The influence of Segmental Lining in Deep TBM Tunnelling

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ABSTRACT



The shield-driven tunnelling method has been mainly adopted for the construction of urban underground tunnels in soft ground due to its flexibility, cost effectiveness and the minimum impact on the ground surface. However, due to the efficiency of the shield-driven tunnelling techniques, Tunnel Boring Machines (TBMs) have also been employed in deep, weak rock tunnelling. In shield-driven, tunnelling techniques the final support is usually composed by assembling precast concrete segments into a ring, and multiple rings placed side-by-side form the final tunnel lining. Due to the ring geometry and joint distribution, segmental liners do not show a two dimensional (2D) behaviour but rather exhibit a three dimensional one (3D). However, due to the complex geometry 2D numerical analyses are employed because of their flexibility and the reduced computational time and cost. For the purposes of this paper, two different types of concrete liners, (i) monolithic, and (ii) segmental liners, are adopted in order to investigate the influence of the in-situ conditions on the structural forces developing in the liner under different ground-tunnel interface conditions.

RÉSUMÉ

La méthode de creusement de tunnel avec bouclier a été adoptée principalement pour la construction de tunnels souterrains urbains dans les sols mous en raison de sa flexibilité, de sa rentabilité et de son impact minimal en surface. Toutefois, en raison de l'efficacité des techniques de creusement de tunnel avec bouclier, des tunneliers à bouclier (TB) ont également été utilisés pour le creusage profond dans des roches fragiles. Dans la méthode de creusement avec bouclier, le support final est généralement réalisé en assemblant des segments de béton préfabriqués dans une bague. De multiples bagues placées côte à côte forment le revêtement définitif du tunnel. À cause de la géométrie des bagues et de la distribution des joints, les segments de revêtement ne montrent pas un comportement en deux dimensions (2D), mais présentent plutôt un comportement en trois dimensions (3D). Toutefois, en raison de cette géométrie complexe, les analyses numériques 2D sont utilisées à cause de leur flexibilité, ainsi que parce qu'elles réduisent les temps et coûts de calcul. Pour cet article, deux types de revêtements en béton différents, (i) monolithiques et (ii) en segments, sont adoptés afin d'étudier l'influence des conditions in situ sur les forces structurales se développant dans le revêtement sous différentes conditions d'interface entre le sol et le tunnel.

1 INTRODUCTION

The shield-driven tunnelling method has been mainly adopted for the construction of urban underground tunnels in soft ground due to its flexibility, cost effectiveness and the minimum impact on the ground surface (Do et al. 2013). However, due to the efficiency of this tunnelling method, Tunnel Boring Machines (TBMs) have also been employed in deep, weak rock tunnelling projects, such as the Gotthard Base Tunnel which crosses the Alps and consists of two 57-km tunnels with the total overburden reaching the 2,300m (AlpTransit, 2015).

In shield-driven tunnelling, the final support is constructed by assembling pre-cast concrete segments into a ring, and multiple rings placed side-by-side form the final tunnel lining (Gruebl, 2006). The pre-cast concrete segments are connected together with joints. Due to the ring geometry and joint distribution, segmental liners do not exhibit a two dimensional (2D) behaviour but rather yield three dimensional performance (3D). However, due to the complex geometry 2D numerical analyses are employed due to their flexibility and the reduced computational time and cost. However, this poses significant limitations as a result of the assumptions made in order to model a 3D problem into two dimensions. Vlachopoulos and Diederichs (2014) summarize these tunnelling-specific limitations quite well.

When designing a segmental tunnel lining, one of the key factors that affect the performance of it is the influence of the segmental joints. A joint of a segmental liner can be considered as an elastic hinge with its stiffness characteristics influenced by a rotational stiffness K_{RO} , axial stiffness K_A , and radial stiffness K_R (Do et al. 2013). There are two methods cited in the literature, direct and indirect, in order to consider the influence of the joints on the tunnel lining. Utilizing indirect methods, the tunnel structure is assumed to be rigid and embedded in a continuous ground model (Muir Wood 1975, Einstein and Schwartz 1979) with the effect of joints usually taken into account by reducing the overall rigidity of the liner. However, the simplified analytical solutions cannot take into account the joint complexity or complex situations of the surrounding ground. In direct methods however, the joints of the segmental liner are incorporated into the tunnel liner (Lee et al. 2001, Blom 2002, Ding et al. 2004).

