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N. Mekahlia[*1],
orcid.org/0000-0002-5586-1938,
Y. Khadri[2],
orcid.org/0000-0002-9040-7036,
S. Bensehamdi[1],
orcid.org/0000-0002-0704-9777,
A. Benselhoub[3],
orcid.org/0000-0001-5891-2860

1 – Mining, Materials, and Metallurgy Laboratory, National Higher School of Mining and Metallurgy, Annaba, Algeria
2 – Badji Mokhtar University, Annaba, Algeria
3 – Environment, Modeling and Climate Change Division, Environmental Research Center, Annaba, Algeria
[*] Corresponding author e-mail: naoufel.mekahlia@ensmm-annaba.dz

ANALYSIS OF SURFACE SETTLEMENTS INDUCED BY TUNNEL EXCAVATION WITH EPB-TBM

Purpose. To investigate the efficiency of various approaches to predict surface settlements due to tunnel excavation.

Methodology. To appreciate the surface displacements, our study is focalized on the case of a real tunnel in a layered ground (Algiers's Metro), where a tunnel boring machine was driven for the first time in this country. Firstly, the surface settlement trough was calculated with empirical, analytical, and numerical (FEM) methods. Secondly, a set of numerical analyses was carried out to inspect the evolution of surface settlement as the TBM progresses. Finally, a parametric study was performed to examine the construction step most productive for surface settlement.

Findings. FEM is a useful tool for predicting surface displacements due to tunnelling, especially when assigning an adequate and sophisticated behaviour model.

Originality. A reference numerical model which represents well the construction procedures of the Algiers tunnel has been established.

Practical value. This study illustrates that the results obtained by FEM with the use of Hardening Soil as a constitutive model to represent the soil are almost identical to those measured during the tunnel excavation. On the other hand, the empirical formulas available in the literature are not always efficient to predict surface movements.

Keywords: surface settlements, tunnelling boring machine, empirical method, analytical method, finite element method

Introduction. Like all large metropolises in the world, the city of Algiers faces the problem of traffic congestion. This is mainly due to the lack of infrastructure that could accommodate the increasing numbers of vehicles circulating in this capital. Since the surface spaces are insufficient and saturated, an underground-type solution has been provided by the creation of an underground metro system. A large part of this project was carried out with the sequential method known as the New Austrian Tunnelling Method (NATM). Whereas, a Tunnel Boring Machine (TBM) with Earth Pressure Balance System (EPBS) is the excavation method recently applied in this project. Understanding the soil-machine interaction becomes imperative given the growing use of these machines. Ground settlement (surface vertical movement) is a critical threat to both the surface and sub-surface facilities [1], particularly in urban areas where surface deformation is one of the key issues in tunnel construction control, regardless of the construction method. The existing methods used in predicting ground surface settlement induced by tunnelling can be grouped under four categories: 1) empirical methods; 2) numerical modelling; 3) physical modelling; 4) analytical methods [2]. Although the volume of the ground loss around the tunnel lining is a major parameter that has an important effect in estimating ground movements due to tunnelling in the design stage, this parameter is often determined by experience [3]. In the case of shallow tunnelling in the sand, the total volume loss is derived by summing the volume loss tunnelling face, along the shield, and at the tail. Thus, settlement control is still a critical phase of every shallow underground construction activity to ensure both surface and underground safety [4]. Since the enormous development of computer technology, especially numerical tools, it is strongly recommended to perform numerical analyses. Numerical simulations can easily obtain results that are not easily calculated theoretically or observed in model tests. Tunnelling is a three-dimensional problem. This has been well demonstrated by [5] based on the analysis of the distribution of stresses and displacements. However, for its speed and relative simplicity, the two-dimensional modelling approach remains the most widely used in the practice of analysing tunnel projects. The 2D numerical approach simulates approximately the observed movements but requires the use of empirical coefficients to represent the 3D problem [6]. The maximum settlement induced by the TBM excavation occurs at the end tail of the shield which is the place of grouting [7] . The constitutive model has a great influence on describing tunnel behaviour and ground displacement [8]. The proper use of the constitutive model leads to a very good agreement between modelling and measurement. The Mohr-Coulomb model (MC) is commonly used in practice despite its many shortcomings, whereas, by testing the impact of the soil behaviour model on ground movements. [9] showed that the Hardening Soil model (HS) overcomes some of the shortcomings of the MC model. In particular, the use of such a constitutive model produces a more realistic ground settlement profile and leads to a surface settlement trough which better represents the observed data. [10] showed that the surface settlements and their relation to the volume losses measured on the tunnel boundary were significantly different among different models, especially between the models with linear and non-linear stress-strain relations.

