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New development using the "convergence-confinement" method in an anisotropic stress field

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ABSTRACT: The French project CIGEO for an underground nuclear wastes repository in the Callovo-oxfordian claystone of the Eastern France entered a new phase of design, which should be the last one before the works launching. The 1st design stage, named as “esquisse” requires some rough but fast analysis of the structural lining of the galleries. The service life of it, is assumed to be of approximately 100 years. In the tectonic context of the Parisian basin, the ratio of anisotropic stresses is quite high at 500m depth and the horizontal high stress leads some squeezing, moderate but associated to the creep behavior. That is why a new development using an approximate solution for the elasto-plastic stress field, according to the “convergence-confinement” method, is coupled with an experimental procedure for the fitting of the long term ground behavior, in order to estimate the equivalent modulus and residual cohesion to the long term. The short term rendering is roughly based on the semi-analytical solution from Detournay & Fairhurst in an elasto-plastic medium, ruled by a Mohr-Coulomb criterion with softening. The empirical creep law is of the p-power type, according to the available measurements in the underground laboratory in Bure. Then, the method is applied as load concept of the structure, depending on the setting delay, according to the tunneling means, with or without TBM .

1 Introduction

At the 2130 horizon, the project CIGEO of the french nuclear waste repository could form a set of galleries, dozens of kilometers long , connecting hundreds of cells, lying in a 15 square kilometers area at 500 m depth into the Callovio-Oxfordian layers (COX) of the Eastern Parisian basin. Thus, according to a hundred years’ service life, the structural lining of the galleries is confronting with the long term behaviour of the rock mass.

The particular characteristics of the argillaceous rock mass, subjected to creep effects, are preponderant in the design of many underground structures, especially at great depths. This mechanism of delayed behaviour progresses slowly charging the underground structures. The role of the lining is to resume these stresses in the long term in order to control the deformation during the service life and to ensure the stability.

In addition, in accordance with the structural context, an anisotropic stress field often exists in these geological layers with a subhorizontal major principal stress, according to the dip direction.

This is the reason why methods of analysis for structural design, even in a preliminary stage, must consider the main characteristics of the long term behaviour, as moderate softening, associated with creep in an anisotropic stress field. At preliminary stage of design, the reference method in axisymmetric conditions is the convergence-confinement approach, to be developed this time in an anisotropic stress field. The method is based on the interaction between the soil and the structure taking into account a simplification of the three dimensional effect around the excavation face. According to the main assumption, the excavation of a circular tunnel could be simulated in plane-strain conditions by the progressive cancellation of the radial stress at the wall, in order to induce an evolution of stresses into the rock, from the initial state to the final state. The present development of the software GEOTUNNEL © allows to generalize the convergence confinement method in anisotropic initial stress fields i.e. where the ratio between the principal stresses is different from K=1.00.

2 Principles of the convergence confinement method

The convergence confinement method is based on the interaction between the rock and the structure expressed by three curves:

- The Longitudinal Deformation Profile (LDP) of the ground u(d) depending on the distance from the excavation face d, along the longitudinal axis of the gallery;
- The Ground Reaction Curve (GRC) i.e. the convergence u(Pi) at the wall of the tunnel versus the fictive internal pressure Pi;
- The Support Confinement Curve (SCC) i.e. the pressure Ps [u-uo] provided by the structure versus the convergence u-uo after its installation at a certain distance d from the excavation face.

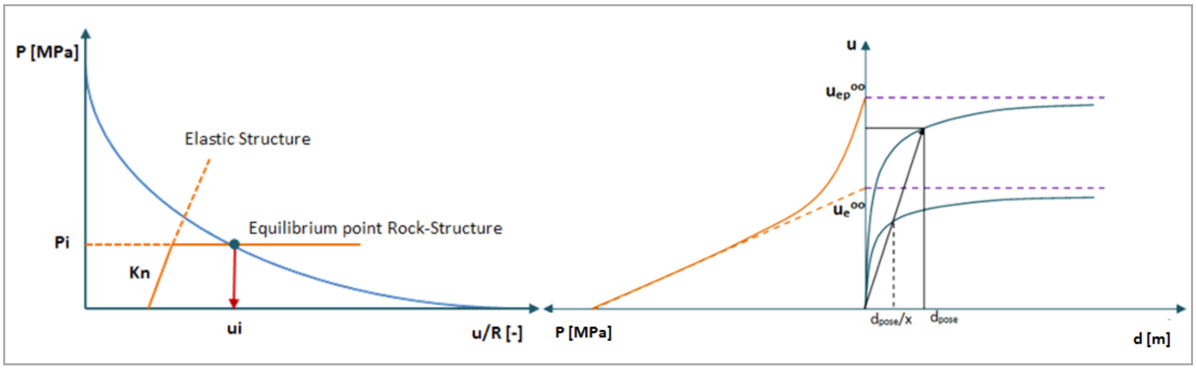

Figure 1 a) GRC-SCC curves, b) Elasto-plastic LDP according to the self-similarity principle, after Corbetta et al. [1]

The development below describes the generalization of the method into the anisotropic stress field for the second and the third curve.

