail. However, when overcutters are used it is still possible that there is an opening at the front of the TBM between the tunnel face and the soil where bentonite can flow over the shield to the tail. Due to the high grout pressure it is also likely that grout flows from the tail over the shield to the front of the TBM.

Somewhere in the middle around the TBM these flows will meet. Calculating the pressure distribution is now complicated by the fact that the direction of the

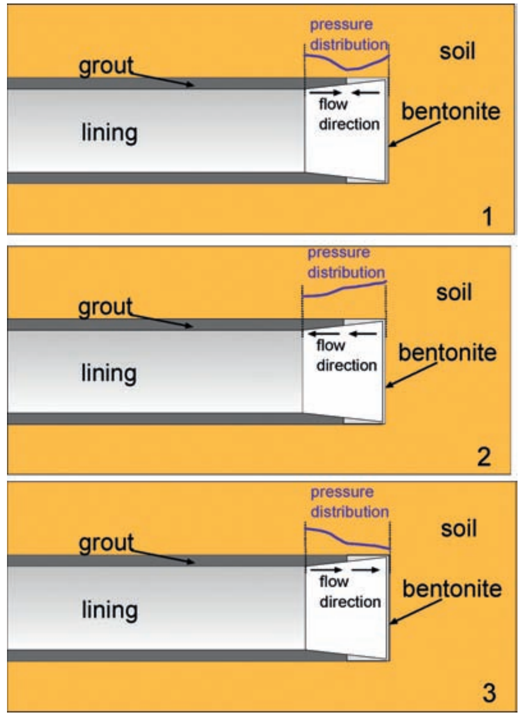

Figure 2. Possible flow directions and sketched pressure distributions along the TBM. The 3rd situation is worked out quantitatively, see Figure 3.

flow is of importance. Both grout and bentonite are usually described as Bingham liquids. This means that a certain pressure gradient is needed to start a flow. Therefore there are 3 possible flow situations, see also Figure 2.

1 Grout flows from the tail to the tunnel face and bentonite flows from the tunnel face to the tail (this situation can only occur when there is some volume loss in the gap between the TBM and the soil for example due to bleeding of the grout or penetration of the bentonite into the soil. For this situation the lowest pressure will be present there where the bentonite and the grout meet.
2 Bentonite is flowing backwards to the tail and pushing the grout out of the gap between the TBM and the soil. The pressure will be the highest at the tunnel face and will decrease when going to the tail. This cannot be a continuous situation, but can occur temporally.
3 Grout is flowing to the tunnel face and pushes the bentonite to the tunnel face. The pressure is highest at the tail close to the injection points of the grout and will decrease going to tunnel face.

The model developed can in principle cope with all these 3 options. However, for simplicity in this paper we will only deal with the 3rd option because this is the most realistic situation for most of the drilling process. As the machine moves forward, a forward flow of material is needed to fill the annulus that is created by the TBM. It is therefore assumed that the grout flows along the TBM shield and pushes the bentonite away. Considering the advancing TBM during drilling, this can be a stable situation. Both the bentonite and the grout can be described as Bingham liquids. For the low flow velocities during tunnelling the yield stress of the liquid will be determining the pressure drop and the viscous forces can be neglected. Assuming that the flow induced friction will develop between the soil and the grout (in the 3rd option the grout and bentonite will not move with respect to the TBM in a stable situation), the pressure drop can be written as:

$$\Delta P = \frac{\Delta x}{s}\tau_y \qquad (2)$$

Where ΔP is the change in pressure due to the flow, Δx a length increment along the TBM, s the gap width between the tunnel and the soil and τ_y the shear stress of the grout around the TBM.

The following calculation procedure is used. The soil around the tunnel is assumed to behave as independent slices with a thickness Δx. Knowing the geometry of the tunnel, the grout pressure at the tail and the bentonite pressure at the tunnel face, the soil pressure and the elastic properties of the soil, the gap width at the face and tail of the TBM can be calculated using Equation (1). That gap width can be used to calculate the pressure drop due to the flow of the grout to the front of the TBM over the distance Δx with Equation (2) and the pressure increase that will occur in the bentonite when it is pushed to the front. The resulting grout pressure and bentonite pressure is calculated independently. As long as at a certain location the calculated grout pressure is higher than the calculated bentonite pressure, it is assumed that the grout pushes the bentonite in the direction of the tunnel face and there will be grout at that location. In the case that the bentonite pressure is higher, there will be bentonite.

The result of such a calculation is shown in Figure 3. The upper plot of this figure shows the pressures and gap widths that would occur if there were only grout (G) or only bentonite (B) in dots and in lines the combined gap width and pressure. In this calculation the grout penetrates between the shield and the soil up to 1.8 m from the tail. The remaining part of the gap between the TBM and the soil is filled with bentonite. The amount of penetration varies with the pressures and shear strength that are chosen. The parameters used in this calculation are shown in Table 1. Although there is an open connection between the face and tail of the TBM (See the lower plot of Figure 3, the gap width of the combined calculation is never zero), there is still a stable boundary between the grout en the bentonite. Since both liquids behave as Bingham liquids, there can be some decrease in the grout pressure and still the position of the grout-bentonite front will be the same.