This paper aims to use several methods to predict surface settlements, such as empirical equations, analytical solutions, and numerical simulations with different constitutive models to represent soil behaviour to compare the calculated and the measured settlement troughs for a better understanding of the ground movement due to such type of excavation. A sensitivity analysis was carried out to investigate numerically the ground loss influence on surface settlements. The numerical analysis was carried out using the PlaxisV20 software for its ability to simulate the different construction stages, taking into account the static loads applied by the surface traffic and by the machine during excavation.

Surface settlement prediction with empirical and analytical methods. The empirical approach is the simplest yet most reliable way to describe the surface settlement extent. The complexity of surface displacement assessment lies in the heterogeneity of soil layers [11]. Analytical methods are based on data obtained from in situ observation and measurements during tunnel excavation [12]. The short-term transverse settlement trough, taking place after the construction of a tunnel, in many cases can indeed be approximated by a Gaussian curve approach presented in (1), which is a classical and conventional method.

$$S(x)=S_{\max} \cdot e^{\frac{-x^2}{2i^2}}. \quad (1)$$

In this expression $S(x)$ is the theoretical surface settlement at the transverse horizontal distance x from the tunnel centreline. S_{max} is the maximum surface settlement (above the tunnel axis), and i is the horizontal distance from the tunnel centreline to the inflection point of the settlement trough (Fig. 1). It can be estimated from the buried depth of the tunnel axis and the diameter (Z_0, D), using many empirical formulas reported in the literature by [11]. In the case of heterogeneous ground conditions, the simple approach based on the (2) is the most commonly used in practice, where k is an empirical constant that represents the trough width parameter and z is the thickness of each ground layer. By analysing the variability of k based on worldwide case studies, the values of the parameter i were summarized by [13]

$$i_y=\sum_i k_i \cdot z_i. \quad (2)$$

There are several empirical methods to determine the critical parameter S_{max} in (1). [12, 14, 15] recapitulate the most known equations used in maximum surface settlements prediction, relying on multiple geometric and mechanical parameters. However, these relationships are not precise for calculating the aimed values [15]. Surface settlement estimation considers effective parameters and additional factors influencing the maximum surface settlement, especially in EPBS tunnelling. Hence, a finite difference analysis and empirical methods were used for a parametric investigation to identify potentially significant factors affecting the prediction of the maximum surface settlement. Therefore, a new and accurate formula (3) was presented by [15], where γ is the total unit weight, σ_s is the surface surcharge, σ_T is the face support pressure applied at the centre of the tunnel face, C is the cohesion, E is the deformability modulus, υ is the Poisson's ratio, and φ is the internal friction angle of the soil, the weighted averages for all the layers. The results of the suggested equation were verified and compared with the results of empirical and field observations of three different case studies.

$$s_{\max}=3198.744\left(\frac{D}{Z_0}\right)\times \left[\left(\frac{\gamma Z_0+\sigma_s-\left(c+0.3\sigma_T\right)}{E}\right)(1-v)(1-\sin\varphi)\right]^{0.8361}. \quad (3)$$

One of the most efficient analytical methods to calculate surface settlement is the Loganathan & Poulos closed-form solution given in (4), through [12]. By assuming an oval-shaped ground deformation pattern around the tunnel section presented by a radial contraction ε, which is defined as equal to $(4gR+g^2)/4R^2$, the gap parameter g is defined as the maximum settlement at the tunnel crown. The physical *gap* $G_p = 2\Delta + \xi$ is usually the difference between the outer diameter of the shield and the liner (Δ is the thickness of the tailpiece, and ξ is the clearance for the erection of lining), U^*_{3D} is $3D$ elastoplastic deformation into the tunnel face, and w is workmanship.

