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Predictive tools for the ground deformation induced by EPB tunnelling: a comparative study

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ABSTRACT The geotechnical design of mechanized tunnelling is extremely demanding due to the complex interrelationship of many determining factors. One of the most challenging and crucial tasks is the prediction of ground deformation, which must be accomplished with high precision as a small subsidence may affect the serviceability or even the stability of buildings and infrastructures. Various tools exist, from empirical closed form relations expressing the ground deformations as function of the soil type and tunnel geometry, to more sophisticated analytical algorithms computing the stress and deformation profiles around the tunnel. In spite of an indubitable practical convenience, these solutions oversimplify or totally neglect the influence of operative factors such as the construction sequence or the pressure applied to the tunnel face. On the other side, numerical models allow to take into account the most important constructive factors, but at the expenses of cumbersome and time consuming calculations. With the aim of checking the predictive reliability of the different tools, the ground settlements computed with empirical or analytical relations are herein compared with the results of a numerical three-dimensional calculations. This analysis is performed with reference to the case of a tunnel driven with EPB shield in Bangkok, thanks to the availability of a set of monitoring data.

1 INTRODUCTION

In general, a tunnelling process consists of a series of operations having the final goal of creating a cavity in the underground space while limiting the disturbance to the surrounding environment. To this aim there are various technologies, broadly classified with the terms “conventional” and “mechanized” tunnelling: the former methodology consists in iteratively performing excavation, mucking and installation of the support systems (e.g. Modoni et al. 2015); the second approach adopts heading machines to carry out a continuous excavation process, articulated in a cyclic sequence of steps.

In densely urbanized areas resting on soft ground, it is customary to build infrastructural lifelines such as metro tunnels, big aqueduct and sewers with tunnel heading machines equipped with shields and rotary cutting wheels. This methodology has become very popular thanks to a continuous development of the technology started in the 19th century, and nowadays allows to operate in particularly difficult conditions such as variable subsoil composition or high hydraulic head.

In a preliminary design stages, the amount of the available information on the subsoil can be rather limited and thus tools for a rough estimate of the stresses and strains induced by tunnelling can be very convenient. Different empirical methods has been developed in order to fulfil these requirements. However, it must not be neglected that these relations lack of a full theoretical understanding of the ongoing processes. More complete solutions, still aimed to a rapid estimation of ground deformations, are the analytical solutions continually developed to include the characteristic of soil (e.g. layered subsoil).

With the increased number and completeness of investigations it is possible to implement more advanced calculation tools, based on numerical analyses. Although more precise and capable of better accounting for the mechanical characteristics of the adopted materials and for the construction sequence, these methods are time consuming and often require a high calculation capability.

In the present work, the predictions of ground deformations with different methods are compared with the idea of checking the capabilities and limitations of various predictive tools for ground deformations. To this aim, the construction of shallow tunnel driven with EPB shield in Bangkok MRTA is simulated using a three dimensional FEM numerical model.

2 REVIEW OF PREDICTIVE TOOLS

The predictive tools developed during years can be summarily grouped into the empirical, analytical and numerical tools.

2.1 Empirical tools

The most frequently used empirical relation in order to get transversal subsidence profile is Gaussian distribution curve, proposed by e.g. Peck (1969). In order to tackle a problem with the matching of settlement profiles with the Gaussian distribution curve (e.g. Branco et al. 1990) and to increase similarity with observations, a modified Gaussian curve introduced by Vorster et al. (2005) can be used. This curve is expressed by the following equation:

$$S_x = S_{\max} \cdot \frac{n}{(n-1) + \exp\left[\alpha\left(\frac{x}{i}\right)^2\right]} \tag{1}$$

$$n = \exp(\alpha)\frac{2\alpha - 1}{2\alpha + 1} + 1$$

where S_{max} is the maximum settlements in transversal direction, x is the distance to tunnel axis and i is the distance to the inflection point. The parameters denoted as n and α are responsible for the shape of the curve.

