# Ground movements during tunnel construction by EPB method along the southern extension of Athens Metro Line 2, Greece

## Mouvements de terre pendant la construction du tunnel avec la méthode EPB pour l'extension sud de la Ligne 2 du Métro d'Athènes, Grèce

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ABSTRACT
This work is based on a significant number of field records, monitoring data of ground movements and data from the EPB machine used for the excavation of the southern extension of Athens Metro Line 2. Several issues concerning shield tunnelling operation are addressed, while the interaction between ground response and influencing factors during shield advancement is studied, with promising conclusions for the continuance of the present work.

RÉSUMÉ
L'article suivant présente et essaie d'évaluer les rapports d'instrumentation des mouvements de terre et les données provenant de la machine EPB utilisée pour le creusement du tunnel de l'extension sud de la Ligne 2 du Métro d'Athènes. Un nombre considérable de questions concernant l'opération des machines de creusement est examiné, ainsi que la relation entre les mouvements de terre et les facteurs qui influençent les déformations pendant le creusement. L'article présente aussi des conclusions intéressantes pour les études futures.

Keywords : shield tunnelling, Athens Metro, ground movements

## 1 INTRODUCTION

In recent years, during shield tunnelling (usually associated with closed face machines) the produced ground movements have been greatly reduced due to technological innovations, mainly associated with the face pressure and grouting control. As a result, shield tunnelling with closed face machines is gaining in popularity in developed urban areas and geotechnical engineers are facing the challenge of accurate ground movement prediction.

On this complex geotechnical topic, Professor Robert Mair, introducing the discussion session 3-3 for underground works in urban areas during the $XV^{th}$ ICSMGE (2001), underlined that, when using pressurized face TBMs, the modern tunnelling technology is ahead of our understanding of the fundamental mechanics and that, although the applied techniques improve stability, we cannot yet reliably quantify their effects on ground movements. Furthermore, Professor Mair asked whether geotechnical engineers know enough about the mechanical engineering aspects of TBMs to fully evaluate the influence of face and grouting pressure on ground movements. Also, he asked whether it is possible to evaluate their influence when tunnelling in heterogeneous ground conditions.

This paper, based on various field records and monitoring data of ground movements, as well as on data from the EPB machine which is used along the southern extension of Line 2 of Athens Metro from the existing terminal station of Aghios Dimitrios towards Elliniko, aims at the better understanding of the functioning of shield tunnels, as well as the clarification of the interaction between ground response and influencing factors during shield advancement.

## 2 MECHANISM OF GROUND DEFORMATION DURING THE TBM SHIELD PASSING THROUGH

Ground deformation due to shield tunneling can be divided in two parts, one that originates from the stress release at the cutting face and at the tail when the shield is passing through, the other being the subsequent deformation occurring after the shield has passed through. The above situation can be expressed with the following equation:

$$\delta_t = \delta_i + \delta_s \quad (1)$$

where:
$\delta_t$ = total deformation
$\delta_i$ = deformation due to stress release when the shield is passing through
$\delta_s$ = subsequent $\delta_t$ occurring after the shield has passed through

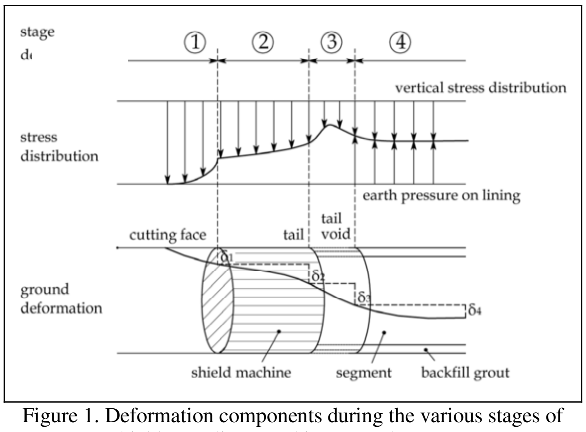


Figure 1. Deformation components during the various stages of shield tunneling (Hashimoto et al. (1999)).

