Geotechnical Aspects of Underground Construction in Soft Ground – Viggiani (ed)
© 2012 Taylor & Francis Group, London, ISBN 978-0-415-68367-8

Flow around a TBM: A Comparison of analytical and numerical models

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ABSTRACT: The flow of bentonite and grout around a TBM influences the settlement trough that is created during TBM tunneling. Not taking into account this flow, leads to a calculated settlement trough that is too deep. Two calculation methods have been developed: a 1-D analytical model taking into account the basic aspects of this flow and assuming flow in only one direction and a 3-D numerical model, which also assumes flow in one direction, but considers the calculated width and spatial distribution of the steering gap using a hybrid Finite Element—Finite Difference model. The paper describes briefly both models (making reference to more elaborate descriptions) and compares the results of both models. It will be discussed what are the limits of the analytical model and where the numerical model has to be used.

1 INTRODUCTION

Nowadays TBM tunneling, especially in cities, has very strict requirements on the allowed settlement. Decreasing the average settlement and the maximum settlement in a tunnel project requires craftsmanship of the operators of the TBM, but also requires improved understanding of the processes that are of importance during tunnelling.

The research to improve the understanding of the tunnelling process follows generally 2 paths. One path was to simulate the tunneling process more and more accurate in numerical programs (Dijk, van & Kaalberg, 1998, Kasper & Meschke, 2004, Möller & Vermeer, 2008). With this research it is possible to improve our understanding of the tunnelling process because it is possible to study the interaction between various mechanisms, more than it is possible without these numerical programs. The other path is to study the individual mechanisms that occur during tunnelling. The basic idea here is that it is not the interaction between various mechanisms that hamper our understanding of the tunnelling process, but that not all mechanisms itself are sufficiently understood. For this second path usually model tests are performed (Merritt & Mair, 2006), field tests are analysed (Bezuijen & Talmon, 2008, Bezuijen et al. 2004) or simple models are proposed (Bezuijen, 2009).

Both these research paths are combined in this paper that presents results of research on the influence of grout and bentonite flow around the TBM on the settlement through. A simple calculation model (Bezuijen, 2009) is implemented in a 3-D FEM program (Nagel, 2009) and the results of the calculation model itself are compared with the results of the numerical program. This allows investigating where the interaction between the models becomes important.

The results of the 1-D calculation model have already be compared with the results of a 2-D numerical model (Bezuijen & Bakker, 2008), but here the TBM was not included in the model and it was only investigated what deformation would occur when the soil around a tunnel is pressurized with a liquid.

2 FLOW AROUND A TBM

A TBM is slightly tapered allowing it to maneuver in the soil, see for an example Figure 1. The drawing in this figure is not to scale because then the difference in diameter would not be distinguishable.

The cross-sectional area at the back is approximately 0.5 to 1% smaller than the cross-sectional area at the front (the actual value differs for each

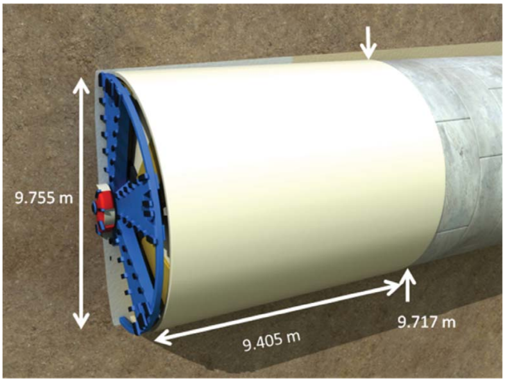

Figure 1. Dimensions of the TBM for the Botlek Rail Tunnel in the Netherlands (drawing not to scale).

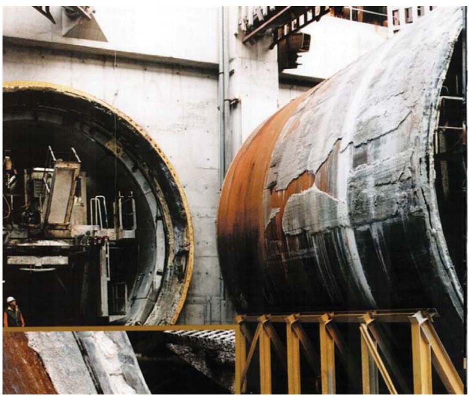

Figure 2. 2nd Heinenoord Tunnel, the Netherlands. Grout on the TBM shield after drilling of the first tube.