The longitudinal settlement profile can be predicted using a relation proposed by Attewell & Woodman (1982), derived from observations and expressed in the following empirical form:

$$S_{x,y} = S_{\max} \cdot e^{-\frac{x^2}{2 \cdot i_x^2}} \cdot \int_{-\infty}^{-y} \frac{1}{i_y\sqrt{2\pi}} \cdot e^{-\frac{y^2}{2 \cdot i_y^2}} dy \tag{2}$$

where i_x and i_y are the distances between the projection at ground level of the tunnel axis in the excavation front and the inflection point respectively in the transversal (x axis) and longitudinal (y axis) direction.

2.2 Analytical tools

Differently from the empirical relations, the analytical solutions are based on the solution of equilibrium and compatibility in simplified conditions (e.g. linear elastic medium). Available analytical solutions can be grouped as follows:

- Exact elasticity solutions
- Approximate elasticity solutions
- Plasticity solutions

An exact elastic solution for circular cavity in homogenous medium was firstly presented by Verruijt (1997) using a complex variable method together with bipolar co-ordinates. This solution was successively extended e.g. by Verruijt & Booker (2000) and Verruijt & Strack (2008).

Starting from solution of Verruijt (1997), several approximate elasticity equations were developed. Sagaseta (1987) proposed solution based on virtual image techniques which then was further developed by various authors. Furthermore, Loganathan & Poulos (1998) extended the solution of Verruijt & Booker (1996) in order to get a more realistic description of the ground loss induced in the short-term. This solution is expressed in the following form:

$$u_x = -R^2 x\left\{\frac{1}{x^2+(z-H)^2} + \frac{3-4\nu}{x^2+(z+H)^2} - \frac{4z(z+H)}{\left[x^2+(z+H)^2\right]^2}\right\} \cdot \frac{4Rg+g^2}{4R^2}\exp\left\{-\left[\frac{1.38}{(R+H\cot\beta)^2} + \frac{0.69z^2}{H^2}\right]\right\} \tag{5}$$

$$u_z = R^2\left\{-\frac{z-H}{x^2+(z-H)^2} + \frac{(3-4\nu)(z+H)}{x^2+(z+H)^2} - \frac{2z\left[x^2-(z+H)^2\right]}{\left[x^2+(z+H)^2\right]^2}\right\} \cdot \frac{4Rg+g^2}{4R^2}\exp\left\{-\left[\frac{1.38}{(R+H\cot\beta)^2} + \frac{0.69z^2}{H^2}\right]\right\} \tag{6}$$

where u_x and u_z are respectively horizontal and vertical displacements, H is depth to tunnel centre line, R is tunnel radius, v is Poisson's ratio and β is limit angle which in the case of clayed materials is equal to 45°. Finally, g is a semi-empirical gap parameter, introduced by Lo & Rowe (1982).

Several another approaches including plasticity can be found in the literature, but have omitted in the presented study for brevity.

2.3 *Numerical tools*

The most complete and flexible solution is nowadays offered by numerical tools based alternatively on the finite element method (FEM), boundary element method (BEM), discrete element method (DEM), or on other calculation methodologies. There are several examples of advanced FEM model for mechanized tunnelling (e.g. Kasper & Meschke, 2004).

3 COMPUTATIONAL MODEL

3.1 *Case study*

In the presented analysis, the case study of Bangkok Metropolitan Rapid Transit Authority (MRTA) Blue Line Project (Suwansawat 2002) is chosen for reference. It is localized in the heart of the city, along a roads characterised by heavy traffic.