$$g=G_p+U^*_{3D}+w. \quad (4)$$

Then the surface settlement trough can be obtained by using (5)

$$S=4(1-v)\varepsilon R^2 \exp\left[\frac{-1.38x^2}{(H+R)^2}\right]. \quad (5)$$

Recently, based on lab-scale model test results, [2] proposed an empirical formula for estimating the subsurface settlement caused by tunnelling in the sand. The effectiveness of this method was validated by [16] using observational data from three case studies

$$S=\frac{0.5V_L}{\sqrt{2\pi k z_0}} \cdot e^{\left(-\left(\frac{x}{\sqrt{2\pi k z_0}}\right)^2\right)}. \quad (6)$$

Equation (6) represents the proposed prediction solution of the surface settlement trough. While V_L is the volume of the ground loss around the tunnel section (per meter length), many empirical formulas have been proposed to estimate the volume loss. In the case of a Z_0/D ratio from 0.4 to 1, a volume loss in shallow tunnelling of less than 0.5 % can be achieved with the condition of careful monitoring. The highest expected volume loss in this range of the Z_0/D ratio is about 3.7 % for tunnelling in the sand [3].

Case study description. The tunnel section taken in this study is part of the extension B1-line 01 of the Algiers metro running between El Harrach and the international airport, which extends over a length of 9,560 km, including 9 stations. The construction was carried out in the Mitidja plain made up of Tertiary land filled in by Quaternary, the study area consists mainly of recent alluvial deposits.

The excavated diameter is 10.5 m at a depth of 12.75 m to the tunnel axis. The subsoil consists of filled soil up to 3 m deep below ground level, underlain by two major formations of sand and clay up to 16 m and beyond which the marly clay is located up to the exploration depth (Fig. 2), the level of the ground water table is 16 m. The EPB-TBM machines balance

