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D. E. Boudjellal[1], A. Hafsaoui[1], Prof., A. Aissi[2], Dr. Sc. (Tech.)

1 – Mineral Resources and Development Laboratory, Badji Mokhtar University, Annaba, Algeria, e-mail: djameljimi28@gmail.com
2 – Mining, Metallurgy and Materials Laboratory, National High School of Mining and Metallurgy, Annaba, Algeria

STUDY ON THE INTERACTION OF THE ROCK MASSIVE AND SUPPORT SETTING OF TUNNELS

Purpose. Studying the behaviour of underground working and specifying the design of its support. Modelling of a tunnel is done in the three-dimensional problem statement due to the interaction between the mine rocks and excavation support.

Methodology. The study on the interaction between an excavated massif and excavation support was conducted using the convergence-confinement method. Moreover, methods were used developed to determine the factors intervening during the interaction of the rock and support and to establish a structural flowchart for the conduct of the tunnel. Two methods are used for defining the factors of interaction of the rock and support. They are computational analytical convergence-confinement method and numerical method based on Plaxis^{2D} software.

Findings. Based on the results of the simulation at (PK-0 + 625) we can claim that the support with dimensions of 0.75 to 1 m + along with shotcrete with a thickness of 0.2–0.3 m, whose parameters are proved empirically, is reliable. Yielding of the construction recorded in the tunnel also testifies to the reliability of this support.

Originality. The results obtained through the empirical method show that the proper support should correspond to type S3.

Practical value. The results of the numerical simulation correspond to the data of the maximum vertical displacement, which makes $4.18 \cdot 10^{-3}$ m for the bed rock. This will allow obtaining possible decrease in tunnel outline, which makes properly $2.7 \cdot 10^{-3}$ m with a floor heave of $0.75 \cdot 10^{-3}$ m and the field of horizontal displacements of the order of $1.45 \cdot 10^{-3}$ m. As a result of the research, the problem of mass displacement is solved with higher technical and economic efficiency.

Keywords: *tunnel, convergence-confinement, interaction, solid mass behaviour, support, Plaxis^{2D}*

Introduction. The convergence-confinement calculation has the main purpose to study the behaviour of an underground construction and to size in first approach its retaining structure or its coating. It corresponds to the method of the same name developed by Champagne De Labriolle. G. [1] and is resumed in the recommendations of the AFTES [1, 2]. The modelling of a tunnel must take into account two essential elements; these are a three-dimensional problem due to the presence of the front of the size and a problem of interaction for which the coupling between the ground and the retaining structures is important.

By convention, this fictitious pressure is noted σ_R. The initial state ahead of the front and at a sufficient distance to neglect its influence corresponds to λ (the rate of decontainment) = 0. As the digging is closer to the section considered, then exceeds it and moves away, λ progressively increases from 0 to 1.

The Convergence-Confinement method makes it possible to deal with the case of circular tunnels, achieved in a homogeneous isotropic mass [1, 3]. The characteristic curve of the ground is defined according to the formula of M. Panet [1], established for a perfect elastoplastic medium of the Mohr-Coulomb type.

The convergence-confinement also method makes it possible to reduce a two-dimensional plane deformation calculation in a plane perpendicular to the tunnel axis, assuming that everything happens as if the convergence was due to the decrease in a fictitious support pressure with the distance of the face of size.

From ground characteristics, support characteristics and flight length (distance not supported d), it is possible to determine the displacement of the ground at the installation of the support uso and the rate of deconfinement λd corresponding to it by five methods:

1. Similarity of Corbetta [4].
2. Similarity of M. Panet [1].
3. Conventional implicit method according to the method presented by M. Panet [1].
4. New implicit method according to the formulation presented by Bernaud and G. Rousset [4].
5. New implicit method of Nguyen Minh and Guo [4] according to the formulation presented in the AFTES recommendations [2, 4].

The balance of the massif is reached at the intersection of the curves of the ground and the support. The values of λ, u_R (the convergence of the wall) are calculated, as well as the pressure applied to the support-lining for this intersection, and the safety factor of the retaining-lining is displayed.

In this work, we have developed an explicit solution for massive rock-support interaction for a tunnel in anisotropic environment. Two methods of computational convergence-confinement method and software numerical method Plaxis^{2D} were used to determine the factors of the interaction of the rock-retaining mass and to choose the support used.

