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S. Berdoudi[*1],
orcid.org/0000-0002-3612-6823,
M. A. R. Morsli[1],
orcid.org/0000-0001-5242-986X,
Z. Mekti[1],
orcid.org/0000-0002-6153-7026,
A. Benselhoub[2],
orcid.org/0000-0001-5891-2860

1 – Lavamine laboratory Badji Mokhtar University, Annaba, Algeria
2 – Environment, Modeling and Climate Change Division, Environmental Research Center (C.R.E), Annaba, Algeria
[*] Corresponding author e-mail: berdoudisaid@yahoo.fr

NUMERICAL STUDY ON DEFORMATION AROUND UNDERGROUND MINING STRUCTURES (ALGERIA)

Purpose. To study the stability of underground mining structures using numerical method based on finite elements, two-dimensional (2D), Finite Element (FE) modeling using GEO5 calculation model.

Methodology. To consider the influence of geotechnical parameters, the tunneling is carried out by the NATM method. In order to check settlements in soft ground and to carry out the work in complete safety, we used software based on the finite element method.

Findings. Determination of the range and prediction of subsoil displacements are necessary when designing this type of structure due to the need to ensure the safety of the active environment located in the zone of influence.

Originality. The originality of this work is the characterization of the soil of the studied region, determining of the different physical and mechanical properties as well as the modeling with a recent calculation model based on the Mohr-Coulomb behavior mode.

Practical value. Given the characteristics of the current section, this study illustrates that the results obtained using the GEO5 calculation code, show the movement exceeding the permitted threshold; its values are 47.80 and 46.6 mm respectively in the first and second step, which can induce significant ground movements. As a solution, there are possibilities of reducing the current declivity within the limits authorized for this type of line (maximum = 40 ‰) in order to increase the cover (height of earth on the key) of the tunnel and consequently reduce surface settlements.

Keywords: *soil displacement, tunnel, numerical modeling, settlement, underground structure, finite element method, GEO5*

Introduction. The problems caused by geological and geotechnical instabilities during the digging of tunnels in a mine cause a modification of the state of stress and deformation of the ground, thus causing inadmissible disorders in the environment, and present a real danger to public safety. This needs an extensive and detailed research to determine the degree of its risks and mitigate its effect on the more or less vulnerable human, economic, cultural and environmental issues.

Several procedures have been developed and proposed to predict the deformation around underground mining structures, Qin. Y (2021) [1], Longhui Guo (2020) [2] and Svoboda, J. S (2014) [3], used a combination of various methods to study the characteristics of mining structures. For Elmanan (2016), different analytical and numerical methods [4] can be used to define the failure limits. They studied the pressure around the tunnels using different methods and hence it was found that the numerical methods are more accurate calculation techniques than the analytical methods, and in this sense the theory of simulations has been approved by Ahmed, S. N. A., et al. (2019) when it comes to three-dimensional models in heterogeneous formation grounds [5]. Elsamny, et al. (2016), studied the factors affecting the stress distribution around two circular tunnels and the internal forces [6]. They found that the modulus of elasticity, Poisson's ratio for clay soil, are some of the factors that affect stress distribution and surface settlement.

Tunnel excavation inevitably causes stress changes in the surrounding soil, which may induce [7] significant ground movement. The research work that we present is focused on the study of the effect of geotechnical parameters on modeling during the digging of a mine tunnel located in the capital of Algeria, Ain Naadja-Baraki line, and notes the displacements that manifest above the arch of the structure. For Hanna Michalak and Paweł Przybysz (2021), displacement of the subsoil depends [8] on a variety of factors, including, primarily, change in the state of stress resulting from the relief and load on the subsoil, the subsoil type, its strength and deformation parameters. The calculation code used for our modeling is GEO5.

The code solves problems of deformation and stability of geotechnical and tunnel excavation works using the mathematical finite element method from the elastoplastic behavioral equations of soils or rocks; anisotropic materials often show different [9] mechanical properties in each direction, The method makes it possible to accurately take into account the actual geometry of the structure, the heterogeneity and anisotropy of the terrain and the stresses [10].This method is very effective when it comes to studying the distribution of stresses in the vicinity of an excavation in unfractured ground

Materials and Methods. ***General characteristics of the studied area.*** The present research covers the section line of the Algiers Metro, corresponding to the *2nd* phase, has a total length of 3,845 meters (Fig. 1).