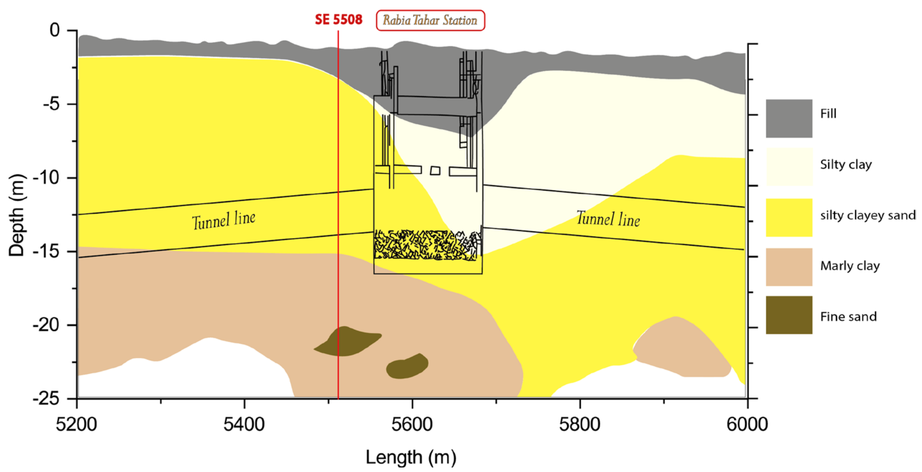

Fig. 1. Geological section of tunnel alignment

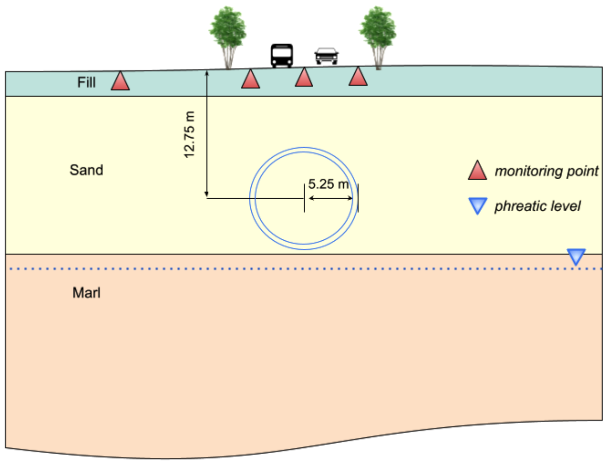

Fig. 2. Tunnel transversal cross-section SE5508

the hydrostatic and the front pressures by confining the material that fell in the cutting chamber (Fig. 3). The face supporting pressure must be as enough to withstand the soil weight and to avoid collapses and ground repression at the tunnel face [17]. The surrounding soil relaxation occurs as the TBM advances and this is due to the conical shape of the shield. The grout is applied just after the exit of the shield and before the installation of the final lining. The control parameters of the TBM during digging (including face pressure, grout injection pressure, etc.) are measured by sensors and inspected by machine pilots to ensure proper operation and prevent blockages or material breakage. Geotechnical surveys and laboratory tests were conducted at the start of the pre-project study by the company Ferconsult CENOR according to Euro code standards. Settlement monitoring was carried out regularly during construction with a geodetic control system installed by Vbss GmbH company. The effect of tunnelling progress on the variation in surface vertical displacement is shown in (Fig. 4), the monitoring point located at the tunnel centreline. The mechanical parameters of the shield and the final lining are listed in Table 1.

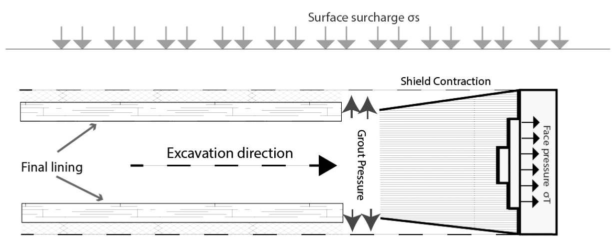

Fig. 3. TBM with earth pressure balance system

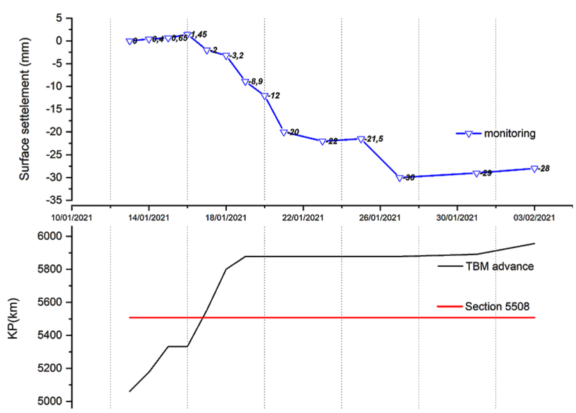

Fig. 4. Site monitoring data at tunnel centreline

Table 1

TBM and lining characteristics

	Unity	EPB-TBM (isotropic)	Lining (Isotropic)
Normal stiffness (EA)	KN/m	$7.6 \cdot 10^7$	$1.4 \cdot 10^7$
Flexural rigidity (EI)	KN/m	$1.01 \cdot 10^6$	$1.43 \cdot 10^5$
Thickness (d)	m	0.39	0.35
Weight (w)	KN/m/m	14	8.4
Poisson's ratio (ν)	–	0.2	0.15

Numerical analysis. The FEM has eventually become the most efficient numerical method given its large field of application. It is extremely powerful since it allows studying correctly continuous structures with complicated geometric properties and loading conditions. One of the main advantages of the 2D finite element calculation method applied to tunnels is to be able to perform and analyse distinctly the many construction stages of tunnelling with a tunnelling boring machine.

This section describes a cross-sectional 2D analysis using the finite element software Plaxis.2DV20 Connect edition, provided by Bentley Systems. The dimensions of the model were considered as 60 × 60 m, this dimension exceeds laterally 5D, so the boundary effects can be minimized. The finite element mesh is shown in Fig. 5. The generated plan strain model consists of 2,194 elements and 18,141 nodes using 15-Noded triangular elements. The lateral movements were restrained on the sides, and both horizontal and vertical displacements were restricted at the bottom.