Description of the layout. The layout, crossing the historical zone of Algiers, includes a tunnel in all its extension and includes 2 underground stations (Ali Boumendjel and Place of the Martyrs). Thus, there are 4 principal wells (PV1), (PV3), as well as the Southern and Northern wells of Ali Boumendjel Station. These wells constitute the faces affected by the underground work, except for the PV3 and the Northern Well of Ali Boumendjel Station. The tunnel is developed at 1453 m and consists of 3 principal parts; the extension and localization are indicated as follows:

- the part situated within –0 + 594 381 km and 0 + 181 121 km of an extension of 413.26 m, ranging between the terminus of the tunnel (in the North) and the Place of the Martyrs Station;
- the part situated within –0 + 21 120 km and 0 + 643 613 km, 664.73 m of extension, ranging between the Place Of The Martyrs Station and Ali Boumendjel Station;
- a part situated within 0 + 799 331 km and 1 + 145 594 km, 346.26 m extension, ranging between the Ali Boumendjel Station and the Ventilating well 3.

Our work goes on the project of execution of the prolongation of the subway tunnel between the terminus of Extension A Line 1, located close to the PV1 (Place Emir Abd-el-Kader – Place of the Martyrs) and the Taleb Abderrahmane Station, which belongs to the Subway of Algiers.

The section under analysis, of an extension of approximately 150 m, will be affected starting from the terminus of Extension A.

General geology. The area of Algiers can be represented in a simplistic form like a metamorphic primary dome, indicated by solid mass of Algiers, which is bordered by tertiary and quaternary sedimentary formations. The metamorphic formations recognized along the layout of this extension of the tunnel primarily consist of schists with sericites, with intercalation of lenticular metamorphic limestone levels. By basing on the geological map of Algeria on the scale 1/50 000, and by the observation of drill-cores, it was possible to individualize the following geological units:

***Recent formation*s.** E-Embankments of 1.0 and 2.5 m and A-Alluvium of 3.4 to 7.8 m.

Metamorphic rocks. S-Schist with sericites, with intercalation of lenticular level of metamorphic limestone. It was possible to distinguish two different geotechnical horizons; the first is more superficial indicated by *Sa*, the second above the first, indicated by *Sb*.

Sa. It consists of much deteriorated schist of broken up and closed fractures, frequently presenting intercalations of argillaceous levels with small schist fragments, produced by the deterioration of the schistous solid mass. The thickness of this horizon varies between 5.0 and 12.0 m.

Sb is primarily made of fairly deteriorated schist with some passage of a little deteriorated and of a close fracture fairly moved away; we often find the presence of clay, which indicates to water circulation in-depth.

Application of the convergence-confinement method. *Convergence.* Table 1 represents the parameters used for the calculation of the convergence in the walls of the tunnel.

I. e. the bringing together of the walls of the tunnel**:**

- shearing modulus $G = 269\ 230.77$ kN/m^2;
- resistance in compression of the solid mass

$$\sigma_c = 460 \text{ kN/m}^2. \text{ So } \sigma_0 > \frac{\sigma_c}{2};$$

- coefficient of thrust $K_p = 5.29$;
- the rate of decontainment at the end of the elastic phase $\lambda_e = 0.81$;
- the radial constraint corresponding at the end of the elastic phase to $\sigma_{\mathrm{Re}} = 115.39$ kN/m^2;
- convergence with the interaction of the characteristic curve of the ground in elastic ehavior and of the axis of convergence in wall $U_{R0} = 5.63 \cdot 10^{-3}$ m.

If the ground remains in the linear elastic range, the final displacement will be 5.63 mm:

- convergence at the end of the elastic phase $U_{\mathrm{Re}} = 4.5 \times \times 10^{-3}$ m;
- calculation of displacements U in the plastic zone:

R_p according to λ by the relation of Panet [1].

Knowing $\alpha = 1$ (Table 2).

Moreover, there is another factor which mostly affected many sites which must be taken into account in this study: the decompressed zone leads to a curve of different characteristics of ground pier and vault (Fig. 1).