However, joint properties are not the only factor affecting the response and performance of a segmental liner. Ground-tunnel interaction is another important parameter that has to be taken into account, as it defines the boundary condition of the tunnel structure and it affects the behaviour of the tunnel lining. In this study, a 2D numerical model (Dassault Systèmes 2011) was developed for the analysis of the segmental lining in deep underground shield driven excavations within weak rockmasses. In the analysis, key features such as the effect of the presence of joints, the ground-tunnel liner interaction and the infilling material of the gap created between the tunnel excavation periphery and the tunnel liner have been taken into consideration and were incorporated into a finite element numerical model. The influence of certain characteristics such as the lateral pressure coefficient ratio K and the ground-tunnel interaction assumptions were examined in order to highlight the importance of the investigation of various parameters as an essential part of an effective design process.

2 NUMERICAL MODELLING

Figure 1 presents the geometry and mesh configuration of the 2D numerical model used in this paper. Away from the excavation face where the analysis takes place, it is valid to assume that plane strain conditions govern the model behaviour. Within the scope of this paper is to present a numerical modelling methodology and its impact on the design process when estimating the structural forces of the tunnel lining.

The numerical model is composed of four major components including (i) the ground/rockmass, (ii) the concrete liner, (iii) the grout/infilling material layer between the excavation boundary and the tunnel liner, and (iv) the surface interaction between the grout layer and the concrete liner. In the case of the segmental liner, joints between the segments comprise a fifth component in order to simulate the connections between the segments (Fig. 1b).



Figure 1. a. 2D Numerical model developed in Abaqus and b. Detail of the excavation area

For this study, the ground was assumed to have an elastic-perfectly plastic behaviour and the Mohr-Coulomb failure criterion has been adopted. On the contrary, for the tunnel structure, including the grout layer and the concrete liner, a lineal elastic behaviour was assumed. The material properties of the aforementioned components, including Young's modulus E, Poisson's ratio v, friction angle ϕ , cohesion c, and dilation angle ψ , are summarized in Table 1.

Table	1.	Mechanical	properties	of	the	different
compor	nents	of the numer				

Material		Elastic		Plastic		
		E (GPa)	v	φ(⁰)	c (MPa)	ψ(⁰)
Ground/Rockmass		2.1	0.25	25	1.4	4
Reinforced		30.0	0.25	-	-	-
Concrete						
Infilling		3.0	0.25	-	-	-
Material/Grout						

The numerical simulations were conducted using the Abaqus finite element program (Dassault Systèmes 2011). The ground material and the grout layer were modelled using 4-node, quadrilateral, plane-strain elements, while the concrete liner was simulated using 2-node, beam elements which utilize the Timoshenko beam theory. In the case of the segmental liner approach, the beam elements were connected together using 2-node connector elements (See Section 2.1). The concrete liner was treated geometrically as an independent entity during the creation of the model and it was connected with it via a specified surface interaction law which is established between the concrete liner and the grout layer as mentioned earlier (See Section 2.2).

In the first step of the analysis the model was set, the plane strain boundary conditions were employed and the initial stress regime was activated. Fixities were employed as boundary conditions and the initial stress state was simulated by a constant stress field which is applicable for deep excavation projects. Then, the tunnel was excavated and the rest of the features of the model, including the grout layer, the concrete liner, their surface interaction and in the segmental lining case the joints were activated in the second step. This approach is rather conservative though, since no relaxation of the rockmass between the initial geostatic state and the complete excavation phase was allowed. The adopted process though considers the worst case for the lining stress state (Do et al. 2013).