To investigate the model's capability to produce the surface settlement trough, the shield and the final lining are modelled with an elastic behaviour, while the soil was analysed using two constitutive models: MC – linear elastic perfectly plastic with a Mohr-Coulomb failure criterion and HS – Hardening Soil; an advanced constitutive model with the same failure parameters of MC but following a nonlinear stress-strain relation instead of a bilinear curve, using an unloading/reloading stiffness. Regardless of the sophistication of the constitutive soil models, there is a considerable need for further research by employing more complex soil behaviours like the HS-small strain model implanted in the calculation program used in this study (Plaxis, 2020). The different soil parameters along the study area are correlated with respect to Euro code standards, obtained from reports of the Consider-Group company of Algeria. The HS model input parameters were completed with the help of the empirical correlations from [18]. Geotechnical specifications used for the different soil layers are listed in Table 2. For the fill and the sand, both MC and

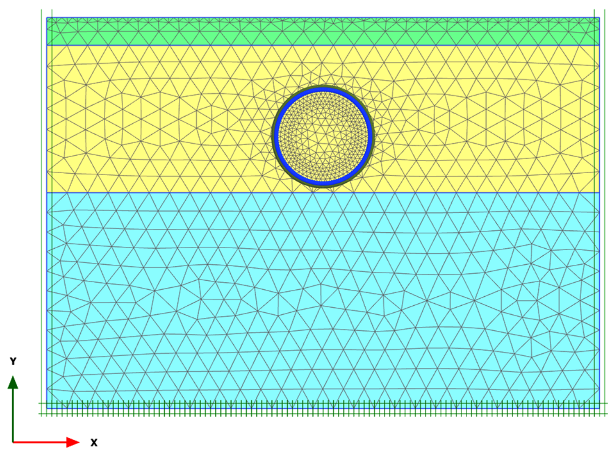

Fig. 5. Generated mesh

Table 2

Geotechnical parameters adopted for the soil layers

		Unity	Fill (Drained)	Sand (Drained)	Marl (Undrained)
MC	γ_{unsat}	KN/m^3	17	18	17
	γ_{sat}	KN/m^3	20	21	–
	E'	MPa	10	70	–
	ν'	–	0.3	0.3	–
	φ'	°	20	34	–
	c'	KPa	0	10	–
	E_u	MPa	–	–	112
	ν_u	–	–	–	0.495
	S_u	KPa	–	–	156
HSM	γ_{unsat}	KN/m^3	17	18	17
	γ_{sat}	KN/m^3	20	21	21
	c'	KPa	0	10	–
	E_{oed}	MPa	10	70	97
	E_{ur}	MPa	30	210	291
	ν_{ur}	–	0.2	0.2	0.2
	φ'	°	20	34	–
	Ψ	°	0	4	–

HSM models use drained material behaviour in which stiffness and strength are defined in terms of effective properties. While the marl is defined by the undrained material behaviour, in which stiffness and strength are defined in terms of undrained properties, and excess pore pressures are included in the effective stresses.

In Plaxis 2D the volume loss around the tunnel can be implanted as a contraction line ratio which corresponds to the relative reduction between the excavated section and the final section of the TBM. Due to the special shape of the shield, the surrounding soil tends to expand radially until it comes into contact with the shield, which is infinitely stiff so the expansion of the terrain is blocked.

The simulation sequence adopted in this study is adequate to follow during the construction of the section in the study. The whole tunnelling process has been performed using the stage construction tool of the software through six calculation phases as follows: 1) ground initial stress generation (K_0 procedure); 2) activation of the surface weight and traffic load $\sigma_s =$ $= 10$ KPa; displacements produced during the previous steps are disregarded; 3) the tunnel cluster is excavated and confining pressure of 70 KPa is applied as a tunnel face pressure; 4) activation of TBM plate elements (knowing the total mass of the shield makes it possible to assign a particular unit weight to this metallic structure while preserving the characteristics of inertia and rigidity of a non-deformable metallic cylinder); 5) assigning a line contraction of 0.5 %, which represents the volume loss due to the conical shape of the TBM; 6) application of sprayed concrete as a radial pressure [7] $\sigma_c = 120$ KPa; and 7) activation of plate elements which represent the final lining.