TBM and also depends on whether over-cutters are used during drilling). Until recently, most calculation models developed to describe the tunneling process of a TBM (Dijk van & Kaalberg, 1998, and Meschke and Kasper, 2004) assume that the soil stays in contact with the TBM over the entire shield. This means that, neglecting the soil dilatancy, the settlement trough caused by the passage of the TBM must be 0.5 to 1%. However, it was found (Bezuijen and Talmon, 2008), that in several cases the total measured settlement trough after passage of the TBM and the consolidation of the tail void grout is smaller than 0.5 to 1%. In some cases even a negative volume loss was reached.

Information from the field also showed that the assumption of the TBM shield staying always in contact with the soil is not correct. When the TBM that was used during the construction of the 2nd Heinenoord Tunnel in the Netherlands had finished the first tube and was turned to start the 2nd one, it appeared that there was grout on the shield, see Figure 2. indicating that grout can flow from the tail void along the shield.

Furthermore, measurements at the Botlek Rail Tunnel showed that the pressures measured at the tunnel face are quite comparable with the pressures measured on the shield 2.8 m from the tunnel face. This also indicates that there exists a communication between the process fluids along the shield for at least 2.8 m (Nagel et al. 2009).

3 MODELLING

3.1 *General*

Assuming that there is a gap between the TBM shield and the soil, this gap must be filled with grout, injected at the tail void, or with bentonite from the front (in case of a slurry shield TBM). The flow process, and the resulting pressure distribution was described in an analytical 1-D model by Bezuijen (2007). The principles of this model were used to enhance a 3-D FEM model to take into account a flow of process fluids within those parts of the TBM shield which are not in full contact with the soil (Nagel, 2009).

3.2 *1-D analytical model*

To model the soil deformation accurately, it is necessary to model the flow around the TBM and around the lining. The interaction between TBM, tunnel lining and the surrounding soil is governed by bentonite slurry that is injected at the tunnel face and grout that is injected in the tail void. Due to overcutting, bentonite slurry will flow along the shield skin of the TBM and the tapered shield will also lead to a grout flow along the shield skin from the tail of the TBM (Bezuijen, 2007). Possible distributions of bentonite slurry and grout are shown in Figure 3 and described below.

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 joint between the TBM and the soil, for example due to grout bleeding or penetration of bentonite into the soil). In this situation, the lowest pressure will be present where the bentonite and grout meet.
2. Bentonite flows backwards to the tail and pushes the grout out of the joint between the TBM and the soil. The pressure will be highest at the tunnel face and will decrease towards the tail. This cannot be a continuous situation, but can occur temporarily.

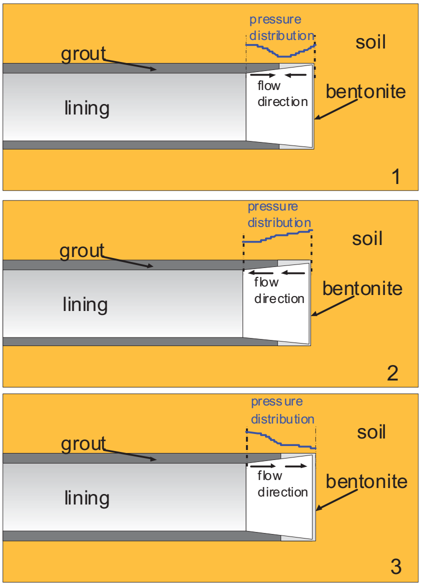

Figure 3. Possible flow directions and sketched pressure distributions along the TBM. The third situation is worked out quantitatively.

3. Grout flows to the tunnel face and pushes bentonite towards the tunnel face. The pressure is highest at the tail close to the grout injection points, and will decrease in the direction of the tunnel face. This can be a continuous situation and will be worked out further.