Regarding the model dimensions, general rules have been stated by various authors (Zhao et al. 2012, Graziani et al. 2007, Eberhardt 2001, Abel and Lee 1973). However, since the performed analysis involves a twostep simulation, the magnitude of the expected plastic deformation is significantly limited. Therefore, since the excavation diameter was assumed to be $D_e=10m$, a distance of $10D_e$ from the excavation both in the vertical and horizontal directions was assumed to be satisfactory in order to eliminate possible boundary effects. The numerical model is 200m wide in the x-direction and 200m height in the y-direction and consists of approximately 30,800 nodes and 30,600 elements (Fig. 1). The major geometrical features of the model are summarized in Table 2.

Component	Ground	Concrete Liner	Infilling Layer	
Height H (m)	200.00	-	-	
Width W (m)	200.00	-	-	
Excavation Radius R _e (m)	5.00	-	-	
Outer Radius R₀ (m)	-	4.88	5.00	
Inner Radius R _i (m)	-	4.48	4.88	
Thickness t (m)	-	0.40	0.12	

Table 2. Dimensions of the different components used in the numerical model

2.1 Segmental Liner Numerical Modelling

As previously mentioned, typical shield TBM tunnels are supported using pre-fabricated concrete segments, which are connected with steel bolts. Segmental liners are constructed using five to seven segments and one keyelement which is smaller than the other segments and installed last in order to complete and stabilize the ring.

The greatest structural difference between the cast insitu concrete liner and a pre-cast concrete liner is the high degree of jointing. The number of segments in each ring, anticipated joint rotation under load, the joint profile, the magnitude of the applied force between the segments in each ring, the construction quality of the ring, and the interaction between the relative rings are some of the factors affecting the overall stiffness and deformability of the segmental liner (Do et al. 2013, Litsas et al. 2015).

In order to conduct an analysis as realistic as possible when a segmental liner is to be employed, the joints have to be incorporated into the numerical analysis. Joints in segmental liners can be divided into two major groups including (i) the longitudinal joints which are created between the segments of a single ring, and (ii) the ring joints which are formed by the contact between the different rings. The longitudinal joints act in-lane and are simulated using 2-node connector elements (Fig. 2) while the ring joints which are employed in the out-of-plane direction, are not considered in this study.



Figure 2. a. 2-node JOINTC connector element in Abaqus (Dassault Systèmes 2011) b. K_A , K_R , K_{RO} stiffness in the axial, radial and rotational directions of a joint (Do et al. 2013)

These connector elements have six degrees of freedom with each one of them corresponding to a spring; three translational components along the x, y and z directions, and three rotational components around each direction respectively. Various values of stiffness can be assigned for each separate component. The most usual attachment conditions employed include: (i) free, (ii) linear spring assigned a stiffness value, (iii) bi-linear spring having a stiffness value and yield strength, and (iv) rigid (Do et al. 2013). In this study, the springs of the connector elements are assumed to be linear with the translational components being assigned a high stiffness value in order to be considered as rigid. On the contrary the rotational components are assigned approximately zero value stiffness.

Both of these assumptions are rather extreme however, they have been adopted in order to simplify the model and demonstrate the impact of the in-situ stresses on the development of the structural forces without further interference. Therefore, the high stiffness translational springs are able to transfer the shear and normal force from one segment to the other but not bending moments. However, it is conventionally agreed upon that joint stiffness depends highly on the internal forces (Blom 2002, Gladwell 1980, Janssen 1983, Muir Wood 1975, Litsas et al. 2015, Van der Vliet 2006) suggesting the use of analytical solutions and numerical analyses in the estimation of joint stiffness.

2.2 Grout Layer-Concrete Liner Interface Modelling

TBM tunnelling is a complex construction sequence when compared to conventional tunnelling methods due to the multiple components involved in the excavation process. Therefore, to conduct an as realistic as possible numerical simulation, all of these different components have to be taken into account. However, in a 2D analysis the features which have to be considered in the simulation, such as the TBM, are significantly less when compared to a 3D analysis (Kasper and Meschke 2004, Lambrughi et al. 2012, Zhao et al. 2015).

