Ground Relaxation in Segmental Lining Design Using the Convergence-Confinement Method

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ABSTRACT

When carrying out 2D plane strain numerical analysis for TBM segmental lining design, the effects of ground relaxation between the TBM cutter head and the back of the TBM shield where the segments are erected needs to be considered. A tunnel relaxation factor can be defined to model this behavior. This factor is dependent on the overburden and cover in a competent geological formation, ground geotechnical properties, tunnel diameter, and pre-support measures such as face and annular pressures in closed face TBMs. There is a common misconception regarding the link between the concept of volume loss and tunnel relaxation. For the calculation of ground movements induced by tunnel construction and building damage assessment, it is common practice to assume a high volume loss and relaxation factors with the aim of generating the maximum expected ground convergence and therefore movements of any affected asset. Conversely, for lining structural design, both lower and upper bound convergence should be checked. This is to ensure the most onerous ground load case is addressed. It is therefore vital to assume a relaxation value for lining design that is not a function of the contractual volume loss but of the range of TBM confining pressures which can be directly monitored during construction. This paper presents a methodology for calculation of ground relaxation for closed face TBM tunneling using the convergence-confinement method which integrates the assessment of induced ground movements and lining design. This method is commonly used in the design of conventional mined tunnels, where the lining installation occurs in stages in relatively close proximity to the face (1 to 3 meters). However, the method is also applicable to a TBM tunnel, where the lining installation occurs much further from the face at the end of TBM shield with the additional complication of the face and annular pressures imposed when the tunnel is constructed using a closed face TBM. The methodology was successfully applied to several recent global tunneling projects. A parametric study is included and the results are compared to other alternative approaches. The method is then used to carry out back analysis of ground movement data for TBM tunnel recently constructed in completely decomposed granite (CDG) in Hong Kong.

1 INTRODUCTION

The analysis of bored tunnel linings in soft ground is usually carried out using continuum 2D analytical or numerical calculations. When defining the 2D load applied to the tunnel lining, the available methods can be classified into four types (AFTES, 2001):

- Assuming that the tunnel is fully loaded by the geostatic stress known as Full Overburden load. This is usually an unrealistic assumption for most tunnels due to the fact that it completely ignores the stress arching around and ahead of the tunnels. The assumption of full overburden is only rarely used on shallow tunnels in very soft ground.
- Empirical methods indicating a rock support pressure defined directly from a classification system. The most widely used methods are the Beinawski (1989) RMR and Barton (1975) Q system. These can be useful aids at the project planning stages for tunnel design in hard ground but have significant disadvantages in weaker materials which include omission of ground-structure interaction and reliance on case histories which are not always compatible.
- Analytical methods for determining the loads acting on the support, regardless of support type and deformation. The most widely use effective stress method is the Terzaghi (1943) arching method which has been developed further by Szechy (1970). These methods assume that a collapse
mechanism is formed above the crown of the tunnel which is the only load applied and the corresponding displacements are not considered. For closed face TBM tunnels in soft ground the confinement provided by the slurry pressure is significant in limit deformations and thus the load on the lining is between full overburden and the collapse mechanism load. As such for this method of construction the use of the arching method is usually non-conservative and the actual load may be much higher. This will be discussed and demonstrated in Section 3.

- Analytical or numerical methods collectively known as “Convergence-Confinement Methods” which can account for elasto-plastic ground-support interaction by providing equations for calculating the ground stress-strain curve as well as the stiffness of the lining. These methods are applicable to both rock and soils. The most widely used analytical solutions are the theory presented by Panet & Guenot (1982) for cohesive and non-cohesive soils and the effective stress method by Hoek et al. (1980; 2007; 2008) and Vlachopoulos & Diederichs (2009) for soils and weak rocks.

This paper will present a methodology for calculation of ground loading for TBM tunneling using methods which can account for both ground and lining deformations when assessing the load on the tunnel in soft ground. The basic theory of the Convergence Confinement Method will be presented followed by an adaptation of the method to TBM tunnels.