Table 1 below summarizes the different geotechnical units encountered during the exploration study.

Geotechnical characterization. According to the ground profile obtained during core drilling, the tunnel section will be excavated mainly in the geotechnical unity QA. At the end of

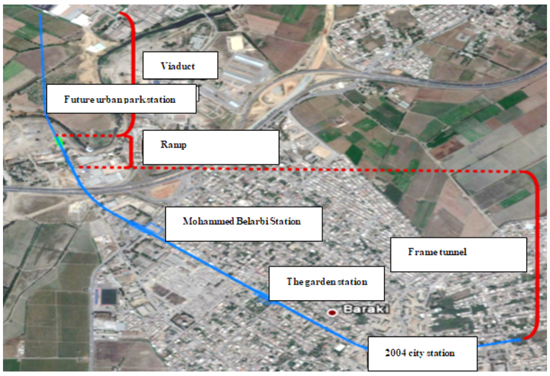

Fig. 1. Section of the Algiers Metro phase 2

Table 1

The different geotechnical units

Geological-geotechnical unity	
Compacted construction deposits	R
Muddy clays	AA
Very steep silty clay	QA
Compact to very dense clay sand	QS
Very dense clay gravels	QG
Hard carbonate marl clay	QM

the section, the excavation of the raft may affect the materials of the QM geotechnical unity.

Fig. 2 shows the nature of the materials encountered during the boring work.

These materials (Fig. 2) constitute yellow clays and silty clays sometimes sandy with kaolinite areas. From results of the geotechnical campaign, the following values can be established for the main geotechnical parameters.

Classification and characteristics:

1. Particle size: The percentage in fraction of diameter less than 2 mm has a value of 95 % and the percentage in fraction of diameter less than 0.08 m is 91 %.
2. Atterberg Limits: The average value of the liquidity limit is 44 %. The plasticity index has an average value of 21 %.
3. Dry density: ρd, is 18.5 KN/m^3.
4. Natural water content: Wn, is 16.7 %.
5. Carbonate content: varied between 4.3 and 41.6 % and an average value of 25 %.

According to the Unified Soil Classification System [11], these materials are mainly classified as silty clays and clayey silt, weakly marly to marly.

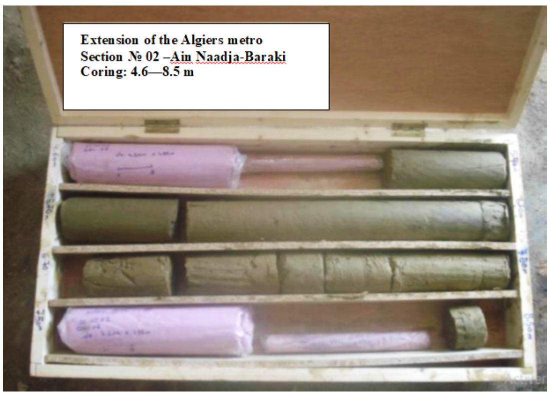

Fig. 2. Core sample shows the structure of constituents

Strength. The limit pressure varies between 0.4–2.2 MPa, (Fig. 3), with an average value of 1.4 MPa. The pressuremeter modulus varies between 2.3 and 61 MPa, with an average value of 24.2 MPa. The increase in the value of the limit pressure with depth is greater in the limit pressure than in the pressuremeter module.

From Fig. 3, an average value of 0.8 MPa is adopted as long as the work is currently carried out at a depth of 9 m.

Fig. 4 shows the variation of the pressuremeter modulus with depth. For a clay with a net limiting pressure of 1.4 and σh of 0.15 MPa (horizontal stress), it is taken as an undrained cohesive value at 150 KPa.

The in situ identification shows that the values of Standard penetration test (S.P.T), (Value of N30) which varies between 7 and 41, with an average value of 22, can be noted in general with the degree of consistency of these materials, (taking into account the limitations of SPT penetration tests) as cohesive materials.

The consistency of these materials varies between stiff – hard, with a medium state of very stiff consistency. Fig. 5 shows the variation of the values of N30 depending on the depth.