Despite the limitations of 2D analysis, it can be proven to be a useful tool in making preliminary estimations, especially when estimating the structural forces of the final tunnel support. An analysis such as this requires taking into consideration components including not only the concrete liner but also the infilling material of the tail gap between the tunnel excavation periphery and the liner; hence creating a grout layer as mentioned earlier. Therefore, it is important to include in the numerical model not just the aforementioned components but also their interaction as the grout layer and the concrete liner are not monolithically connected.

In order to take the grout layer-concrete liner interaction into account, a contact pressure-overclosure relationship was established between these two components using Abaqus. Contact relationships are usually divided into (i) "tied" contact in which the surfaces are not allowed to separate once in contact, (ii) "hard" contact which minimizes the penetration between the surfaces, and (iii) "soft" contact in which the contact pressure is a function of the clearance between the surfaces. Additionally, frictional properties can be added to the interface.

In this study, the authors examined two extreme interfaces cases: (i) No-slip between the grout layer and the concrete liner, and (ii) full-slip between the two components. For the first case, a "tied" contact relationship is employed, while for the latter a "soft" contact relationship with an exponential law was employed by assigning a zero frictional coefficient. While a "hard" contact relationship is preferred because surface penetration is required to be minimal, a "soft" exponential contact relationship (Fig. 3) was employed in order to obtain the required contact behaviour and minimize numerical issues. Additionally, numerical tests conducted comparing the results between a "tied" contact and a "soft" contact with an extremely high frictional coefficient produced the same results. This verifies the authors' selection for a "tied" contact relationship for the no-slip condition interface.



Figure 3. Exponential "softened" pressure-overclosure relationship. The surfaces begin to transmit contact pressure once the clearance between them, measured in the contact (normal) direction, reduces to c_o . (Dassault Systèmes 2011)

3 RESULTS AND DISCUSSION

In order to investigate the influence of the in-situ stresses on the internal liner forces during deep TBM tunneling in weak rockmasses three major scenarios were employed with the lateral pressure coefficient values equal to K=0.5, 1.0 and 1.5 respectively, while the tunnel overburden is H=20D_e=200m. The unit weight of the ground material is assumed to be γ =25 kN/m³. For each of these scenarios two liner approaches were examined, as mentioned, including (i) a continuous/monolithic liner approach, and (ii) a segmental liner approach for two different grout layer-concrete liner interfaces: (i) No-slip condition, and (ii) Full-slip condition.

In this study, the selected overburden height, all the material properties and the 5-joint pattern of the segmental liner approach (Fig. 4) remained constant in all of the analysis performed. The results of the aforementioned scenarios are presented and discussed in the following sections.



Figure 4. Segmental liner configuration. 1. Joint J_1 is at 54°, 2. Joint J_2 at 126°, 3. Joint J_3 at 198°, Joint J_4 at 270°, and 5. Joint J_5 at 342°

3.1 Impact of the Concrete Liner Approach

In this section, the results for a single K ratio and a single type of grout layer-concrete liner will be discussed in order to examine the overall effect of the liner approach adopted in the numerical simulation. The scenario discussed is for K=1.5 and a "no-slip" grout layer-concrete liner interface.

In Fig. 5, the bending moment [M] and normal force [N] are illustrated along the tunnel liner periphery. It can be observed that the presence of joints reduces the magnitude of the generated bending moment and alters its distribution, especially when the joints are located to positions of high magnitude moments such as J_3 , J_4 and J_5 . However, joint contribution decreases when they are close to zero value bending moment locations such as J_1 and J_2 . On the contrary, normal force seems to be insensitive to the presence of joints which is in agreement with the work of Do et al. (2013).



Figure 5. Comparison of tunnel liner bending moments [M] and normal forces [N] between a monolithic (red) and a segmental (purple) liner approach for K=1.5

3.2 Impact of the Lateral Pressure Coefficient

The orientation and the magnitude of the stress regime have a significant impact on the internal forces created within the tunnel liner affecting both their magnitude and their distribution around the liner. In order to investigate the effect of the in-situ stresses on the tunnel liner different scenarios for the each of the K=0.5, 1.0 and 1.5 under the "no-slip" condition for both liner approaches were examined.