2 ANALYSIS OF GROUND RELAXATION

2.1 The Convergence Confinement Method

The progress of excavating and installing delayed support in tunnel in soils can be calculated using the theoretical framework set out by Panet & Guenot (1982) and Panet (1995) which has been developed further by numerous authors and established as a standard method in AFTES (2001). The method is known as the Convergence-Confinement Method (CCM).

This method is regularly used in the design of mined tunnels, where the lining installation occurs in stages in relatively close proximity to the face (1 to 3 meters). The same methodology applies to TBMs which work in open face mode and the lining is assumed to be installed at least one diameter from the face. In the case of TBMs using pressurized shields, the method requires some modification as the convergence is affected by the face pressure applied by the TBM cutterhead and radial confinement around the shield. In addition, the lining is initially not directly acting against the ground, but it connected through an intermediary grout injected under the annular gap pressure.

Whether the analysis is performed using analytical (closed form) solutions or with plane strain numerical models, a Longitudinal Displacement Profile (LDP) and Ground Reaction Curve (GRC) are required to relate tunnel wall deformations at successive stages in the analysis to the actual physical location along the tunnel axis. This provides a percentage of the in-situ ground stress ($\sigma_0$) and is commonly referred to as the deconfinement rate or relaxation factor ($\lambda$).

As shown in Figure 1, the relaxation factor can be illustrated by considering the volume excavated is initially filled with a fictitious radial fluid pressure ($\sigma_r$) which gradually decreases $\sigma_0$ down to 0. If this fictitious pressure is written as $(1 - \lambda) \sigma_0$, we see that the relaxation factor is not a constant but function of the position $x$ of the face with respect to the studied section $\lambda(x)$, which ranges from 0 (very far ahead of the front) to 1 (far behind the face). This procedure is commonly known as the stress reduction method.

![Figure 1: Stress reduction method conceptual model (after Panet & Guenot, 1982)](image-url)
An illustration of the stress reduction method and CCM principles are presented in Figure 2. The mathematical relationship relating \((1 - \lambda) \sigma_0\) to the radial convergence \(U/r\), gives the GRC curve. The LDP function is then constructed separately to determine the radial convergence ahead or behind the face \(X_d\). This then provides the link between the LDP and GRC and allows to determine the relaxation factor at the location of support installation \(\lambda\). The load taken by the support at the point of force equilibrium is then given by the interaction point between the GRC and the Confinement Curve which is a simple stress-strain function derived from the stiffness of the support.

Figure 2: Convergence confinement curves (after Panet & Guenot, 1982 and Hoek, 2007)

2.2 Loads on TBM tunnel linings using CCM

Aristaghes & Autuori (2001) introduce an explicit method to assess the behavior of bored tunnels by accounting for the TBM face and annular pressure \(P\) using the principles of the CCM. The pressure \(P\) provides a balancing force to the pore water pressure in the ground \(u\) and also provides a partial restraint to the in-situ effective stresses. The overall relaxation is therefore given by consideration of the construction sequence and the difference between the ground forces including water and the applied face and annular pressure and not the full load. The method proposed is ideal for simulation using multi-stage finite element numerical analysis as follows:

- Stage 1 – excavation from TBM face to end of shield: The stress in the soil around the excavation is loaded by the deconfinement pressure as follows:

\[
\sigma'_{\text{soil}} = \lambda(\sigma'_0 - P')
\]

Where: \(\lambda\) = relaxation factor calculated using CCM equations; \(\sigma'_0\) = initial effective stress = \(\sigma_0 - u = (\sigma'_v + \sigma'_h)/2\); \(P' = P - u\) = TBM effective confinement pressure (difference between earth or slurry pressure and the hydrostatic pore water pressure).

- Stage 2 – end of deconfinement at point of lining installation: The stress in the soil applied is then given by the remaining deconfinement forces as follows:

\[
\sigma'_{\text{soil}} = (1 - \lambda)(\sigma'_0 - P')
\]

The resulting effective load on the lining when \(P'\) is removed (i.e. far away from the shield) will thus be given by the following equation:

\[
\sigma'_{\text{Lining}} = (1 - \lambda)(\sigma'_0 - P') + P' = (1 - \lambda)\sigma'_0 + \lambda P'
\]

It is clearly shown that the load applied on the lining will increase as a result of the TBM confining pressure. When the TBM pressure is only used to balance the water pressure (i.e. \(P' = 0\)) then the overall relaxation will be equal to an open face excavation in terms of effective stresses.