Table 2 illustrates the relationship between material consistency and undrained shear strength. These materials have a very stiff consistency [12] and an undrained cohesion Cu = 146 KPa. Taking into account the variability of the data and the characteristics of the material, a value of 150 KPa is adopted.

From the test results, the internal friction angle (Φ), varies between 8–26° and the cohesion value (C) varies between 4–80 KPa. These values are not considered representative for these materials.

If we jointly indicate the breaking points of the direct shear tests carried out by the regression line, it allows us to adjust the values of C = 45 KPa and Φ = 20°.

Fig. 6 presents the results of the direct shear tests.

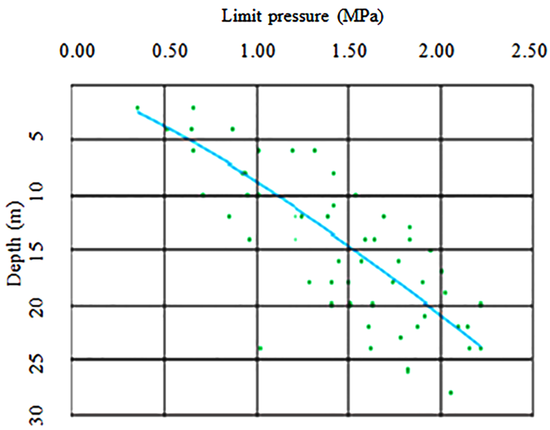

Fig. 3. Variation of the limit pressure (PL) with depth

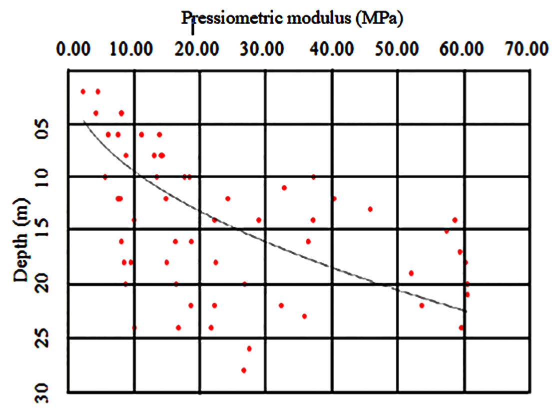

Fig. 4. Variation of pressiometric modulus (EP) with depth

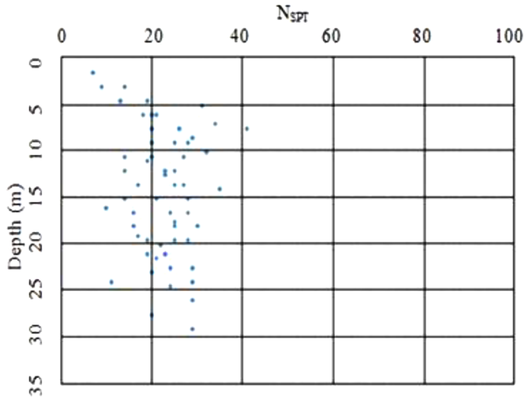

Fig. 5. Variation of NSPT with depth

Table 2

Relationship between material consistency and undrained shear strength

N_{spt}	Consistency	Site identification	Shear strength (KPa)
0–2	Very mole	Soil easily penetrated for a few cm by the fist	<12
2–4	Mole	Soil easily penetrated for a few cm by the thumb	12–25
4–8	closed	Soil penetrated with several thumbs with moderate effort	25–50
8–15	stiff	Soil marked easily with the thumb, but penetrated with a lot of effort	50–100
15–30	very stiff	Soil easily scratched by thumbnail	100–200
>30	hard	Soil striped with difficulty using a thumbnail	>200

According to the study developed by Z. Ouyang, P. Mayne (2018), from triaxial tests [13], we get the value of the drained internal friction angle in relation to the plasticity index. For a plasticity index between 15 and 20 %, an internal friction angle value between 20.2 and 45.1° is obtained.

For the resistance parameters in drained conditions, the following values can be adopted:

1. Angle of internal friction, $\Phi' = 25°$.
2. Cohesion, $c = 40$ KPa.

Deformability Modulus. The undrained cohesion is inversely proportional to the plasticity index. Taking into account the plasticity index of the soil, for an interval of plasticity between 15 and 30 % a drained deformation modulus of 37 MPa is obtained for Cu = 150 kPa.