Assume a TBM with a small change in diameter (diameter decrease from front to back). The TBM is boring in soil that is assumed to be linearly elastic with a shear modulus G. Assume that the soil around the shield is in contact with the shield skin. This leads to a decrease in soil stresses around the TBM from the tunnel face towards the tail. A simple approach is to ignore the influence of gravity and assume a tunnel that is positioned perfectly symmetrically in the borehole. In such a situation and for linear elastic soil behaviour, the relationship between deformation and stress reduction can be written as (Verruijt, 1993):

$$\Delta\sigma = 2\frac{\Delta r}{r}G \qquad (1)$$

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 flow) will lead to an ongoing pressure reduction. However, grout is injected at the tail of the TBM. If this is injected at a pressure higher than the pressure calculated using Eq. (1), the soil will be pushed from the TBM and the situation shown in Figure 4 will occur. The grout will flow in the joint between the soil and the TBM, which will lead to a pressure drop in the grout. The pressure is highest in the tail and decreases when flowing through the joint in the direction of the TBM's tunnel face. This pressure drop is determined by the yield stress in the grout and the width of the joint. Assuming that most flow-induced friction will develop between the soil and the grout and also assuming that the grout behaves as a Bingham liquid, the pressure drop can be written as:

$$\Delta P = \frac{\Delta x}{s}\left(\tau_\gamma + \eta\frac{dv}{dy}\right) \qquad (2)$$

In (2), ΔP is the change in pressure due to the flow, Δx a length increment along the TBM, s the joint width between the tunnel and the soil, τ_γ the shear stress of the grout around the TBM, η the dynamic viscosity, and dv/dy the velocity gradient in the flowing liquid perpendicular to the joint. In Bezuijen et al. (2004) the measured yield strength of grout was assumed to be 1500 Pa, the dynamic viscosity is 50 Pa.s, and the drilling speed was less than 1 mm/s. To influence the soil deformation, the joint width between the TBM and the soil must be in the order of centimeters (because the tapering is in order of centimetres, see Figure 1), which means that dv/dy is in the order of 0.1 1/s. Viscous forces can be ignored in such a situation, because the second term on the right hand side of Equation (2) is much smaller than the first.

Using these equations and the known shape of the tunnel, it is possible to calculate the joint width that can be expected and the distance the grout in the tunnel flows over the TBM shield.

The following calculation procedure is used: the soil around the tunnel is assumed to behave as independent slices with a thickness Δx, see Figure 4. Knowing the geometry of the tunnel, the grouting pressure, the soil pressure and the elastic

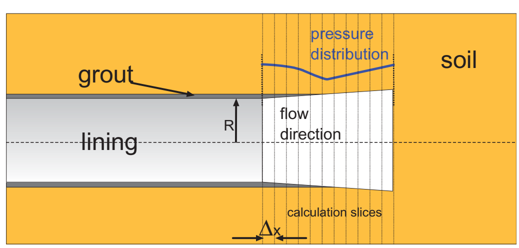

Figure 4. Idealised sketch of a TBM and lining in grout (picture not to scale).

properties of the soil, the joint width at the tail of the TBM can be calculated using Equation (1). This joint width can be used to calculate the pressure drop caused by grout flow towards the front of the TBM over the distance Δx using Equation (2). The resulting pressure (the original grouting pressure minus the pressure drop) is used to calculate the deformation and joint width of the next slice, and so on. For each calculation, a check is made whether the calculated joint width is still positive. If this is not the case, the soil is in contact with the TBM and the pressure is calculated from Equation (1) using the actual diameter of the TBM for that slice.

The same procedure can be applied for the bentonite flow when the bentonite around the shield is pushed to the front by the grout. Only the yield stress of the bentonite will be different from the yield stress of the grout and the pressure at the front is known and the pressure at the boundary between the grout and bentonite (somewhere on the shield) has to be calculated. In the program the pressure distribution of the grout is calculated from the tail to the front as well as the pressure of the bentonite from the front to the tail. It is assumed that there is bentonite when the calculated bentonite pressure is higher than the calculated grout pressure and that there is grout at the locations where the calculated grout pressure is the higher one; see also Bezuijen, 2009.

3.3 *Numerical model*

The analytical formulas for computation of the flow of grout and bentonite within the steering gap as presented in Equation (2) and used for the 1-D analytical model are implemented within a FE-model recently developed for the transient, process-oriented numerical simulation of the shield supported tunnel advance in partially and fully saturated soft soil (see Nagel et al. 2010) This simulation model allows to analyze the shield supported tunnel construction as a time variant process taking into account the TBM, the surrounding soil and the tunnel tube as separate components (see Figure 5).