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Taking into consideration the different mechanisms of deformation during the various stages of shield tunnelling, Hashimoto et al. (1999) and Bai et al. (2000), distinguish four deformation components instead of two, dividing $\delta_i$, into 3 parts, $\delta_1$, $\delta_2$ and $\delta_3$, as is shown in Figure 1.

**1st stage, $\delta_1$** Deformation that occurs in front of the cutting face due to the unbalance between earth pressure and shield face pressure.

**2nd stage, $\delta_2$** Deformation that occurs while the body of the shield is passing through, mainly due to the disturbance of the surrounding ground and the friction between the shield body and the ground or due to decrease in the soil modulus.

**3rd stage, $\delta_3$** Deformation that occurs immediately after the shield has passed through, due to stress release by tail void or sometimes due to the excessive back-fill grouting pressure (ground heave).

**4th stage, $\delta_4$** Long term deformation caused by consolidation of the disturbed ground. This last component is a considerable portion of the total deformation, especially in cases of soft clay.

It is remarkable that, according to Bemmebarek et al (2000), one more immediate settlement component is distinguished between $\delta_1$ and $\delta_2$, at the time of passage of the shield face. Furthermore, the $\delta_2$ mechanism is differentiated, when the shield is of conical shape (eg. 3 cm along the length of the shield) which facilitates the control of shield position, reduces soil – shield friction, but, due to the gap in the shield shape, conditions for additional direct movement can be created.

In recent bibliography, numerous shield tunnelling cases were described and interesting field observations were pointed out, providing useful information, technical notes, recommendations, and various meaningful approaches for movement prediction. The following are some indicative conclusions:

- In modern shield tunnelling technology, when the earth balance at the cutting face is carefully controlled, the deformation $\delta_1$ is significantly reduced. However with EPB machines it is often difficult to control face pressures.
- In the case of the second Heinenoord tunnel, where slurry tunnelling was used in soft clays, the proportions of the 3 components of the $\delta_i$ were 15% prior to shield arrival, 35% from the tapered shield and 50% from the tail void. The conclusion, based not only on the experience from the specific tunnel, but from general practice, is that tail grouting influences the magnitude as well as the shape of the settlement trough. Specifically for EPB machines, the initial percentage of 50%, according to *Bai et al. (2000)*, is reduced to 20% – 30%.
- Movement predictions using various computational methods often offer promising approaches, but there remains a constant need to check and calibrate them with field observations, especially at the early stages of construction.
- In general, the shape of the settlement trough under free field conditions can be simulated by the normal distribution curve. It is characteristic that in the case of four subway lines in the Taipei basin (Chang et al. 2000), where the normal distribution curve was fitted by the least square method on the recorded settlement data, the values of the coefficient $R^2$ were as a rule greater than 0.8, with the highest value being 0.96 and the lowest 0.37.
- Ground movements are very sensitive to tunnelling progress. In order to measure this progress, an index was introduced by Chang et al. (2000), namely the Ground Loss Index (GLI), defined as the sum of the division of back fill volume by tail volume and the division of chamber pressure by the in-situ stress at tunnel depth.

## 3 EVALUATION OF GROUND MOVEMENTS

The detailed analysis of tunneling construction using TBM and the evaluation of ground movements require the solution of an intricate soil – structure interaction problem, which may prove a rather exhausting task. This occurs because numerous complex factors are involved, such as ground excavation, overcut or annular space between the external side of the lining segments and excavation side, face support by pressure application, installation of rings and grouting of annular space.