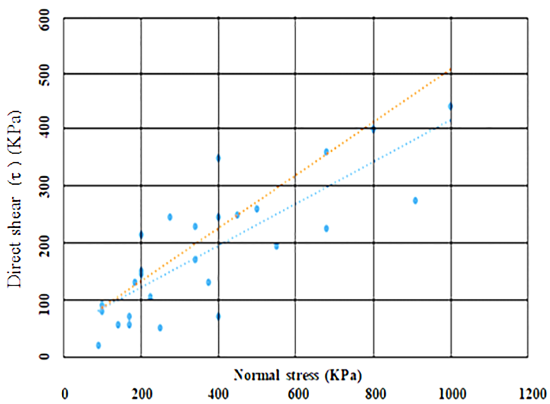

Fig. 6. Results of direct shear tests

Table 3 below indicates the relationship between the undrained cohesion and the plasticity index

According to Üzeler, Volkan (2013), the pressuremeter modulus is commonly used in geotechnical practice for foundation designs, because in many cases, the soil or rock shows elastic behavior before the failure conditions. From the pressuremeter modulus and according to relation (1) below for clays with $\alpha = 2/3$ (rheological coefficient) and a pressuremeter modulus value of 20 MPa, an oedometric modulus of a 36 MPa is considered.

$$Em = EM/\alpha, \quad (1)$$

where EM is a pressuremeter module; α is a correction factor; Em is a pressuremeter module after correction.

For an oedometric modulus with a value of 36 MPa and for $\nu = 0.3$, (Poisson's ratio) a drained modulus value of 26 MPa is considered.

$$E = Eoed \cdot \frac{(1+\nu)\cdot(1-2\nu)}{(1-\nu)}. \quad (2)$$

For Burt Louk, et al. (2007), the elasticity modulus for the very stiff clays at long-term varies between 15 and 35 Mpa [14]. Taking into account the variability of the data and the characteristics of the material, for the drained strain modulus a representative value of 35 MPa is adopted.

Permeability. Nine (9) Lefranc permeability tests were carried out at different depths of the geotechnical unity QA. The permeability values obtained vary between $1.5 \cdot 10^{-9}$ and $1.2 \cdot 10^{-6}$ m/s. The average value is about $k = 3.6 \cdot 10^{-7}$ m/s.

Basing on the particle size characteristics of the material, the permeability (k) of these materials is very low and can be considered as practically impermeable. The tests belonging to this unit are grouped together in Tables 4 and 5.

Analysis methods. For the study and prediction of the behavior of an underground geotechnical structure, there are several possible methods, namely: empirical or semi-empirical methods, analytical methods and numerical methods. Numerical models based on finite elements, finite differences or even distinct elements have the advantage of being able to address analytically insoluble theoretical problems.

Numerical study. The numerical methods are a powerful tool for solving many engineering problems in order to check the stability of the walls during the excavation phase in loose soil and to carry out the work in complete safety.

Serratrice (2004) carried out 2D calculations in plane strains in the case of a circular tunnel, dug at a shallow depth in an elastoplastic material of the Mohr-Coulomb type. Based on the numerical results [15], he proposes a formulation for the estimation of the final settlement ($s0$). In this case the deconfinement rate λ is taken as a datum and the settlement mainly depends on the triplet E, c and λ.

In our case, we used software based on the finite element. The GEO5 two-dimensional modelling model (Bently Geostructural Analysis), is one of the most widely used software today to solve geotechnical stress-strain problems in a continuous medium. At any point of the massif, the stress-strain tensors are known, which makes it possible to visualize the phenomena. Among these problems we have ground settlements

The essential geotechnical data used for this model are summarized in Table 5.