Following the sequence described above, a set of FEM calculations using the HS model with the same layout were carried out to investigate the influence of the contraction ratio set up in the model.

Results and discussion. The maximum displacements produced in numerical simulation occurred at the tunnel crown, are 22.64 and 37.9 mm for MC and HS models respectively. The total displacements distribution at the final lining stage is shown in Fig. 6. Calculated maximum surface settlement from

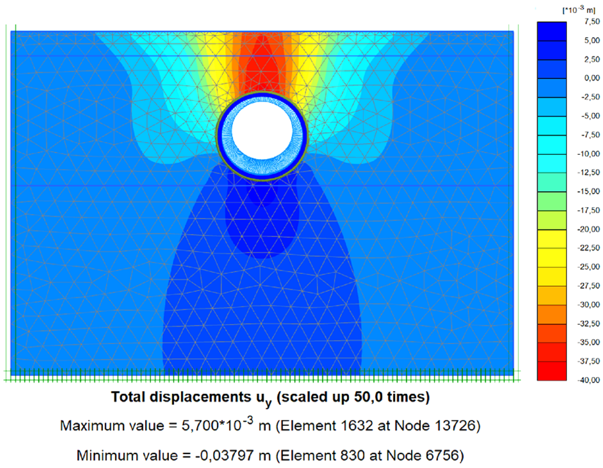

Fig. 6. The model total displacements at the final stage

the empirical, analytical and numerical methods are compared with field observation data in Table 3.

The calculation results showed the efficiency of the Loganathan & Poulos analytical method to predict better maximum surface settlements (99 % of accuracy in this case study), while the recent method proposed by [2] gave inaccurate results. Indeed, the equation developed is more oriented towards the prediction of subsurface settlements by considering the variation of the volume of settlement trough with depth.

Hence, the transverse surface settlement (Gaussian troughs) calculated through each method is compared to field measurements (Fig. 8). The [2] method and the numerical simulation with the Mohr-coulomb model show low values of surface settlement at the tunnel centreline whilst a ground heave appears 17 metres away from the tunnel axis, this can be

Table 3

Comparison of maximum surface settlement at tunnel centreline

	Observed	Mohr-Coulomb	Hardening Soil	Loganathan & Poulos (1998)	Wang et al (2016)	Chakeri (2014)
S_{max} (mm)	29	21	34	28.76	17.77	35.59
Deviation of actual value, %	–	28	17	1	39	23

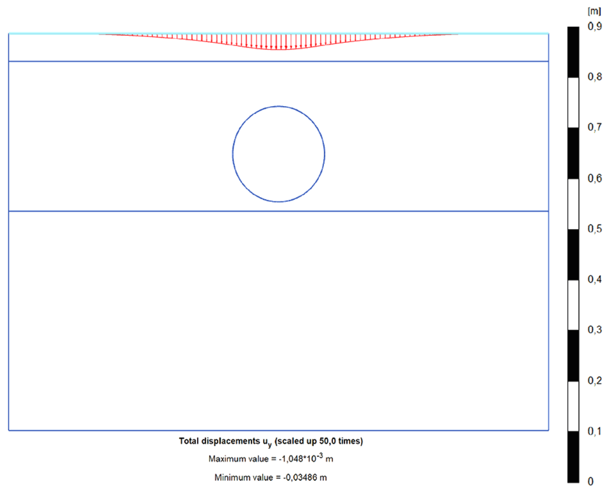

Fig. 7. Surface settlement trough at the final stage (HSM)