The TBM pressure \(P\) can be exerted in different ways, depending on the type of TBM: compressed air pressure, slurry pressure or earth pressure. If the stability of the ground is not ensured only by the ground strength characteristics, \(P\) must not only compensate for water pressure, but should be such that the effective pressure \(P'\) is sufficient to stabilize the soil face (Anagnostou & Kovari; 1996). In some cases, particularly in
proximity of sensitive structures, $P'$ may be even higher in order to control surface deformations on nearby foundations. These considerations have a significant effect on the load applied to the tunnel lining.

2.3 Analytical solution for $\lambda$ with TBM pressure

A key assumption of the method presented in Section 2.2 is that the relaxation factor $\lambda$ can be derived from CCM curves. However, this assumption requires that the calculation of both LDP and GRC curves is adjusted to incorporate the effect of the TBM pressure. Another assumption in the method presented is that the ground response up to installation of the lining is elastic. This assumption is not always satisfied particularly for low TBM confining pressures in weak ground under relatively high geostatic stress.

An analytical elasto-plastic axis-symmetric solution is thus proposed for the calculation of LDP and GRC curves with the inclusion of TBM pressure. The solution is based on the methodology proposed by Hoek (2007; 2008) and Vlachopoulos and Diederichs (2009) which is modified for the inclusion of the TBM effective confining pressure $P'$.

The methodology assumes that the cavity behavior can be represented by a plastic zone which gradually forms around the excavation (see Figure 3) in accordance with the deconfinement away from the face. The plastic radius equation given by Hoek (2007) utilizes an effective stress Mohr-Coulomb envelope failure criteria. The solution allows for the calculation of the GRC curve by iterating

$$\lambda = (1 - \frac{P}{P_0}) : 0 \rightarrow 1$$

for which $P_i$ and $P_0$ are modified to include the TBM confining pressure.

The relaxation $\lambda(x)$ at the location of lining installation can then be obtained by constructing the LDP curve elasto-plastic solution given by Vlachopoulos and Diederichs (2009) which is modified here for the inclusion of TBM pressure:

$$u(x) = u_{(\lambda=1)} \left( 1 - \frac{P'}{\sigma_0} \right) \left[ 1 - \left( \frac{u_f}{u_{(\lambda=1)}} \right) \left( \frac{3x}{\sigma_0} \frac{2r_p}{\sigma_0} \right) \right] = \lambda^* u_{(\lambda=1)} \left[ 1 - \left( \frac{u_f}{u_{(\lambda=1)}} \right) \left( \frac{3x}{\sigma_0} \frac{2r_p}{\sigma_0} \right) \right]$$

(4)

Where: $x =$ distance behind the face; $u_f =$ radial displacement at the face of the TBM and is a function of the plastic radius; $u_{(\lambda=1)} =$ final radial displacement at a distance where $\lambda = 1$ (if no support if provided); $u(x) =$ Longitudinal Displacement Profile (LDP).

The modified solution presented in Eq (4) is similar to the one presented by Vlachopoulos and Diederichs (2009) except for the inclusion of the term $(1-P'/\sigma_0)$ which is introduced here as the TBM Confinement Ratio ($\lambda^*$). A parametric study of the effect of the $\lambda^*$ on tunnel design using the CCM is given in Section 3 for a typical tunnel in soft ground.
2.4 Volume loss and relaxation in bored tunnels

The linkage between relaxation factor and volume loss is the ground convergence occurring ahead and behind the face until the ground has loaded the lining. As discussed above, this convergence occurs due to stress relaxation during the excavation of the tunnel which is significantly influenced by the TBM confining pressure.

It is noted that this TBM pressure is the only parameter that can be directly and reliably monitored and controlled during construction. Other components which may produce additional volume loss are related to areas where the soil is not in direct contact with the TBM (e.g. over-excavation, tailskin gap etc.). These are almost impossible to monitor directly or predict with any certainty during design and construction of the tunnel.