Confinement. To calculate the convergence acquired by the wall at the time of the installation of support setting, we use the formulations of M. Panet (Principle of the similarity) [1, 4]:

Table 1

The parameters used for the calculation of the convergence in the walls of the tunnel

The radius of the excavation (R), m	5.12
Initial constraint (σ_0), kN/m²	592.85
The cohesion of the ground (C), kN/m²	100
The internal friction angle, φ°	43
Young modulus (E), kN/m²	700 000
The Poisson's ratio (ν)	0.3

Table 2

Various values of curved convergence-confinement

λ	$R_p(\lambda)/R$	σ [kPas]	$U(\lambda)$
0.00000	0.766	592.85	0.00000
0.57000	0.893	254.93	0.00362
0.61070	0.907	230.80	0.00374
0.65140	0.923	206.67	0.00387
0.69210	0.940	182.54	0.00401
0.73280	0.960	158.41	0.00418
0.77350	0.981	134.28	0.00437
0.81420	1.006	110.15	0.00459
0.85490	1.034	86.02	0.00485
0.89560	1.066	61.89	0.00516
0.93630	1.105	37.76	0.00554
0.97700	1.153	13.64	0.00604
1.00000	1.186	0.00	0.00638

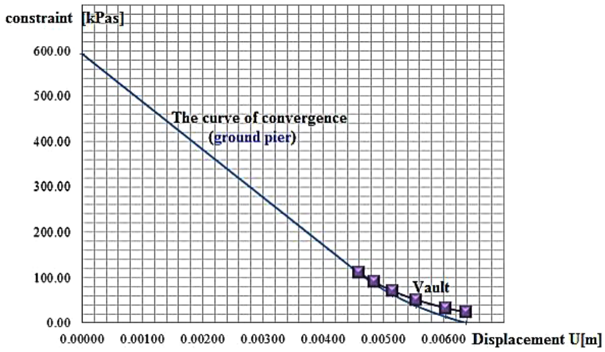

Fig. 1. Taking into account the effect of gravity in the calculation of the characteristic curve of the ground

- the assumption of application of the method is checked $N_s = 2.57 < 5$;

with U_0 radial displacement with the face of the tunnel ($U_0 = = U_{R0}$).

$$U_{s0} = 3 \cdot 10^{-3} \text{ m};$$

while respecting the following condition

$$U_{s0} > 0.265 \cdot U_{R0};$$

$$U_{s0} > 1.48 \cdot 10^{-3} \text{ m};$$

- stiffness of the hanger;
- the stiffness of the hanger K_c (KPa)

$$K_c = 222.71 \text{ KPa};$$

- maximum pressure of the hanger HEB 160

$$\sigma_{c\,\max} = 0.25 \text{ MPa};$$

- displacement of the hanger HEB160

$$U_c = 5.9 \cdot 10^{-3} \text{ m};$$

- thickness of shotcrete, e

with γb (voluminal weight) = 25 kN/m^3;

f_{c28} (compressive strength in 28 days) = 25 MPa;

2 cm < e' < 4 cm, we take e' equal to 4 cm;

e'' before installation of the shotcrete, a layer of 3 cm is made to adjust the shape of the vault (legalization layer), so $e = 22$ cm;

Table 3

Characteristics of the hanger HEB 160

Hanger HEB 160	
The modulus of elasticity I (MPa)	210 000
The section of the hanger A_c (cm²)	54.30
Spacing e (m)	1
The radius R (m)	5.12
The resistance of the steel σ_e (MPa)	240
Stiffness of the hanger K_c (KPa)	222.71
I_c (m^4)	$10^{-5} \cdot 0.005696$
E_cA_c (KN/m)	$10^6 \cdot 1.14$
EI (KN.m²/m)	$10^3 \cdot 5.233$

- the stiffness of the shotcrete $K_b = 0.45$ MPa;
- maximum pressure of shotcrete $\sigma_{b\,max} = 0.62$ MPa;
- displacement of the concrete $U_b = 7 \cdot 10^{-3}$ m;
- maximum support pressure P_{max} (concrete + hanger)

$$P_{max} \text{ of retaining (concrete + hanger)} = \sigma_{c\,max} + \sigma_{b\,max};$$

$$\sigma_{c\,max} \text{ (hanger)} = 222.71 \text{ KPa};$$

$$\sigma_{b\,max} \text{ (concrete)} = 608.72 \text{ KPa};$$

$$P_{max} \text{ (hanger + concrete)} = 831.43 \text{ KPa};$$

$$P_{max} > \sigma_0;$$

- displacement of supporting (hanger + shotcrete)

$$U_{c+b} = 6.32 \cdot 10^{-3} \text{ m};$$

- final displacement of support $U = 9.32 \cdot 10^{-3}$ m.