The tunnel advance is simulated in a sequence of excavation and down-time steps by elongation of the hydraulic jack elements and step-wise activation and deactivation of soil, grouting mortar and lining elements. The TBM is modeled as a three dimensional deformable body connected via frictional contact to the surrounding underground and by the hydraulic jacks to the lining tube. Its conical geometry as well as a possible overcutting is taken into account. For description of the surrounding underground a CamClay-type model known as the CAS-model is used (Nagel and Meschke, 2010; Yu, 1998).

To consider within this model for an inflow of bentonite and grout into the steering gap a consecutive algorithm is applied: on the one hand side the *Finite Difference Method* (FDM) is applied to compute the pressure distribution within the steering gap based on the differential relationship given in Equation (2) (see Figure 6); on the other hand side a modified contact algorithm is introduced to take into account the effects of a pressurized liquid film between the two contact surfaces.

By application of the FDM, the pressure distribution within the steering gap is computed before each time step for simulation of TBM advance according to the actual gap width and

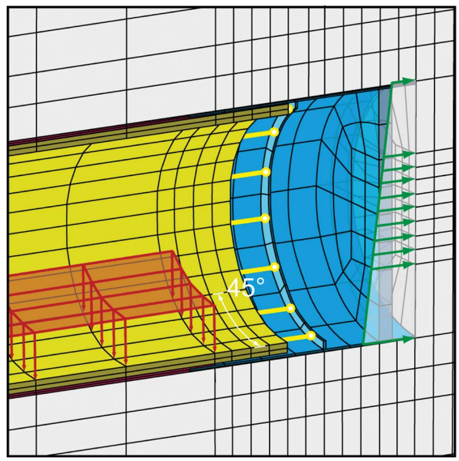

Figure 5. FE-model for simulation of TBM advance including TBM, lining tube and soil.

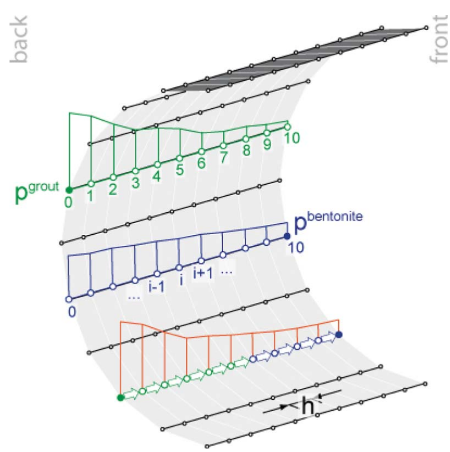

Figure 6. FDM mesh for computation of fluid pressures within steering gap.

gap distribution. Subsequently, advance of the TBM is simulated taking into account the so-obtained pressure distribution along the shield skin by application of the modified contact algorithm. Using this scheme, the pressure distribution within the gap is evaluated explicitly and held constant during the time step whereas the coupling of this fluid pressure onto the soil displacements is computed implicitly.

For the computation of the flow of the process liquids around the TBM only the third of the scenarios presented in Figure 3 is considered. To compute the pressure distribution Equation (2) is evaluated separately for both the face support and the tail void grout considering the grouting pressure at the tail as a boundary condition for the flow of the grouting mortar and the face pressure at the front end of the TBM as a boundary condition for the flow of the support medium. The FDM mesh applied to solve Equation (2) is assumed as a set of parallel lines distributed uniformly around the shield skin at constant distances (see Figure 6).

To model contact between the deformable TBM and the surrounding soil via the shield skin a surface-to-surface contact algorithm is employed. For consideration of a pressurized liquid between the two contact surfaces the contact normal force t_N acting on both contact surfaces is modified as the liquid pressure acts on both surfaces if the gap is not closed.