Additionally, other processes such as ground creep and consolidation often result in more movement and an apparent increase in volume loss back-analysed from the ground settlement profile. However, although these processes are important, they often occur well after the TBM has passed and lining installed, and thus the analysis accounting for these processes should be separated from that used to estimate either the relaxation or volume loss during construction.

It is thus important to consider the aim of the analysis when determining the volume loss and relaxation factor:

- For damage assessment carried out with the aim of capturing the maximum ground movements, calculation of ground movements induced by tunneling should assume high volume loss based on allowable movement. This maximum volume loss should then be linked to a minimum allowable TBM confining pressure which must be equal or above the one required to maintain the stability of the excavation.

- For lining design, the ground movement assumed (i.e. volume loss) is directly proportional to the relaxation therefore dictating the maximum and minimum possible ground load in the lining as shown by equations (1) and (2). Although the maximum ground load (and lower relaxation) will induce high lining forces these will not always be the most onerous case due to increased ground confinement which contributes to stability. Lower ground loads corresponding to higher ground movements will thus sometimes dictate the design where higher distortions are induced with little ground confinement.

- As such for lining design one should assume both minimum and maximum allowable TBM confining pressures correlating to lower and upper bound volume loss. These pressures should be determined from the minimum pressure to maintain face and radial stability and the maximum pressure governed by the risk of blowout or TBM flotation. This assessment should be decoupled from the contractual volume loss given by independent ground movement assessments.

3 PARAMETRIC ANALYSIS

The calculation methodology presented in Section 2 is now investigated for a typical tunnel in Hong Kong as illustrated in Figure 4 below.
3.1 Ground response and relaxation

The calculated GRC and LDP curves providing the resulting tunnel convergence (i.e. theoretical volume loss) and relaxation factor $\lambda$ at a distance of $1D$ from the face for a range of TBM confining pressures are presented in Figure 5. The TBM confining pressure is represented by the TBM confining ratio ($\lambda^*$).

![Figure 5: GRC (left) and LDP (right) curves for different TBM pressures and resolved convergence and relaxation](image)

3.2 Numerical model of segmental lining

The relaxation factors calculated in Section 3.1 are now used to carry out a Plaxis 2D finite element numerical model in accordance with the methodology presented in Section 2.2. A segmental lining composed of 300mm thick, $6+1$ No. concrete segments is assumed. The numerical model and the resulting forces plotted in a lining capacity N-M diagram are presented in Figures 6 to 7.

![Figure 6: Plaxis 2D finite element model](image)

Stage 1: Apply relaxation factor and TBM internal pressure (assume no excess water pressure, i.e. drained analysis).
- Stress in soil given by Eq. 1

Stage 2: Install lining apply remaining load and remove TBM internal pressure.
- Stress in soil given by Eq. 2

\[ \sigma_0 = 600\text{kPa} \]
\[ u = 300\text{kPa} \]
\[ P = 360 \text{ to } 480\text{kPa} \]
\[ \lambda^* = 0.4 \text{ to } 0.8 \]

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\[ u = 300\text{kPa} \]
\[ P = 0\text{kPa} \]
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$\sigma_0 = 600$ kPa

$u = 300$ kPa

$P = 360$ to 480 kPa

$\lambda^* = 0.4$ to 0.8

Stage 2: Install lining apply remaining load and remove TBM internal pressure.

$\sigma_0 = 600$ kPa

$u = 300$ kPa

$P = 0$ kPa

Figure 7: Lining force calculated in the numerical model for different TBM confining pressures plotted in an N-M capacity diagram for a C50 concrete with a nominal 0.1% reinforcement ratio

As shown in Figure 6, the increased TBM confinement results in more ground load imposed on the lining in stage 2 of the analysis resulting in increased hoop forces and bending moments. However, as discussed in the Section 2.4, the overall increase is shown to actually contribute to stability in this case due to the fact that the N-M envelope is sloping upwards as well. This clearly demonstrates that in analysis of segmental linings investigation of both upper bound and lower bound TBM confining ratios is required to ensure a robust design.