Fig. 2 represents the convergence-confinement curve and the equilibrium point obtained by the intersection of convergence and containment characteristic curves.

The point of balance obtained by the intersection of the characteristic curves of convergence and confinement

$$U_{eq} = 4.2 \cdot 10^{-3} \text{ m and } \sigma_{eq} = 161 \text{ kN/m}^2.$$

Validation of support setting by PLAXIS^{2D} software. The main objective of this study is to evaluate the soil-structure interaction during tunnel digging by the NATM method in a numerical simulation of bidimensional tunnel digging. To achieve this result, two-dimensional numerical simulations were conducted in parallel [5].

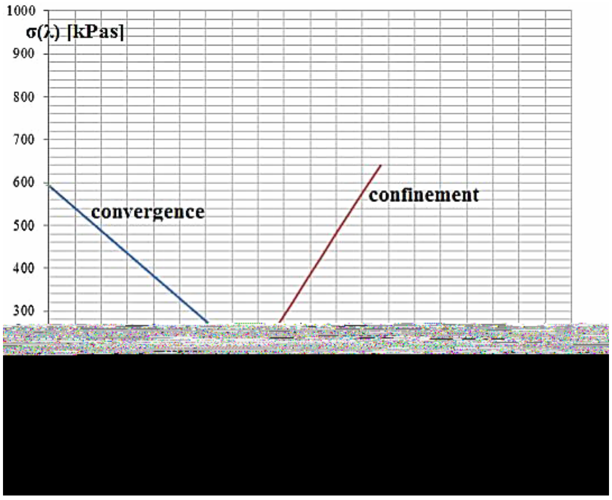

Fig. 2. The convergence-confinement curve and the equilibrium point obtained by the intersection of convergence and confinement characteristic curves

The calculation code, which was used for this analysis, is Plaxis^{2D} [6, 7].

The NATM method dates from the late thirties. It has proved to be effective and can limit the decompression of land in the vicinity of the excavation [8]. This method is characterized by the following [9, 10]:

- the establishment, as quickly as possible, of a deformable support consisting of, on the one hand, bolts that arm the ground and, on the other hand, a layer of shotcrete;
- the purpose of this support is to guarantee the stability of the excavation by creating a carrier ring, constituted by the armed ground;
- the excavation is done in a full or half section and sometimes in a divided section in order to put the support in place quickly;
- the sealed anchors (mortar or resin) create an armed vault in the ground;
- a thin shell of shotcrete (5 to 25 cm), generally reinforced with welded mesh, protects the ground against the alteration, creates continuity between the elements of the ground, brings a radial pressure of confinement and distributes the forces reported to the head anchors;
- optionally, light slider vaults strengthen the hull of the shotcrete;
- the set, therefore, allows the ground to participate in the support because of the formation of a monolithic vault (pavement + ground) The terrain thus becomes self-supporting.

The results obtained by the empirical methods remain very vague as regarding the dimensioning of the solutions; their use will have to be considered as a process of pre-dimensioning which will have to be validated and supplemented by recourse to pushed numerical analyses. For the realization of these analyses, we will resort to the Plaxis^{2D} software; the constitutive example chosen to model the behaviour of these formations is Mohr-Coulomb [2, 5].

This includes a perfectly plastic elastic behaviour; it presents as a major advantage the fact of being the most commonly used model in the simulation of geotechnical works, which allows the experienced user a greater ease in the interpretation of its behaviour; the massif will be discretized by resorting to triangular elements of 15 points.

Digital simulation (km No –0 + 625). ***Characteristics of the temporary support type S3.*** The support to be applied around the section consists of HEB160 hangers, moved away by 1.0 m, associated with 25 cm of shotcrete including an electro-welded mesh (1.96 cm^2/m).

The subsequent application of a 3 cm layer of projected concrete will be used to regularize surface, giving a total thickness of 28 cm of projected concrete.

A.1 – Projected voluminal weight (Table 4):
– voluminal weight: $\gamma = 24$ kN/m^3;
– young modulus: $E_0 = E_\infty = 10\,000$ MPa;
– Poisson's ratio: $\nu = 0.2$;
– the thickness of the concrete: $e = 0.28$ m.

A.2 – Hanger HEB 160 (Table 3).

A.3 – Combination (projected concrete + HEB160 hanger): (Table 5) shows the characteristics of the superstructure according to the combination (projected concrete + hanger HEB160).