In order to overcome this obstacle, a simplified model was proposed by Oteo & Sagaseta (1982) and applied to actual cases. According to this model, the maximum settlement at the surface above the tunnel axis, $\delta_{max}$, can be expressed as:

$$\delta_{max} = \psi(0.85 - \nu)\gamma\frac{D^2}{E} \tag{2}$$

where:

$D$: tunnel diameter,
$\gamma$: density,
$\nu$: Poissons ratio,
$E$: an average Young's modulus,
$\psi$: an overall factor $\psi \leq 1$. The value of this coefficient according to the previous paragraph, depends on various shield characteristics and operational parameters.

In Figure 2, $\psi$ values scatter; their large range and the influence of shield velocity are shown.

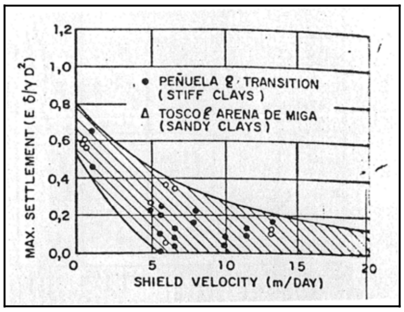



Figure 2. Influence of shield advance rate according to Oteo & Sagaseta (1982).

A similar relationship is included in a recent article on settlements induced by tunneling, presented by the working group "Research" ITA/AITES Report (2006),

$$s_{max} = \kappa\lambda\frac{\gamma R^2}{E} \tag{3}$$

where $\kappa$ and $\lambda$ depend on construction method, ground stresses, tunnel geometry, workmanship and experience.

On the other hand, in many published works ground movements are evaluated using numerical 2D or 3D analyses, based on different approaches simulating tunneling process. In the most common 2D cases, the methodologies described are based either on the use of a stress release factor $\lambda$, ($\lambda$-method) or on the assumption that the support forces are reduced until a particular volume loss value is computed (VL-method). The capability of both approaches depends mostly on the accepted values of $\lambda$ and VL correspondingly, as well as on the used constitutive model for ground behavior, and on the ground modulus and $K_0$ values. The use of 3D analyses, in addition to the increase in computational time, requires the realistic estimation of large uncertainties concerning the interaction between the shield and the soil and the magnitude and distribution of tail void after grouting.

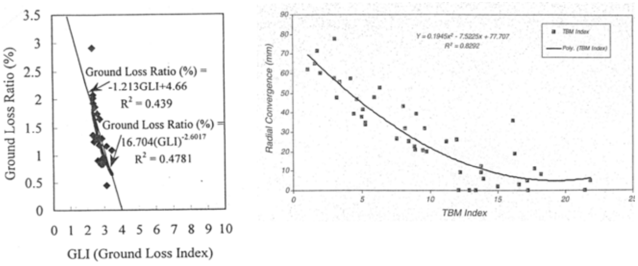
Interesting experimental relationships, between recorded ground movements and influencing factors related to shield tunneling operation, have been recently proposed. Thus, according to Chang et al. (2000) the Ground Loss Ratio is determined with the following formula (see Figure 3(a)):

$$\text{Ground Loss Ratio} = 16.70\ (\text{GLI})^{-2.60} \tag{4}$$

However, results from other work sites, with different geological – geotechnical conditions and other types of boring machines, differ significantly (Emeriault et. al).

For that matter, Chang et al. (2000) clarify that the coefficients in Equation 4 should be carefully adjusted, if the formula is to be applied in other locations than the one proposed for.

Another interesting experimental relationship for the estimate of tunnel radial convergence is presented in Figure 3(b). The index $I_{TBM}$ in this figure is defined as the ratio $TQ^2/Th/ROP$, where $TQ$ is the cutterhead torque (kNm), $Th$ is the thrust (kN) and $ROP$ is the penetration rate (m/h).




(a) (b)

Figure 3. (a) Ground Loss Ratio versus Ground Loss Index according to Chang et al. (2000). (b) The relationship between $I_{TBM}$ and Tunnel Radial Convergence for different areas of the Ghomroud tunnel excavated length, according to Farrock & Rostami (2008).