If the gap is opened (i.e. $s > 0$), the pressure transmitted by the liquid onto the surfaces is expressed by equivalent forces in the direction of the outward normal of the soil element facets. Thus, the normal contact force t_N is given by

$$t_N = \begin{cases} p_{liquid} & p_{liquid} \geq \langle \lambda_N + \varepsilon_N s \rangle \\ \langle \lambda_N + \varepsilon_N s \rangle & otherwise \end{cases} \quad (3)$$

Expression of the contact normal force by Equation (3) is equivalent to separation of the soil from the shield skin if the liquid pressure exceeds the normal contact force between TBM and soil.

4 COMPARISON OF RESULTS

To be able to compare the results of both models it is necessary to determine a shear modulus from the model parameters of the CAS-model used within the numerical simulation. The constitutive law in the numerical model uses a stress dependent bulk modulus (K):

$$K = \frac{\mu p'}{\kappa} \quad (4)$$

where $\mu = 1 + e$ with e the void ratio p' the volumetric effective stress and k is the slope of the URL. The relation between K and the shear modulus G can be written as:

$$G = \frac{3K(1-2\nu)'}{2(1+\nu)} \quad (5)$$

where ν is the Poisson ratio. The bulk modulus of the soil close to the TBM will be determined by the pressure loading of the grout at the tail and the pressure of the bentonite at the front. To calculate G the mean pressure was determined (270 kPa) and this value was used to calculate K and G. The other parameters of the CAS-model are assumed as: $\nu = 0.3$, $\kappa = 0.01$ and $\lambda = 0.05$. Using these parameters it was found that G was 15 MPa for the soft soil case and 30 MPa in the stiff soil case. It should be mentioned that for a tunnel in sand both values would represent 'soft soil', but here the definition given by Nagel et al. 2010 is used for consistency.

Some of the situations simulated by Nagel et al. 2010 were also simulated with the analytical model. However, in this case the only situation that can be compared is the situation where a constant pressure around the TBM is assumed, since the analytical model is a 1-D model. Consequently, for the analytical model, the shape of the gap will differ from the shape in a field situation where the pressure is a function of the height. Only the soft soil situation was simulated by Nagel in case there was 0.025 cm overcutting. Therefore only the soft soil simulations were used in the analytical model calculations.

The results are presented in Figure 7 and Figure 8.

The parameters used in the analytical model are presented in Table 1.

The analytical model presents only one value for the gap, since it is a 1-D model. The value found for the tail is in reasonable agreement in both models, when comparing the result of the analytical model with the average gap in the numerical model, see also Table 1. However, for the other locations (on the middle and at the front of the TBM) the agreement is less good. This is caused by the boundary conditions. Without overcutting it is assumed that there is no possibility for the bentonite to flow around the TBM, only grout can flow from the tail to the front. This results in a situation where at most locations the calculated pressure is lower than the soil pressure and thus the soil stays in contact with the shield skin. The analytical model works with only one soil pressure (the vertical soil pressure), Consequently, the horizontal stress in the soil is overestimated. The numerical model, which calculates both horizontal and vertical stresses shows that there is still a gap.

In the situation with overcutting the analytical model predicts that directly behind the cutting

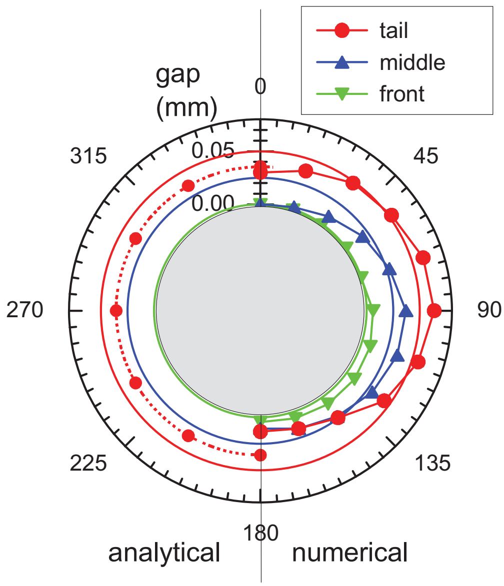

Figure 7. Gap width calculated with the numerical (right) and the analytical model (left) compared. The analytical model resulted in no gap width for the middle and front position, no overcutting, soft soil. The drawn circles over 360 deg show the theoretical opening without soil deformation.