Design of the Model. ***Geotechnical characterization***. The Q-system also makes it possible to define the support mode to put in place, provided that we know the value of Q, the width of the excavation and the function of the excavation.

From the analysis of more than 200 underground caverns (mainly road and hydroelectric tunnels), Barton of the Norwegian Geotechnical Institute (NGI), have proposed an index for the determination of the quality of a rocky massif for tunnelling [10].

Quality index Q is determined by 6 parameters as follows

$$Q = (R.Q.D/Jn).\ (Jr/Ja).\ (Jw/SRF).$$

Table 4

Characteristics of shotcrete

Shotcrete C 25/30	
The modulus of elasticity E (Mpa)	10 000
The thickness e (m)	0.22
Poisson's ratio ν	0.2
The stiffness K_b (Mpa)	0.45
Maximum pressure $\sigma_{b\max}$ (Mpa)	0.60
A (m²)	0.28
I (m^4)	$10^{-3} \cdot 1.82$
D	1
EA(kN/m)	$10^6 \cdot 2.8$
EI(kN.m²/m)	$10^4 \cdot 1.8293$

Table 5

Characteristics of the superstructure according to the combination (projected concrete + hanger HEB160)

Parameter	Name	Value	Unit
Type of Conduct	Type Material	Elastic	–
Normal stiffness	EA	$1.38 \cdot 10^7$	kN/m
Bending stiffness	EI	$2.35 \cdot 10^4$	kNm²/m
Equivalent thickness	d	0.143	m
Poisson coefficient	n	0.2	–

Three classes of rocks with more or less distinct characteristics were selected:

CR1, CR2 and CR3 to which three Geotechnical horizons correspond (Table 6).

Rock class CR1 Horizon Sb; Description: altered schists (W3), with slightly altered passages (W2), and close to medium-distant fractures (F4-3); GSI34 to 52; Quality of the solid mass: reasonable; Resistance of the intact rock 40 and 80 MPA, average resistance values of the order of 60 MPa; g (kN/m^3) 28.

Rock class CR2 Horizon Sa; Description: highly altered to decomposed schists (W4-5) and very close-fractures (F5); GSI23 to 34; quality of the solid mass: reasonable to low resistance of the intact rock: 15 and 30 MPa; γ (kN/m^3) 25.

Rock class CR3 Horizon Sa; Description: materials of deterioration of the schists; GSI10; Quality of the solid mass: very low; resistance of the intact rock: 5 MPa; γ (kN/m^3) 21.

For tunnel extension, starting at PK 29 + 405 and ending at PK 29 + 255 next to Taleb Abderrahmane Station, the parameters of calculation of the rock classes were distributed for the representative kms: km 29 + 300 (profile B) and km 29 + 375 (profile C). In a general way, the tunnel will cross a solid mass made up of Schist with sericites, including lenticular intercalations of metamorphic lime stone levels mainly out of schist fairly not very altered (W3-2) and very moderately fractured (F4-3), which corresponds to the geotechnical unit Sb, with a passage from the geotechnical unit Sa, very altered schist to decomposed (W4-5), and very fractured (F5); the water levels detected by the piezometers are between 29 and 30 m of depth. Along the development of the tunnel, covering is about 25 m.

For our model, boundary conditions will be defined as typical fixations for this type of the problem, i. e. blocked horizontal displacements at the lateral ends of the model and vertical and horizontal displacements prevented at the base of the model. The borders of the model will be defined with a sufficient distance from the area concerned by the tunnel, so that the effect is not felt in the conduct of the model. The threading elements are not taken into account in the calculation of the developed models for the pre-dimensioning of the supporting set in the preliminary draft. These elements are considered not to relate to dimensioning sufficiently close to reality and they work as protection of the roof during the excavation and installation phases of metal hangers.

Generation of the mesh. The model of reference is done by elements with 15 nodes. The number of elements is 314 elements and the number of nodes is 2677 knots. The mesh is presented in (Fig. 3).

Executive phasing includes [5]:

Phase 0. Generation of the initial state of constraint through the loading of the gravity.

Phase 1. Excavation of the upper half-section, with a decontainment coefficient of 40 %.

Phase 2. Installation of the shotcrete support of the upper half-section.

Phase 4. Excavation of the Stross with a decontainment coefficient of 60 %.

Phase 5. Execution of the final raft.