## 4 EXPERIENCES FROM EPB TUNNELING ALONG THE SOUTHERN EXTENSION OF ATHENS METRO LINE 2

The double track tunnel of the Athens Metro extension from the existing southern terminal station of Aghios Demetrios towards Elliniko (Figure 4), was constructed during 2007 – 2008, by the Joint Venture “Aktor S.A. – Siemens A.G. – Vinci Constructions Grands Projets”. A Herrenknecht EPB machine was used to excavate 4.65 km of tunnel alignment, with the whole project being 5.5 km long. The diameter of the cutterhead was 9.49 m, but with the added use of regulated cutting tools controlled overexcavation was sometimes effected, reaching up to a tunnel diameter of 9.53 m. The tunnel was lined with rings made by 8 precast concrete segments, with an external diameter of 9.18 m and a thickness of 0.35 m. During tunneling, the influencing factors, used in Equation 4 and in the equation of the $I_{TBM}$ index, were monitored with sensors placed in the excavation chamber, such as face pressure, the quantity of grouting used to fill tail voids, tunneling rate, cutterhead torque and cutterhead thrust.

The encountered geology is multilayered along the alignment, as the ground profile along the whole project usually consists of 3 to 5 layers. In total, 17 ground layers are encountered and classified in separate geological formations, the most important being:

- Athens clay schist – metasandstone – metasiltstone
- Marly limestone with variable composition
- Products of weathering and erosion.

It is noticeable that the average Compression Modulus of the main formations varies between 900MPa (marly limestone) and 100MPa (products of weathering).

The following observations, (a) that the average tunneling rate was 10m/day and (b) that the maximum measured ground settlement was 17 mm, while along 80% of the whole alignment the recorded settlements were lower than 5 mm, strongly indicate that the applied EPB tunneling method must be considered successful.

In the present work, in order to study the interaction between settlements, ground properties and tunneling parameters, a particular section of approximately 300 m length was selected. Along this particular section the measured settlements varied significantly, with remarkably high values monitored in some locations.

In Figure 5 the following monitoring and computed data are plotted:

a) The settlements above the tunnel axis (Figure 5(a)).
b) The computed weighted value of an imaginary uniform soil profile modulus, $E_w$ (Figure 5(b)). This modulus is suitable for use in equations (2) and (3). $E_w$ is defined as the unique value of Young’s Modulus for a homogeneous idealized elastic material, which stands for the in-situ multilayer ground profile. The $E_w$ value is estimated using a systematic algorithm based on the following:
    - The Compression Modulus of each layer, determined in the geotechnical investigation and evaluation carried out during the Predesign and Design phase of the Project.
    - The thickness and position of each layer with respect to the tunnel position (Karaoulanis & Tsotsos (2008)).
    - The variability of stress conditions around the tunnel (extension, compression, unloading), as shown in Figure 6.
    - The strain magnitude, which decreases with distance from the excavated area.

The detailed description of the $E_w$ determination process lies beyond the scope and the intentions of the present work.

In the following figures (Figures 5(c)-5(f)) GLI, GLI independent components and $I_{TBM}$ values, as described above, are plotted against chainage for the above mentioned section of 300 m, which is located at approximately the center of the studied alignment

An intensive effort was made, based on the measured data of Figure 5, to examine the possible relationship between settlements and the various tunneling parameters. The conclusion of this work was that, for the specific data, all the considered sets of values seem to be practically unrelated, as the correlation coefficients computed with the aid of smoothing splines and moving averages techniques are low.


Figure 4. Map of Athens Metro, where the southern extension is shown.

## 5. CONCLUSIONS

Reviewing the available experience from the interpretation of the above work, the following observations were formulated:

- In cases where the measured settlements are not significantly higher compared with the observed measurement error, the determination of an experimental relationship must be considered as unfeasible or at least unreliable.