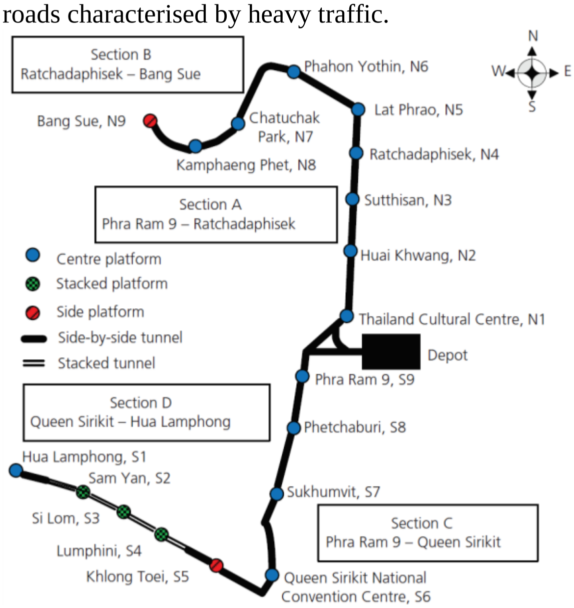

Figure 1. Plan view for MRTA Blue Line Project after Surarak (2010).

Two parallel tunnels were driven with use of EPB shields for a total length of 40 km. The whole project is divided into four sections (Fig. 1). The numerical analysis has been performed in the reference section localized between Bonkai and Sirikit Centre station (~15+200 km) where three strata appear (Table 1).

Table 1. Soil profile for computational section.

Nr	Depth [m]	Name
1	0-15	Bangkok soft clay (BSC)
2	15-25	Stiff clay (SC)
3	25- 60	Dense sand (DS)

In the presented Section D, the tunnel is driven with a 6.46 diameter and 6.19 long EPB shield equipped with 40 thrust cylinders producing a total thrust of 28,300 kN. The lining having an outer diameter equal to 6.3 m and a thickness of 0.3 m consists of six clusters and a key element having 1.2 m length. The tail void gap between the lining and the cavity walls, equal to about 9 cm thickness, was backfilled with grout (Suwansawat 2002).

3.2 *Numerical modelling*

The computational model was built using commercial finite element code ABAQUS^{TM} (Abaqus 2013) and reproduces the three dimensional development of the tunnel construction taking advantage of the geometrical symmetry.

The studied block of soil has dimensions of 60 x 60 x 120 m, with a depth to the tunnel spring line equal to 21 m. It was meshed with 4-node coupled stress/displacement and pore pressure elements with variable dimensions. The shield heading machine was modelled as a perfectly rigid part using first-order 8-node brick elements. The face pressure given by the earth paste was modelled with an external load as hydrostatic distributed pressure. The interaction between shield skin and surrounding soil was simulated using additional contact elements. The final lining was modelled using 8-node stress/displacement brick elements, while for the tail void backfilling, elements coupled with pore pressure were used. Furthermore, the grouting pressure after the passage of the shield was simulated as a hydrostatic distributed load acting peripherally on the surrounding soil on a section of 1.2 m behind the shield. Thrust force generated by jacks was simulated as a pressure acting on the transversal side of the lining elements while the self-weight of the trailer behind the shield was included as concentrated forces.

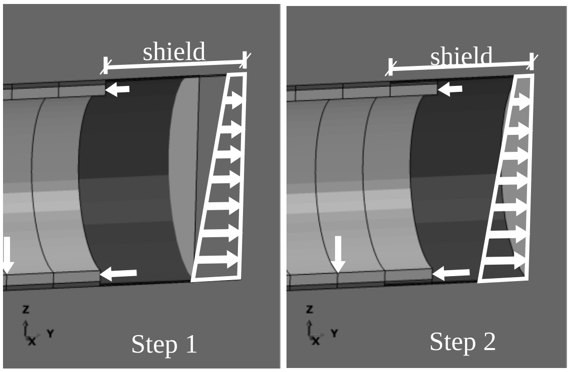

Figure 2. Construction sequence used for the numerical analysis of tunnel heading machine.