When K=0.5, the roof and the invert of the tunnel are going to be under tension while both spring lines are under compression as illustrated in Fig. 6. As discussed in the previous section, the joint at J_4 reduces the compressive bending moment at the right spring line, while the left spring line has the same maximum value. Regarding the roof and the invert, the maximum tensile bending moment is similar between the two liner approaches with the employed joint pattern. Additionally, in Fig. 6, the normal force plots are approximately the same for both liner approaches, as previously discussed, with the maximum compressive normal force occurring at the right and left spring lines and the minimum at the roof and the invert.



Figure 6. Comparison of tunnel liner bending moments [M] and normal forces [N] between a monolithic (red) and a segmental (purple) liner approach for K=0.5

In Fig. 7 and for K=1.0, the bending moment and normal force plots are illustrated. As expected, it can be observed that the bending moment for both liner approaches is limited and the normal force has an approximately uniform distribution around the tunnel, as a result of the isotropic stress state. Therefore, in such a case the presence of joints does not affect the internal forces of the liner.

For K=1.5, in Fig. 5, both the bending moment and the normal force plots are illustrated. It becomes evident that since the maximum in-situ stress is the horizontal, the maximum tensile bending moments develop at the left and right spring lines while the maximum compressive moments are located at the roof and the invert of the tunnel. Again, in this case it can be observed the effect of the joints and the decrease in the moment magnitude at specific locations. Regarding the normal force and for both liner cases, the roof and the invert have the greatest compressive normal force. On the contrary the minimal values are located at the left and right spring line. Both liner approaches produce approximately the same normal force along the periphery of the support with the presence of joints not affecting it.



Figure 7. Comparison of tunnel liner bending moments [M] and normal forces [N] between a monolithic (red) and a segmental (purple) liner approach for K=1.0

3.3 Impact of the Grout Layer-Concrete Liner Interface

When employing a Finite Element Analysis (FEA), it is common practise to consider the different model components attached to one another by sharing common nodes. However, in a mechanized tunnel process the prefabricated lining segments and the infilling material between the excavation periphery and the concrete liner are not monolithically connected, and therefore, an interface between these two components is created. Hence, this interface has to be taken into account in the modelling process and simulated properly, as mentioned earlier.

In Fig. 8, the bending moment and normal force results, for K=1.5 and a monolithic/continuous liner approach, are illustrated for a "no-slip" and a "full-slip" condition. It can be observed that the both the bending moment and the normal force are affected depending on the adopted assumption. A "full-slip" assumption results in higher magnitude moments, as it can be observed, and an approximately uniformly distributed normal force around the tunnel liner. On the contrary, a "no-slip" assumption results in lower magnitude bending moments and the normal force is not uniformly distributed and its minimums and maximums depend on the in-situ stress orientation and magnitude. For most tunnels, the condition on the tunnel-ground interface is between a "full-slip" and "no-

slip" condition, therefore both cases must be studied for critical forces on the lining (Giannakou et al. 2005).



Figure 8. Comparison of tunnel liner bending moments [M] and normal forces [N] between a "no-slip" (red) and a "full-slip" (purple) grout layer-concrete liner interface assumption for K=1.5

3.4 Parametric Investigation and Further Discussion

In the previous sections, the influence of the liner approach adopted, the initial in-situ stress state and the grout layer-concrete liner interface were discussed. However, these are just a few of the factors affecting the internal forces associated with a tunnel liner within such conditions. Other factors that must be accounted for include the relative stiffness and interaction between the ground, the grout layer and the concrete liner, the stiffness and strength of the ground, the joint number and pattern for a segmental liner approach etc..

Investigation of these factors is crucial for a design process but sometimes their interaction may be even more important; hence a holistic approach has to be adopted and combinations of different scenarios have to be modelled. In Fig. 9, the maximum normalized normal force and maximum bending moment of the segmental liner within respect to the monolithic/continuous liner are illustrated for the different K ratios. Regarding the normal force, it can be inferred that despite of the initial stress state segmental the normal force is insensitive to the liner approach (as also mentioned earlier). The bending moment is controlled by the joint presence and the deviatoric stress q; therefore, governed by the orientation of the initial geostatic stresses. For the cases of K=0.5 and 1.5, the deviatoric stress is the same resulting in the same normalized values. However, in this case the selected joint pattern did not affect drastically the magnitude of the greatest bending moment resulting in the same maximal values for both liner approaches.