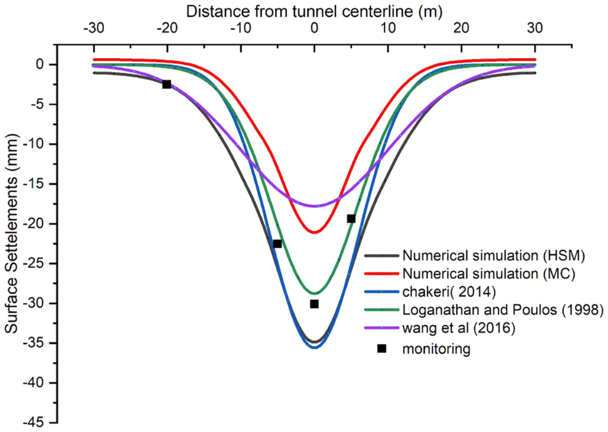

Fig. 8. Compared surface settlement troughs

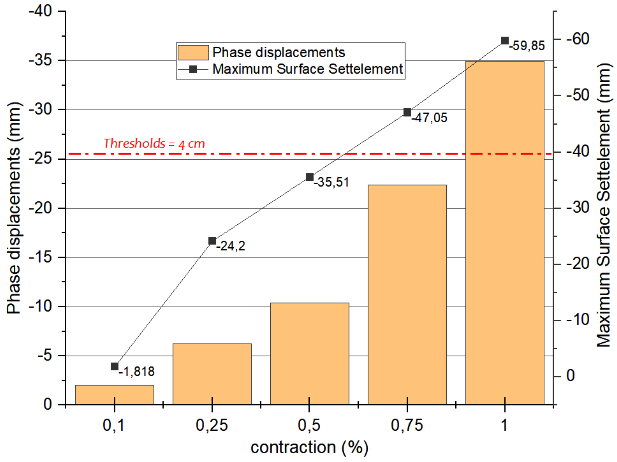

Fig. 10. Contraction ratio influence on surface settlements

explained by the fact that the stiffness is constant which lead to a trough that fails to coincide with the real ground response to such type of excavation. Surprisingly, the Loganathan & Poulos analytical solution produces a shape that coincides with reality nearby the tunnel centreline. Noted that at 20 m from the centreline of the tunnel, the [2] method and the HS model give errorless values. Overall, the [15], Loganathan & Poulos solutions, and the numerical simulation using the HS model give slightly narrower shapes of surface settlement troughs. As described before, every single stage produces its displacements. As shown in Fig. 9, it is the contraction phase that favours the emergence of displacements among all processes by 41 % for the HS model, and this is due to the volume loss around the conic shape of the shield. On the other hand, the grouting phase minimizes the settlements by 7.6 and 1.5 mm for MC and HS models respectively, which is the main role of this tunnelling step in reality. While a negligible quantity of settlement occurs at the final lining installation. The surface settlement evolution at the tunnel centreline during the numerical simulation stages is in good agreement with the topographic monitoring, which demonstrates the reliability of the established numerical model.

According to the literature [7], the contraction ratio affects significantly the surface settlement, so the determination of the real ratio is so important. Fig. 10 shows the variation of settlement produced according to different amounts of contraction, note that the phase displacement is defined as the settlements engendered just during the construction stage. The results show that introducing low contraction ratio values of 0.25 and 0.1 % into the model reduces the phase surface settlement by 39 and 80 % respectively. However, applying a high contraction ratio (around 1 %) leads to settlements that greatly exceed the thresholds (S_{max} = 06 cm). The results indicate that the FE model predicts well the maximum surface settlements for 0.5 % of contraction, which represent well the real diameter reduction of the shield in this case study. The wide range of variation in settlement values with change in this parameter makes sense that the currently preferred technique of imposing contractions on the tunnel location does not simulate all of the complexities associated with actual excavation procedures.