After computation of the initial stress state in the soil block using gravity loading procedure (Abaqus 2013), a sequence of two cyclic steps (Figure 2) is used to simulate the tunnel advance. In the former one, translation of shield together with updating position of all loads is carried out. During translation, the position is changed of the hydrostatic pressure at the tunnel face, of the pressure from hydraulic jacks to the last lining elements, of the backup trailer load and of the injection pressure of grout backfilling. In the later step, loads from adjacent soil are transferred on the shield by deactivation of the corresponding soil elements. Thereafter, elements correspond to tunnel lining and grout backfilling are activated. Each of the 335 calculation steps in the presented analysis were performed computing the transient coupled pore fluid flow and stress analysis.

3.3 Constitutive models

The mechanical behaviour of subsoil encountered in the analysed section was modelled with a hypoplastic constitutive model for granular materials (later defined as basic hypoplastic model for sand), and hypoplastic model for clays (later defined as basic hypoplastic model for clay) together with intergranular strain concept (ISC).

The basic hypoplastic model for sand was introduced by von Wolffersdorff (1996) and requires 8 material parameters; the basic hypoplastic model for clay, firstly proposed by Mašin (2005), requires 5 material parameters having similar physical interpretation as the parameters of the modified Cam-clay model. These two models are capable of predicting the behaviour of soil under monotonic loading in the range of medium to large strain. In order to extend the prediction capabilities under cyclic loading and refine them at small strains a mathematical formulation called intergranular strain concept was added (Niemunis & Herle, 2007).

These constitutive models were calibrated based on the extensive laboratory tests which have been reported i.e. by Surarak (2010). Parameters for calibrated constitutive models for Bangkok soft clay (1), stiff clay (2) and dense sand (3) are shown in the Table 2.

Table 2. Parameters of constitutive models used to describe mechanical behaviour of Bangkok subsoil.

Basic hypoplastic model for clay (Mašin 2005)								
Layer	φ_c	N	λ^*	κ^*	R			
	[º]	[-]	[-]	[-]	[-]			
1	27.7	2.1	0.29	0.043	0.2			
2	26	1.5	0.1	0.02	0.2			
Basic hypoplastic model for sand (von Wolffersdorff 1996)								
Layer	φ_c	h_s	n	e_{d0}	e_{c0}	e_{i0}	α	β
	[º]	[GPa]	[-]	[-]	[-]	[-]	[-]	[-]
3	33	1.5	0.28	0.55	0.95	1.05	0.25	1.5
Intergranular strain concept (Niemunis & Herle 2007)								
Layer	A_g	n_g	m_{rat}	R	β_r	X		
	[-]	[-]	[-]	[-]	[-]	[-]		
1	30	1	0.5	1e-4	0.033	0.8		
2	210	1	0.5	1e-4	0.033	0.8		
Layer	m_R	m_T	R	β_r	X			
	[-]	[-]	[-]	[-]	[-]			
3	5	2	1e-4	0.5	6			

The mechanical behaviour of the cemented materials was simulated using simple linear elastic constitutive model. Additionally, the time dependent response of the grout backfilling due to cement hydration was included using simplified relations. For stiffness relation proposed by Weber (1979) was used:

$$E(t) = E_{28} \cdot \mu \cdot e^{\eta / t} \tag{7}$$

where E is value of Young's modulus at time t expressed in days, E_{28} is their value after 28 days, μ and η are dimensionless material constants.

4 INTERPRETATION OF RESULTS

The prediction of ground deformations with the above presented tools is compared with field measurements. The comparison is performed for the transversal and the longitudinal section. In particular, the transversal monitoring section SSS-5T-47A-F has been chosen for reference, while for the longitudinal section, the extensometer RE-T5-001 located in the vicinity of transversal section has been taken. Both monitoring sections were used to record ground deformation for southbound tunnel. It means that there was no influence of northbound as it was built later.

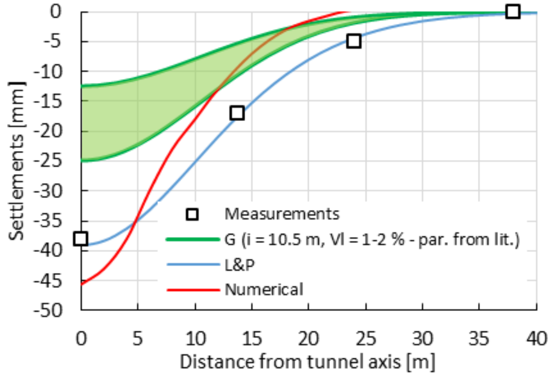

Figure 3. Prediction of transversal subsidence profile from various tools compared with measurements for SSS-5T-47A-F section at the end of construction.