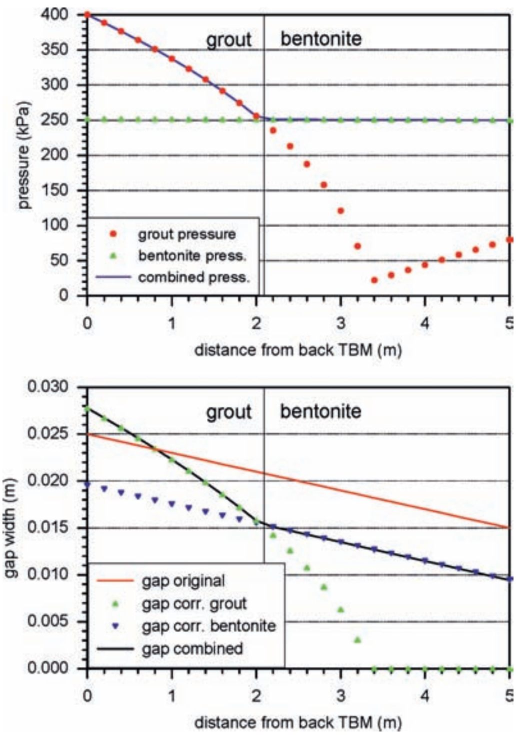

Figure 3. Pressures and gap width along a TBM. Grout pressures and bentonite pressures. Parameters see Table 1. Plots show pressures and gap width for the bentonite and grout pressure separately and the combined result.

Table 1. Input parameters used in calculation with bentonite and over cutting.

length TBM shield	5	m
Diameter	10	m
diameter reduction	0.2	%
over cutting	0.015	m
asymmetric (1) or symmetric (2)	2	
grain stress	150	kPa
grout pressure	400	kPa
pore pressure	200	kPa
pressure on tunnel face	250	kPa
shear modulus (G)	90	MPa
shear strength grout, 1 sided friction assumed	1.6	kPa
shear strength bentonite 2 sided friction assumed	0.08	kPa

According to the results, the tail of the TBM is never in contact with the soil. In reality there can be some contact due to forces not taken into account in this calculation (vertical loading from the lining, moments induced by the hydraulic jacks). However, it is clear that tapering of the shield does not necessarily lead to a certain volume loss, as was assumed by Kaspar & Meske (2006). The calculation results presented here are also in line with the TBM data, which normally show that more grout have to be pumped into the tail void than corresponding to the original tail void. The grout volume that flows into the gap between the soil and the TBM is not an extra volume, because it is a constant volume during tunnelling, but the grout volume increases because the grout pressure makes a wider gap.

In the Western Scheldt Tunnel project a soil sample was taken trough the tail of the shield. In that case no grout was found between the shield and the subsoil, only some bentonite in the sand (Thewes, 2007). This indicates that in reality the soil deformation is not as uniform as suggested here. At some locations the tunnel will remain in contact with the TBM. This will also be the result of the numerical calculations presented in the next session.

4 NUMERICAL SIMULATIONS

Some of the assumptions in the calculation model have been tested by means of numerical calculations, using the Plaxis programme.

The assumptions tested are the axial-symmetric deformation of the soil that is assumed and the influence of the non linear behaviour of the sand. Therefore a 2-dimensional plane strain simulation is performed using the mesh shown in Figure 4. The simulation is performed for soil conditions that are common in the western part of the Netherlands: A Holocene top layer or 10 m and a tunnel with a diameter of 10 m embedded in Pleistocene sand. The small strain hardening soil model was used in the simulation (Brinkgeve et al., 2006). The parameters used are presented in Table 2.

In the simulation it is assumed that the pressure on top of the tunnel is 350 kPa at the beginning and the pressure is increased up to 400 kPa to simulate the increase in pressure due to grouting behind the TBM. In a second calculation the stress is first reduced to 320 kPa and increased to 400 kPa after that. This simulates the unloading in the soil that can occur due to the overcutting and/or lower bentonite pressures. The results will show to what extent a linear elastic soil behaviour can be assumed.

The pressure increase with depth in the grout is assumed to be 18 kPa/m. The calculated displacements around the tunnel opening due to the pressure increase are shown in Figure 5 for both loading situations. With

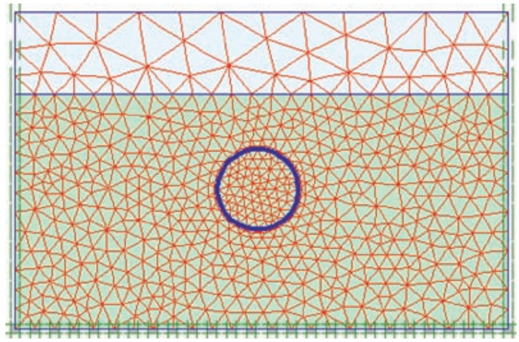

Figure 4. Mesh used in 2-D simulation. The diameter of the tunnel is 10 m.