3.3  Comparison to alternative methods

In order to provide a verification of the convergence-confinement methodology described above the results are compared to the alternative approach of Terzaghi arching theory discussed in Section 1 which gives an estimate of vertical effective stress on the tunnel lining (Szechy, 1970):

$$\sigma'_v = \gamma'(r_0 + \tan(45 - \phi'/2))/\tan\phi' \tag{5}$$

The vertical ground effective confining stress on the lining in the numerical model can be calculated from the average hoop force $\overline{N}$ as follows:

$$\sigma'_v = 2\sigma'/((1 + K_0) = 2(\sigma - u)/(1 + K_0) = 2(\overline{N}/r_0 - u)/(1 + K_0) \tag{6}$$

The comparison of results is given in Table 1 below:

<table>
<thead>
<tr>
<th>TBM Confinement Ratio</th>
<th>Relaxation applied in numerical model ($\lambda$)</th>
<th>Full Overburden Effective Vertical Stress (kPa)</th>
<th>Terzaghi Effective Vertical Load (kPa)</th>
<th>CCM Effective Vertical Load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>0.35</td>
<td>300</td>
<td>103.5</td>
<td>261.3</td>
</tr>
<tr>
<td>0.7</td>
<td>0.75</td>
<td></td>
<td>153.9</td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>0.90</td>
<td></td>
<td>95.8</td>
<td></td>
</tr>
</tbody>
</table>

It is shown that the CCM and Terzaghi arching give comparable lower bound vertical loads when the TBM confining pressure is low and the relaxation $\lambda \rightarrow 1$. This is compatible with the fact that for high relaxation values the CCM predicts high plastic strains as shown Figure 5 (i.e. high volume loss) and thus the initiation of a failure mechanism above the tunnel. However, as shown the Terzaghi method significantly underestimates the load on the tunnel with increasing TBM confining pressures which limit soil displacements (i.e. low volume loss) and result in lower relaxation factors. It is also shown that, as expected, when $\lambda \rightarrow 0$
the CCM analysis results in upper bounds loads which approach the full overburden case due to limitation of ground arching around the tunnel. It is thus demonstrated that the CCM method can capture both upper bound and lower bound load cases by consideration of the TBM confining pressures.

4 CASE STUDY: SHATIN TO CENTRAL LINK C1103, HONG KONG

The Contract 1103 of the SCL consists of 4km of line between Diamond Hill and Hin Keng stations. A 1.7km section of 7.1m OD twin bored segmentally lined tunnels connect Diamond Hill Station and the Lion Rock mined tunnel. The tunnel was excavated using a slurry TBM through Completely to Moderately Decomposed Granite (CDG to MDG) with the water table approximately at surface level. The tunnels at the beginning of the drive are approximately 25-35m deep. Construction of the uptrack tunnel commenced in August 2014 and was completed in June 2015.

Monitoring data of the TBM parameters and ground surface settlement at the beginning of the drive (near Diamond Hill station) was available for this study as shown in Figure 8 below.

Back analysis of largest displacement Section CS1-CS1b (CHD 97+041) was carried out based on the observed deformations and TBM parameters. Two types of analysis were performed:

1) Back analysis using a 2D Plaxis numerical model by directly applying a volume loss inside the tunnel.
2) Back analysis using the CCM methodology as follows:
   - Calculating the expected relaxation using the CCM analytical solution described is Section 3. The analytical solution is given Figure 9.
   - Back analysis of observed displacements using 2-stage 2D Plaxis numerical analysis in accordance with Section 2 by applying the average observed TBM confining pressure of 280kPa. The Plaxis model is shown in Figure 9. The tunnel is shown to be constructed at 25m depth below ground level in the CDG geological formation. The modelled tunnel lining and CDG strength parameters are similar to those defined in Section 3. The depth-dependent stiffness of the CDG as given by SPT site investigation data was explicitly modelled. The 3m high embankment on the north side of the tunnel was modelled by applying a surcharge at surface level.

Figure 8: Observed ground surface displacements and TBM confining pressures during tunnel construction next to Diamond Hill Station (Chainage CHD97+052). Tunnel depth at 25.4mBGL, water level assumed at surface.
The results from the back analysis are shown in Figure 10. For the purpose of the back analysis, the measured displacements were corrected for approximately 2mm of recorded seasonal displacements which were clearly observed in the site monitoring data.