Results obtained. ***Vertical displacements.*** Fig. 4 shows the vertical displacement field obtained in the final phase of computation making approximately $4.18 \cdot 10^{-3}$ m on the level of the keystone, from which the occurrence of reduced subsidence at the surface can be predicted, of the order of $2.7 \cdot 10^{-3}$ m with a lifting of the raft $0.75 \cdot 10^{-3}$ m.

Horizontal displacements. Fig. 5 represents the field of horizontal displacements in the (X-X) direction with a maximum displacement of the order of $1.45 \cdot 10^{-3}$ m.

Deformations at the tunnel level. Fig. 6 shows the deformations at the vault, raft and piers (the tunnel elements).

Conclusion. Based on the results of the simulation at (PK-0 + 625) we can claim that the support with dimensions of 0.75 to 1 m + along with shotcrete with a thickness of 0.2–0.3 m, whose parameters are proved empirically, is reliable. The proper support should correspond to type S3 (HEB 160 hanger with a spacing of 1m + shotcrete with a thickness of

Table 6

The geotechnical parameters used to calculate the vertical and horizontal displacements of the model

Rocky Classes	CR1		CR2	CR3	Embankments
–	Non-disturbed	disturbed	–	–	–
γ (kN/m^3)	28	28	25	21	18
ϕ (°)	52	48	48 (profile B) 43 (profile C)	32	30
c (kPa)	350	250	90 (profile B) 100 (profile C)	10	5
E (GPa)	3.5	1.5	0.7	0.1	0.015
K_0	0.8	0.8	0.6	0.5	0.5

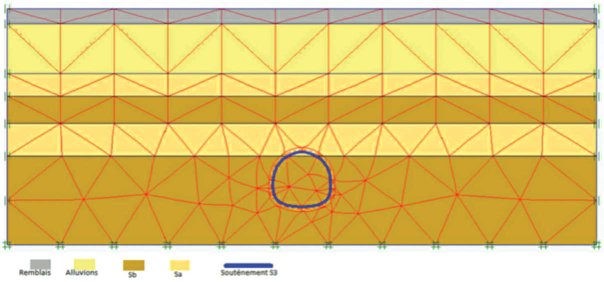

Fig. 3. Model mesh

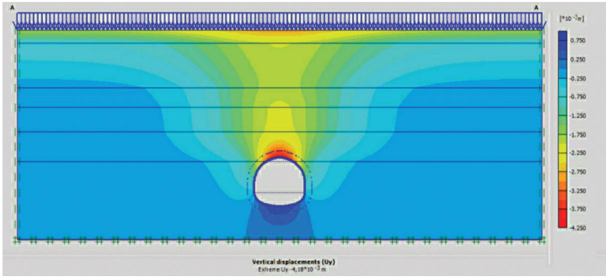

Fig. 4. Vertical displacement in 2D

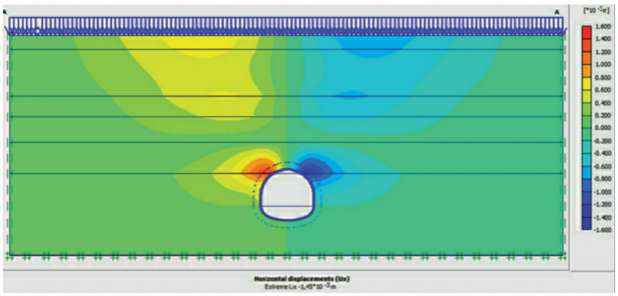

Fig. 5. Horizontal displacement in 2D

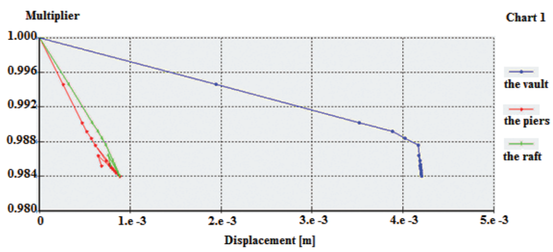

Fig. 6. The deformations at the level of the vault, the raft and the piers

0.28 m). The convergences recorded at the level of the tunnel testify to the reliability of this support.