Table 2. Input parameters for the Finite Element analysis. Parameters give (from top to bottom) the wet volumetric weight, elasticity parameters (Brinkgeve et al., 2006), cohesion, friction and dilatancy angle.

		Holocene	Pleistocene Sand	Grout
γ_{sat}	kN/m^3	18	20	pm
ν_{ur}	–	0.35	0.2	0.499
E_{oed}^{ref}	MPa	10	180	pm
E_{50}^{ref}	MPa	10	180	0.45
E_{ur}^{ref}	Mpa	–	720	–
G_0^{ref}	Mpa	–	675	–
$\gamma_{0.7}$	–	–	1.00E-04	–
c'	kPa	5	1	15
ϕ	°	20	32	0
Ψ	°	–	2	–

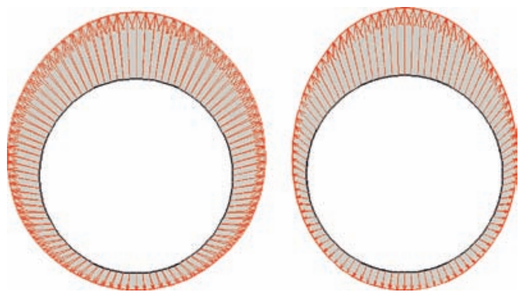

Figure 5. Deformation pattern around tunnel due to a pressure increase of 50 kPa starting at a pressure of 350 kPa (left), resulting in a maximum displacement of 22 mm. Right the same calculations but first a stress reduction to 320 kPa was applied, resulting in a max deformation of 25 mm.

the parameters used in the simulation, the maximum displacements are in the same order as the displacements calculated with the analytic model (27 mm), but this depends on the choice of the parameters.

It is clear that the displacement is not symmetrical around the tunnel axis. The soil will predominantly move upwards. For the situation where there was first unloading the deformation pattern is a bit more 'egg shaped' (see Figure 5).

The deformation pattern is therefore not completely axial-symmetric as is assumed in the analytical calculations. Furthermore, in reality the TBM can rotate with a horizontal rotation axis perpendicular to the axis of the TBM in the just created opening. This can also lead to an uneven distribution of the gap around the TBM. Therefore from this calculation it cannot be concluded what will be the gap width distribution between the TBM and the soil around the tunnel at different locations, what can be concluded is that the gap width shall not be evenly distributed along the TBM circumference. Simulations of the deformation of the soil movements for a tunnel at a larger depth resulted in a more symmetric deformation pattern.

5 DISCUSSION

The results of the numerical calculations show qualitative agreement with the results of the analytical model. However, the symmetric deformation that is assumed in the numerical model is not found in the numerical simulations, but looking at the results it is reasonable to assume such a symmetric deformation as a first assumption. The order of magnitude of the deformation is the same, but this is to a large extend a matter of 'tuning' the parameters. The parameters used in the small strain model lead to a stiffer behaviour of the sand than is normally assumed. When in a case study reliable soil data are available, both the elastic parameters and the parameters from the small strain model can be determined.

The influence of unloading before loading seems to be limited and therefore an elastic model can be used. Based on analytical calculations of Wang & Dussault (1994) it can be expected that the influence of unloading becomes more important when the pressures are decreased further.

The model, as it is now, is suitable to get a qualitative indication on what is the influence of bentonite and grout flow. In this way it is a step forward compared to models that simply assume that the soil flows over the TBM and that predict a settlement trough that is too large. For quantitative results and to see what parts of the TBM are in contact with soil and what are in contact with the grout or bentonite, more sophisticated models are needed then the one described here.

6 CONCLUSIONS

A one dimensional analytical model has been developed to describe the influence of bentonite and grout flow around a TBM on the pressure distribution in the soil around the TBM and the influence on the settlement trough. Calculations with this model show that the influence of the flow alaong the TBM can be significant, leading to calculated volume losses that are more in agreement with the measurements compared to results of calculations that do not take into account the flow around the TBM. A consequence is that the TBM is (partly) not in contact with the soil. At high soil pressures and therefore also high grout pressures this may increase the risk of deformation of the tail shield.

Comparing the results of the analytical model with the results of numerical calculations showed that some of the assumptions in the model (symmetric deformation, linear elastic soil behaviour) are a somewhat crude representation of reality. This means that it is worthwhile to include the principles described in the paper in more sophisticated numerical models. The model as described here can be used for a first estimate on what is the influence of bentonite and grout flow around a TBM on the settlement trough.

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