4.1 *Transversal section*

The transversal subsidence profile predicted with one empirical relation and one analytical solution together with results obtained from the numerical analysis are now compared with field measured settlements at the SSS-5T-47A-F section (Suwansawat 2002) as shown in the Figure 3. The empirical Gaussian distribution curve (G) with parameter $i = 10.5$ m (taken from Mair er al. 1993) and S_{max} calculated from the volume loss in the range of 1-2 % (taken from Mair et al. 1996) was used. Whereas, solution presented by Loganathan & Poulos (1998) was used as analytical method. It can be seen that the empirical relation gives significantly lower value of subsidence due to the assumed ground loss. Meanwhile, analytical solution (L&P) gives very good prediction of the subsidence profile, once the values of v and g (see Eq. 6) are put respectively equal to 0.3 and 9 cm. Gap parameter was assumed in simplified manner to be equal to tail void gap. On the other side, the numerical model calibrated with laboratory tests produce a narrower subsidence profile with a slightly higher maximum settlements. A possible reason to explain this difference is the mechanical behaviour of the overlying soil, here described as softer than the real one.

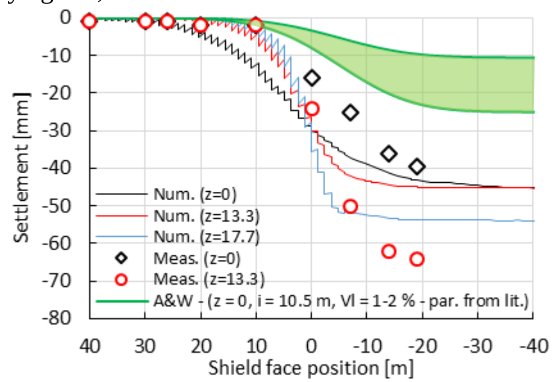

Figure 4. Comparison of longitudinal subsidence profiles during shield passage.

4.2 *Longitudinal section*

The longitudinal subsidence profile induced by tunnelling is compared with the predictions with an empirical method (Attewell & Woodman, 1982) and the numerical analysis. The results computed at the ground surface (z = 0 m) are compared with the measurements from extensometer RT-T5-001 (Suwansawat 2002) (Figure 4). In the empirical prediction method, the distance of the inflection point *i* from the excavation front is assumed to be equal with that for transversal profile (10.5 m). The maximum settlement was obtained using the same volume loss as assumed for the transversal section. It can be seen from the figure that the shape of empirical curve is similar to that recorded on site. However, the maximum subsidence for the whole range of assumed volume loss is significantly lower than measured. The results obtained with the numerical model differ only slightly from the measured ones. At the ground surface, the profile is shifted toward the tunnel face, thus obtaining rather similar shape and maximum subsidence. The subsurface settlements profiles at depth of 13.3 m and 17.7 m (tunnel crown) are steeper and their maximum values are larger than those at the ground surface.

5 CONCLUSIONS

A comparative study is presented between empirical, analytical and numerical methods for the prediction of ground deformations induced by tunnelling. In spite of their simple use, the empirical methods provide inaccurate predictions, both in the transverse and longitudinal sections. Although the analytical method is based on elastic continuum solutions, the accuracy of prediction in the transverse section was satisfactory. The numerical calculation gives a more complex solution, including a wider spectrum of information (e.g. stresses in the lining) and the possibility to more closely simulate the construction process. However, usage of this method is justified only when sufficient information on the subsoil is available. The prediction of settlements with numerical model proved to be quite accurate in terms of absolute values, less in terms of profiles shape.

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