Table 3

Undrained cohesion depending to the plasticity index

Plasticity index	E/Cu
10–20	270
20–30	200
30–40	150
40–50	130
50–60	110

Table 4

Test results for the geotechnical unity (QA)

Geotechnical unity QA	k (m/s)	Pressuremeter modulus	Limit pressure	Creep pressure	SPT (N)
Number of tests	9	58	58	58	58
Maximum	1.20E–06	60.62	2.21	41	41
Minimum	1.47E–09	2.32	0.36	7	7
Medium	3.64E–07	24.21	1.43	22	22

Table 5

Test results in the laboratory of the geotechnical unity QA

Geotechnical unity QA		Number of tests	Maximum	Minimum	Midium
Dry density (KN/m^3)		11	19.20	16.80	18.50
Water density (KN/m^3)		20	22.10	20.30	21.30
Gs (g/cm^3) Solid Particle density		15	2.70	2.57	2.65
Water content (%)		16	22.50	5.10	16.66
Grain size(mm)	20	12	100	100	100
	10	16	100	100	100
	5	16	100	92	98
	2	16	100	86	95
	0.4	16	100	82	92
	0.08	16	100	77	91
Atterberg Limits	WL	16	57.9	38	44
	WP	16	27.6	19.7	23.60
	IP	16	30.3	15.40	20.70
Direct shear (type CD)	φ°	9	26.57	8.62	17.07
	C (t/m^2)	9	80	4	16.32

Results and discussion. The analyses are performed by GEO5 in our case study; we used the Mohr-coulomb model. The Mohr-Coulomb model, being a robust and simple non-linear model, is based on soil parameters that are known in most practical situations. Not all nonlinear features of [16] soil behavior are included in this model, however. The Mohr Coulomb model may be used to compute realistic support pressures for tunnel faces, ultimate loads for footing, and so on.

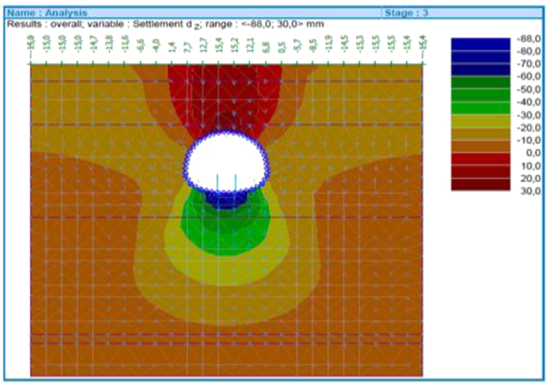

Fig. 7. State of settlement after the total excavation of the tunnel

Settlement state after digging. Total excavation with 25 cm of shotcrete supports. Fig. 7 shows the stress concentration zone above the structure, a concentration of stresses was observed in the parts of the structure closest to the surface, and as was shown by Volkmann & Schubert (2006), the support with 25 cm shotcrete seems to reduce the movements above the tunnel and their propagation to the surface. This phenomenon was also shown by Shin, et al. (2008) for the pre-support [17, 18].

Excavation of the calotte with 30 cm of shotcrete and support for the temporary base.

Step 1:

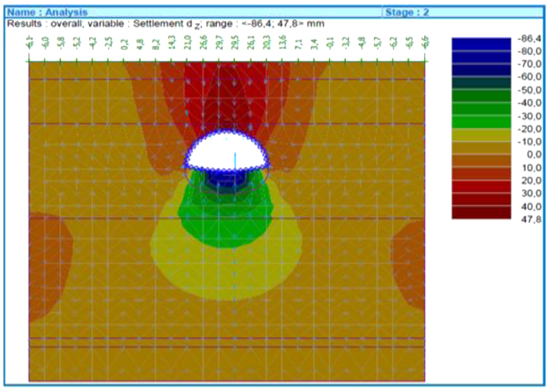

Fig. 8. Settlement after excavation of the calotte with 30 cm of shotcrete and installation of a support at the provisional slab

Step 2:

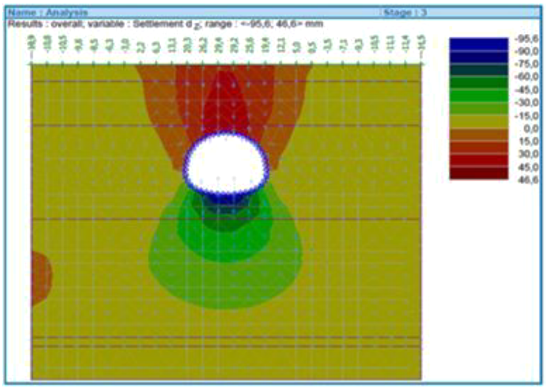

Fig. 9. Settlement after excavation of the entire section with 30 cm of shotcrete

Figs. 8 and 9 show the distribution of stresses induced by sagging for the two cases studied respectively. There is a significant increase in the vertical stress above the structure with displacements of 47.80 and 46.6 mm, whereas the zone located under the structure seems confined to the point that subsidence induces very few constraints.