The following can be observed:
- Generally a good match to the observed settlement troughs is achieved with both forms of analysis with minimum and maximum measured displacements within 10% when corrected for seasonal variations
- A better fit is observed on the southern side (left) where there are no significant surface structures and weak alluvium and colluvium deposits. The presence of an embankment on the northern side (right) is shown to induce additional surface displacements. These are only partially captured by the Plaxis model due to the simplifying assumptions deployed. However, this is of little significance to the purpose of this study.
- The CCM numerical analysis is shown to give identical results to the trough calculated using the direct volume loss approach giving approximately 0.55% maximum value. It is noted that this volume loss is lower than the 1% contractual volume loss assumed for the SCL1103 project.
5 CONCLUSIONS

This study has presented a methodology for analysis of tunnel linings constructed in soft ground using closed face TBMs by utilizing the concepts of the Convergence-Confinement Method (CCM). The findings of the study can be summarized as follows:

- The CCM is a theoretically rigorous analytical method that can account for ground-structure interaction when assessing the load on a tunnel and is thus naturally suitable for numerical modelling and monitoring during construction. As described in Section 2 the method can be easily adapted to include TBM confinement by using theoretical solutions that account for the formation of a radial plastic zone around the tunnel to obtain the Ground Response Curve (GRC) and implementing the concept of the TBM Confinement Ratio ($\lambda^*$) for obtaining the Longitudinal Displacement Profile (LDP). These can then be used to derive the relaxation factor ($\lambda$) which is then utilized to carry out ground-structure 2D numerical analysis of the tunnel lining.

- As shown in Section 3, when compared to other traditional analytical methods such as Full Overburden or Terzaghi Arching the CCM is superior due to its ability to relate ground deformations and method of construction to the ground load applied to the lining. This is of particular significance for tunnels constructed using closed face TBMs where the TBM confining pressure is directly affecting the stress path in the ground. For low confining pressure, ground deformations are maximized giving lower bound lining forces. Correspondently, high TBM confining pressures give lower bound ground movements and upper bound lining forces.

- The ability of the CCM to relate the method of construction to the ground deformations allows it to be used as a means for providing a theoretical estimation of the volume loss in TBM construction as demonstrated in Section 4. This is achieved by the same procedure used to calculate the ground relaxation factor $\lambda$ and can thus be correlated to the loads imposed on the lining and the TBM confining pressure that can be directly monitored. For damage assessments where ground movements are to be maximized, lower bound TBM confining pressures (limited by the requirement to maintain face stability) should thus be used and compared to the structure deformation limit.

- Complications arise when the additional long term movements that are not directly related to the ground movements during construction are “lumped” together when determining the contractual volume loss. In these instances, the contractual volume loss should not be used to carry out tunnel design as it may grossly underestimate the loads on the lining as well as disassociate the actual performance of the TBM from observed deformations.

As such, although the traditional concepts such as empirical volume loss and simplified analytical methods have been used in the past, it has been demonstrated in this study that these are not always on the conservative side and can rarely be verified with certainty on site. It is thus recommended that future designs of TBM tunnels in soft ground utilize the CCM concepts presented in this paper which will ensure that both ground movements and lining forces are addressed in an appropriate manner.

Furthermore, the back analysis presented above has shown that it is possible to obtain a very reasonable match to the observed displacement trough. This is in contrast to the fact that finite element analyses utilizing simple linear isotropic soil constitutive models are known to have difficulties in predicting the settlement troughs induced by tunneling works. The reason for the success of the back analysis is assumed to be that the CDG at the investigated location is by large a normally consolidated homogenous non cohesive soil, with isotropic stiffness and strength exhibiting relatively little dilation and negative excess water pressures during short term unloading. These characteristics make it suitable for the standard Mohr Coulomb model as demonstrated in numerous historic and current projects in Hong Kong.

ACKNOWLEDGEMENTS

The authors would like to thank to MTR Corporation Limited in Hong Kong for their ongoing support and permission publish information contained in this paper. Special thanks is also given to the design and construction teams in Arup and Vinci whose dedication and expertise has led to the success of the SCL1103 project providing the basis for this study.
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