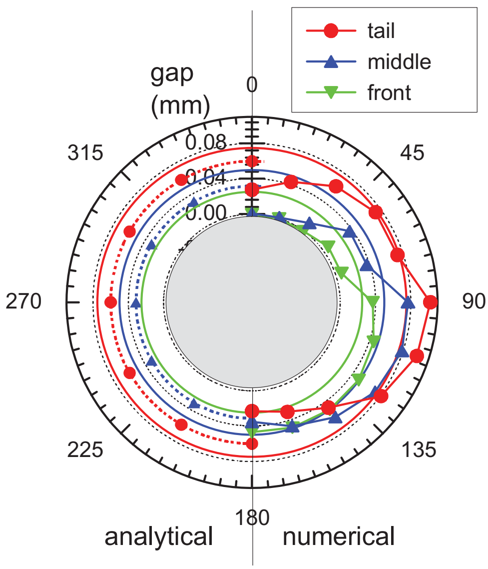

Figure 8. Gap width calculated with the numerical (right) and the analytical model (left) compared. The analytical model resulted in no gap width for the front position, 0.025 m overcutting, soft soil. The drawn circles over 360 deg show the theoretical opening without soil deformation.

Table 1. Results of the analytical and numerical models: calculated volume loss and gap width at the tail of the TBM.

	Numerical		Analytical	
Situation	ΔV %	Gap mm	ΔV %	Gap mm
Without overcutting	0.29	41	0.57	36
0.025 m overcutting	0.45	60	0.45	63

wheel the soil collapses on the TBM. This makes the calculation of the bentonite pressures along the TBM a bit arbitrary and so the gap width. In the calculation shown in Figure 8 a minimum gap width of 0.01 m at the front is assumed.

The way to calculate the pressure distribution and the corresponding gap width is the same in both methods. What is different is that a stress dependent soil model is used in the numerical model and that the position of the TBM is a result of the equilibrium forces acting on the shield skin, whereas in the analytical solution it is assumed that the TBM is in the middle of the hole. The stress dependent soil model will result in a lower Young's modulus when the stresses are lower and therefore more deformation can be expected at lower stresses.

Comparing the calculated volume loss and the average gap as calculated with both models, it appears that the volume loss is overestimated in the analytical model for the situation without overcutting. The result with overcutting shows an almost perfect agreement. As can be seen from Figure 8, this agreement is valid for the average value of the gap, not for the gap at each position. The numerical solution shows a smaller diameter vertically and a larger diameter horizontally.

5 CONCLUSIONS

The results of a 1-D analytical model for the flow of viscous process fluids along the shield skin are compared with the results of a 3-D finite element program for TBM tunneling enhanced by a contact model for the soil-TBM-interface considering fluid flow. It appeared that the 1-D analytical model is able to show the principle of what can be expected when the gap between the TBM shield and the soil is not closed. The model also shows why this gap is not closed in most cases during drilling. The 3-D numerical model goes much further. It takes into account the stress dependent behavior of the soil, the vertical gradient in the pressure (which cannot be neglected, Nagel, 2009) and the position of the TBM with respect to the hole that is drilled by the TBM.

It should be realized that the present study constitutes a comparison between calculation models only and that validation by means of measurements is needed. A first verification was possible, using the results of the Botlek Rail Tunnel (Nagel et al. 2009), however, here the instruments were not placed at the ideal positions for verification of the theory.

The inflow of process fluids into the steering gap of the TBM reduces friction and leaves more space for the TBM to move with respect to the just drilled hole. The pressure distribution around the TBM depends on the TBM position with respect to the excavated hole, which itself is influenced by various factors such as the soil stiffness, the overcut, the steering trajectory and others. On the other hand, the fluid pressure has an influence on the gap width and the distribution of the gap around the shield. To fully account for these rather complex spatio-temporal interactions during tunnel construction, the 3D numerical simulation model for TBM advance enhanced by a contact model for the fluid flow along the shield seems to be a suitable tool.

The difference in resulting settlement trough and volume loss as found with both models, taking or not taking into account the influence of the bentonite and grout flow around the TBM, shows that the flow around the TBM becomes certainly important when volume losses become less than 1% as is nowadays the case. A better understanding of the flow pressures around the TBM and the resulting soil deformations is therefore necessary to further minimize volume losses.

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