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Дослідження взаємозв'язку між гірським масивом і кріпленням тунелів

Д. Є. Буджелал[1], *А. Хафсаyі*[1], *А. Аіссі*[2]

1 – Лабораторія мінеральних ресурсів і розвитку, Університет Баджі Мохтар, м. Аннаба, Алжир, e-mail: djameljimi28@gmail.com

2 – Лабораторія гірничої справи, металургії та матеріалів, Національна гірничо-металургійна школа, м. Аннаба, Алжир

Мета. Вивчення поведінки підземної виробки й визначення конструкції її кріплення. Моделювання тунелю виконано в об'ємній постановці завдання через наявність взаємодії між гірськими породами та кріпленням виробки.

Методика. Дослідження взаємодії між масивом гірських порід і кріпленням гірничої виробки проводилися методом наближень і обмежень. Також використовувалися методи, що розроблені для визначення перешкоджають факторів, які виникають у процесі взаємодії масиву та кріплення, і розробити структурну схему прове-

дення тунелю. Для визначення факторів взаємодії масиву та кріплення використовуються два методи. Це обчислювальний аналітичний метод наближень та обмежень і чисельний метод, заснований на програмному забезпеченні Plaxis^{2D}.

Результати. На підставі результатів моделювання (PK-0 + 625) можна стверджувати, що кріплення розміром від 0,75–1 м спільно з торкретбетоном товщиною 0,2–0,3 м, параметри якого обґрунтовані емпірично, є надійною. Піддатливість конструкції, зареєстрована в тунелі, також свідчить про надійність даного кріплення.

Наукова новизна. Результати, отримані за допомогою емпіричного методу, показують, що відповідне кріплення повинно відповідати класу S3.

Практична значимість. Результати чисельного моделювання відповідають даним максимального вертикального зсуву, що становить $4{,}18 \cdot 10^{-3}$ м для корінних порід. Це дозволяє отримати можливе зниження контуру тунелю, що становить близько $2{,}7 \cdot 10^{-3}$ м при обдиманні ґрунту $0{,}75 \cdot 10^{-3}$ м і областю горизонтального зміщення приблизно $1{,}45 \cdot 10^{-3}$ м. У результаті виконаних досліджень розв'язано проблему зсувів масиву з вищою технічною та економічною ефективністю.

Ключові слова: *тунель, наближення та обмеження, взаємодія, поведінка масиву, кріплення виробки, Plaxis^{2D}*

Исследование взаимосвязи между горным массивом и креплением туннелей

Д. Е. Буджлал[1], *А. Хафсауи*[1], *А. Аисси*[2]

1 – Лаборатория минеральных ресурсов и развития, Университет Баджи Мохтар, г. Аннаба, Алжир, e-mail: djameljimi28@gmail.com

2 – Лаборатория горного дела, металлургии и материалов, Национальная горно-металлургическая школа, г. Аннаба, Алжир

Цель. Изучение поведения подземной выработки и определение конструкции ее крепления. Моделирование тоннеля выполнено в объемной постановке задачи из-за наличия взаимодействия между горными породами и креплением выработки.

Методика. Исследование взаимодействия между массивом горных пород и креплением горной выработки проводились методом приближений и ограничений. Также использовались методы, которые разработаны для определения препятствующих факторов, возникающих в процессе взаимодействия массива и крепи, и разработать структурную схему проведения тоннеля. Для определения факторов взаимодействия массива и крепи используются два метода. Это вычислительный аналитический метод приближений и ограничений и численный метод, основанный на программном обеспечении Plaxis$^{2D.}$

Результаты. На основании результатов моделирования (PK-0 + 625) можно утверждать, что крепь размером от 0,75–1 м совместно с торкретбетоном толщиной 0,2–0,3 м, параметры которой обоснованы эмпирически, является надежной. Податливость конструкции, зарегистрированная в тоннеле, также свидетельствует о надёжности данного крепления.

Научная новизна. Результаты, полученные с помощью эмпирического метода, показывают, что соответствующее крепление должно соответствовать классу S3.

Практическая значимость. Результаты численного моделирования соответствуют данным максимального вертикального смещения, что составляет $4{,}18 \cdot 10^{-3}$ м для коренных пород. Это позволяет получить возможное снижение контура туннеля, которое составляет порядка $2{,}7 \cdot 10^{-3}$ м при пучении почвы $0{,}75 \cdot 10^{-3}$ м и областью горизонтального смещения порядка $1{,}45 \cdot 10^{-3}$ м. В результате выполненных исследований решена проблема смещений массива с более высокой технической и экономической эффективностью.

Ключевые слова: *тоннель, приближения и ограничения, взаимодействие, поведение массива, крепь выработки, Plaxis^{2D}*

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