Figure 9. Normalized bending moment M_s/M_c and normalized normal force N_s/N_c (reference monolithic/continuous liner) for K=0.5, 1.0 and 1.5 for both "no-slip" and "full-slip" conditions

In Fig. 10, using as a reference the "no-slip" assumption, the maximum normalized normal force and bending moment are demonstrated. It can be inferred again that the normal force is insensitive to the liner approach adopted but it is affected by both the grout layer-concrete liner interface and the in-situ stress state. However, and as stated earlier, the deviatoric stress q governing the bending moment is the same for K=0.5 and 1.5 and in order to draw a conclusion more cases have to be investigated.



Figure 10. Normalized bending moment M_f/M_n and normalized normal force N_f/N_n (reference full-slip/no-slip interface condition) for K=0.5, 1.0 and 1.5 for both a monolithic/continuous and a segmental liner approach.

In Fig. 11, the normal force and bending moment are normalized in respect to the lateral pressure coefficient K=1 case. For the normal force it can be inferred that for the "full-slip" condition, the liner approach adopted does not affect the results but as there is an increasing trend as the K ratio increases, showing that the in-situ stress magnitude affects highly the normal force. The same can be inferred for the "no-slip" condition. However, for K=0.5 and 1.0, the major principal in-situ stress is the same resulting in unity normalized values but for K=1.5 this upward trend appears as well. Regarding the bending moment, it can be inferred that for K=0.5 and 1.5 for each specific liner case (liner approach and interface conditions the same) the normalized value is approximately the same. For different approaches though, it can be observed that the worst case scenario for both K values is to assume a monolithic/continuous liner under a "full-slip" condition while the lowest bending moment occurs for a segmental liner under the "no-slip" condition.



◆ Con-No_Slip □ Con-Full_Slip △ Seg-No_Slip × Seg-Full_Slip Figure 11. a. Normalized bending moment $M_K/M_{K=1}$ and b. normalized normal force $N_K/N_{K=1}$ (reference K=1 case) for both a monolithic/continuous and a segmental liner approach and for both a "no-slip" and "full-slip" grout

4 CONCLUSIONS

layer-concrete liner interface.

A 2D finite element numerical analysis is presented in this study in order to simulate segmental liners in shield-driven tunnels at high overburdens within rockmasses. This work focuses on the key features that should be incorporated into the numerical model when examining the response of a segmental liner which is typically employed in shielddriven tunnelling. These features include the joint properties and pattern, the infilling material between the tunnel excavation periphery and the tunnel lining, and the interaction between this infilling material and the tunnel lining. Despite being useful, a 2D numerical analysis is not the most appropriate type of analysis, as both shielddriven tunnelling and segmental liners are strongly associated with 3D phenomena which dominate their response. However, preliminary results can assist in investigating the impact of different parameters and increase the design efficiency.

In order to highlight its importance in the design process, a series of different scenarios is conducted in this study focusing on the in-situ stress regime and its effect on the internal forces of a liner under different assumptions including a comparison between a monolithic/continuous liner and a segmental liner approach and the grout layer-concrete liner interface. In the conducted analyses, different K ratios are examined in order to show how the orientation of the initial in-situ stresses interacts with the presence of joints and its impact on the distribution of the bending moment and normal force along the liner. Furthermore, the grout layerconcrete liner interface is simulated using a "no-slip" and a "full-slip assumption". By examining these two extreme cases, a bending moment and normal force envelope can be derived for the different liner approaches which may assist in the design process along with other preliminary results.

However, a 2D numerical analysis cannot capture the effects such as the presence of the TBM in the excavation or of the joints connecting the rings along the tunnel axis in the out-of plane direction and how it affects the rigidity of the overall tunnel structure. Therefore, 3D numerical modelling is crucial if a shield-driven tunnelling process is to be simulated as realistically as possible.

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