For reinforced concrete structures, the tensile deformation of the structure must be at least equal to 1.5 mm/m for cracks to appear; then for deformations of the structure of 3 mm/m, the damage can be considered as severe. For the French Association of Tunnels and underground Space, an order of magnitude [19] for the deformations tolerated on the surface in underground works and in urban sites is about 1/1,000 (0.1 %) of distortion. In our case the displacements exceed the authorized limits, and given the characteristics of the current section, there are possibilities to reduce the current declivity within the limits authorized for this type of line (maximum slope = 40 ‰) in order to increase the coverage (height of earth on the key) of the tunnel and consequently, reduce surface settlements.

Conclusion.

1. The study has highlighted the significant influence of different geotechnical parameters on the stability of mining structures, in particular the angle of internal friction, the Young's modulus and the behavior model. So, we must give great importance when estimating and choosing these parameters.
2. The model used for modeling requires a limited number of factors and the combined influence of several parameters has not been considered here, and of course the rest of the parameters can be even more important.
3. The work approaches changes in tunnels made in mines then in urban areas, the objective should be the feasibility of construction and the limitation of surface settlements, so that neither the buildings nor the infrastructures near the tunnel are impacted
4. The impact of deformation is greater when the distance between the tunnel and the ground surface is small.
5. An influence of the soil-structure interaction down to a depth of about 5 m is quite remarkable, which coincides with the experimental results of KWIATKA (1998), who recommends in areas at risk of subsidence a geotechnical survey at least 3 m deep.
6. The geotechnical study must include the reconnaissance of the grounds crossed by the structure but also of those likely to subside under the untreated thickness.
7. This problem, whose superficial consequences can be serious when using unsuitable prediction methods, must be treated wittingly.

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Чисельне дослідження деформацій навколо підземних гірничих споруд (Алжир)

С. Бердуді[*1], *М. А. Р. Морслі*[1], *З. Мекті*[1], *А. Бенсельгуб*[2]

1 – Лабораторія Lavamine, Університет Баджі Мохтар, м. Аннаба, Алжир

2 – Лабораторія збагачення корисних копалин і навколишнього середовища, Університет Аннаби, м. Аннаба, Алжир

[*] Автор-кореспондент e-mail: berdoudisaid@yahoo.fr

Мета. Дослідження стійкості підземних гірничих споруд з використанням чисельного методу на основі кінцевих елементів, двовимірного (2D) кінцево-елементного моделювання із застосуванням розрахункового модуля GEO5.

Методика. Для того, щоб урахувати геотехнічні параметри, прохідка тунелів здійснюється Новоавстрійським способом (НАСП). З метою контролю осадки в м'яких ґрунтах і проведення робіт у повній безпеці ми використовували програмне забезпечення, засноване на методі кінцевих елементів.

Результати. Визначення діапазону зсувів ґрунту та їх прогнозування необхідні при проектуванні даного типу споруд для забезпечення безпеки активного середовища, розташованого в зоні його впливу.

Наукова новизна. Новизна даної роботи полягає в тому, що охарактеризовано ґрунтовий масив регіону, який вивчається, визначені різні фізико-механічні властивості, а також виконане моделювання за допомогою сучасного обчислювального коду, заснованого на моделі поведінки ґрунтів Кулона-Мора.

Практична значимість. Результати цього дослідження, отримані для однієї із секцій з використанням розрахункового коду GEO5, показують, що переміщення перевищують допустимий рівень; їх значення становлять 47,80 і 46,6 мм відповідно на першому та другому кроках, що може призвести до значних зміщень ґрунту. В якості рішення можливе зменшення поточного ухилу в межах, дозволених для даного типу лінії (максимальний ухил = = 40 ‰), для того, щоб збільшити перекриття (висоту від ґрунту до замку склепіння) тунелю і, відповідно, зменшити осідання поверхні.

Ключові слова: *зсув ґрунту, тунель, чисельне моделювання, осідання, підземна споруда, метод кінцевих елементів, GEO5*

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