- According to Figure 5, GLI values lied between 1.5 and 2.3 and the range of the volume loss was between 0.05 and 0.4%, while the application of Equation 4 leads to values between 2.0% and 3.0%. This large discrepancy confirms the fact that relationships such as the one presented in Equation 4 are valid only under geotechnical conditions comparable to the ones of the equation data. It is important to notice that, concerning the GLI computation, questions and difficulties arise, as the value of the in-situ stress at tunnel depth and the value of the chamber pressure with respect to time show successive peaks and valleys due to reasons that are not yet determined. Furthermore, the surface displacements do not depend alone on the operational conditions, as is shown in Figure 5, but also on the conditions along a surrounding area of appropriate width.
- According to Figures 5(a) 5(b), as well as the results of the application of Equation 2, the highest measured values of settlement correspond to $\psi$ values between 0.6 and 0.8. This observation indicates that the application of relationships for values of $\psi$ approaching 1.0, forms an upper limit for ground settlements.

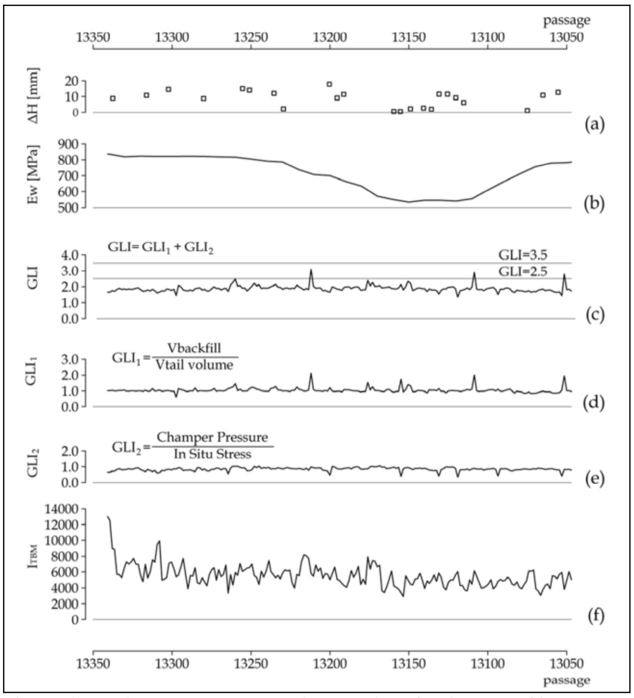



Figure 5. Settlements, $E_w$, GLI and $I_{TBM}$ values for the area of interest.

The combination of the above remarks guides us towards a more general approach, as shown in Figure 7. However, the determination of a more general relationship, requires a more developed database, enriched with data from various works and different ground conditions, as expressed by $E_w$.

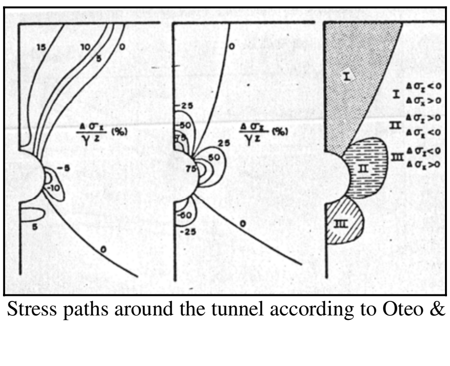


Figure 6. Stress paths around the tunnel according to Oteo & Sagaseta (1982).

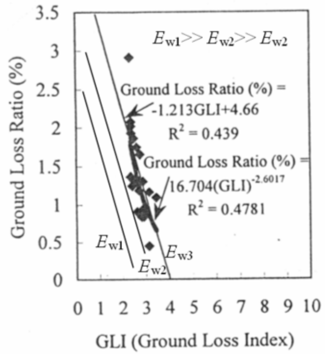



Figure 7. Ground loss ratio versus ground loss index for various $E_w$ values.

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