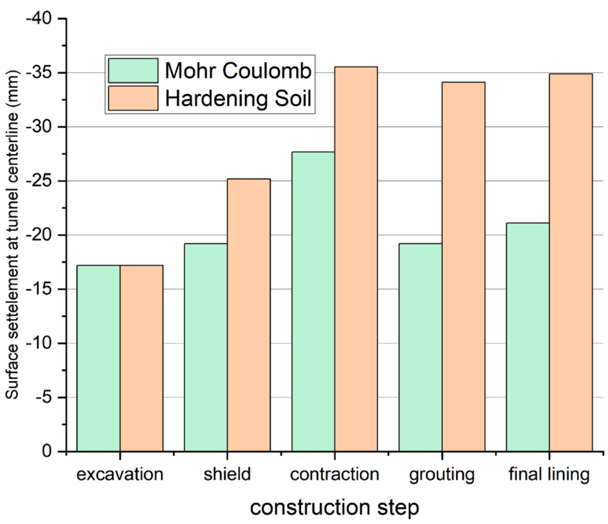

Fig. 9. Maximum surface settlement evolution during calculation

Conclusion. This work provided a surface settlement prediction caused by tunnelling with the EPB system. Various empirical and analytical methods available in the literature were adopted in this case study. The comparison of the calculated settlement troughs with monitoring data showed that the choice of analytical and empirical methods must be appropriate. Whereas, the finite element simulation with the use of a sophisticated constitutive model such as the Hardening Soil Model gives more detailed and realistic results. The numerical models were established assuming a step-by-step simulation to take into account as much as possible the tunnel sequencing from the face excavation to the installation of the final support. The maximum settlement induced by the machine excavation occurred just after the passage of the shield; however, these values are reduced in the next step where the grouting is applied. Thus, the study highlighted the importance of determining the shield contraction ratio, given its significant effect on ground displacements. This article has shown how surface settlements calculated at the same site can vary with the choice of soil and model parameters. The numerical results reveal the efficiency of the finite element method in predicting ground movements due to underground constructions.

Recommendation. Experimental and numerical studies should be performed to investigate tunnel behaviour under dynamic loads.

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Аналіз поверхневих просадок при проходженні тунелю з використанням тунелепрохідницьких машин із ґрунтопривантаженням

Н. Мекахлія[*1], *Ю. Харді*[2], *С. Бенсехамді*[1], *А. Бенселгуб*[3]

1 – Лабораторія гірничої справи, матеріалів і металургії, Національний гірничо-металургійний університет, м. Аннаба, Алжир
2 – Університет Баджи Мохтар, м. Аннаба, Алжир
3 – Відділ навколишнього середовища, моделювання та зміни клімату, Науково-дослідний центр з охорони довкілля, м. Аннаба, Алжир
[*] Автор-кореспондент e-mail: naoufel.mekahlia@ensmm-annaba.dz

Мета. Вивчення ефективності різних підходів до прогнозування просадок поверхні, викликаних проходкою тунелю.

Методика. Щоб оцінити поверхневі зміщення, наше дослідження проводиться в умовах реального тунелю в шаруватому ґрунті (Алжирське метро), де вперше в цій країні було запущено тунелепрохідницьку машину (ТПМ). По-перше, поверхнева мульда просідання була розрахована за допомогою емпіричних, аналітичних і чисельних (FEM) методів. По-друге, було проведено ряд чисельних аналізів із метою вивчення розвитку просідання поверхні у міру просування ТПМ. Нарешті, було проведене параметричне дослідження для вивчення етапу будівництва, найбільш продуктивного для поверхневого просідання.

Результати. FEM є корисним інструментом для прогнозування зміщення поверхні, викликаного прокладанням тунелів, особливо при визначенні адекватної та продуманої моделі поведінки об'єкта.

Наукова новизна. Була створена еталонна чисельна модель, що добре відображає процес будівництва тунелів в Алжирі.

Практична значимість. Дане дослідження показує, що результати, отримані за допомогою FEM із застосуванням схеми зміцнення ґрунту в якості структурної моделі для представлення ґрунту, майже ідентичні результатам, отриманим під час проходки тунелю. З іншого боку, емпіричні формули, що є в літературі, не завжди ефективні для прогнозування поверхневих переміщень.

Ключові слова: *поверхневі просідання, тунелепрохідницька машина, емпіричний метод, аналітичний метод, метод кінцевих елементів*

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