2.1 Anisotropic stress field

The initial stress field is defined by the major and minor principal stresses P_1 & P_2 in directions 1 & 2 respectively. Three radial convergence curves are obtained, corresponding to the directions of the major, the minor and the average stress in a direction of 45° of the previous. The three points are represented in the figure below. The curve depends on:

- the initial stress half-deviator $P_d = ½.(P_1\text{-}P_2)$;
- the average stress $P_m = ½.(P_1+P_2)$;

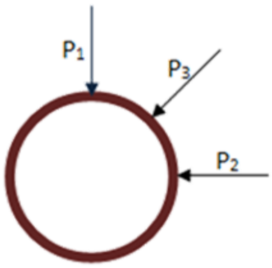

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Figure 5 Tunnel section in an anisotropic stress field

As far as the average curve is concerned it is calculated from the formulas corresponding to an isotropic stress field submitted to the average stress. In the elastic domain, the convergences according to each direction are given by the following formulas:

$$\frac{u_1}{R} = \frac{1+\nu}{E}.\lambda.\left(Pm + (3-4.\nu).Pd\right) \quad (1)$$

$$\frac{u_2}{R} = \frac{1+v}{E}.\lambda.\left(Pm - \left(3-4.v\right).Pd\right) \quad (2)$$

$$\frac{u_3}{R} = \frac{1+v}{E}.\lambda.Pm \quad (3)$$

where R is the tunnel radius and E, v and λ denote the deformation modulus, the coefficient of Poisson and the deconfinement ratio respectively.

In the elastoplastic domain, in the anisotropic field is made the assumption that the plastic convergence occurring at the point 2 is of the form of the isotropic solution. The corresponding convergence is given as:

$$\frac{u_2}{R} = \frac{1+v}{E} \cdot \left[C12 + C2 \cdot \left(\frac{R}{Rp_2}\right)^{Kp-1} + C3 \cdot \left(\frac{Rp_2}{R}\right)^{Kd+1} \right] \quad (4)$$

where Rp denotes the radius of the plastic domain, Kp, the Rankine's passive pressure coefficient and Kd, the dilatancy coefficient $(1<Kd<Kp)$ .The coefficients $C12$, $C2$ and $C3$ ensure the continuity between the elastic and the plastic domain.

In addition is made the assumption, that the elastoplastic convergence at the point 1 depends roughly of the one at the point 2, according to :

$$\frac{u_1^{ep}}{R} = \frac{u_1^{e}}{R} + a_{11} \cdot \frac{u_1^{p}}{R} + b_{21} \cdot \frac{u_2^{p}}{R} \quad (5)$$

Where the coefficients a_{11} and b_{21} should always respect:

$$a_{11} + b_{21} = 1 \quad (6)$$

u_1^e/R is given from the equation (1) and the u_1^p/R *and* u_2^p/R from the following equation:

$$\frac{u_i^p}{R} = \frac{1+v}{E} \cdot \left[C1i + C2 \cdot \left(\frac{R}{Rp_i}\right)^{Kp-1} + C3 \cdot \left(\frac{Rp_i}{R}\right)^{Kd+1} - \frac{E}{1+v} \cdot \frac{u_i^e}{R} \right] \quad (7)$$

with the plastic radius resulting from the Detournay & Fairhurst [2] relation :

$$\frac{Rp_2}{Rp_1} = \left(\frac{\lambda_{em} \cdot P_m + P_d}{\lambda_{em} \cdot P_m - P_d} \right)^{\frac{2}{K_p+1}} \quad \text{and} \quad \frac{Rp_2 + Rp_1}{R} = \frac{Rp_m}{R} \quad (8)\ \&\ (8b)$$

Rp_m the radius of the plastic domain and $\lambda_{em} \cdot P_m$, the half-deviator on the plastic domain frontier, in equivalent isotropic conditions under average stress P_m, are given by :

$$\frac{Rp_m}{R} = \left(\frac{(1-\lambda_{em}) \cdot P_m + \frac{R_{cr}}{Kp-1}}{(1-\lambda) \cdot P_m + \frac{R_{cr}}{Kp-1}} \right)^{\frac{1}{Kp-1}} \quad \text{and} \quad \lambda_{em} \cdot P_m = \frac{Kp-1}{Kp+1} \cdot P_m + \frac{R_c}{Kp+1} \quad (9)\ \&\ (9b)$$

As it has already been mentioned, the rock-mass strength is characterized in the short term by softening, what could be represented according to the Mohr-Coulomb criterion by couples of peak vs residual (noted r) parameters Kp, R_c, with R_c, the unconfined compression strength of the rock mass. In the above, it is assumed :

- Friction angle at peak = residual — Cohesion at peak > residual
- Kp = Kpr — Rc > Rcr

2.2 Comparison with a 2D numerical analysis

In order to validate the accuracy of the results, a parametric study is made by compare witha finite element model using the Z-SOIL software. The role model is a 3m diameter circular tunnel of as presented below.

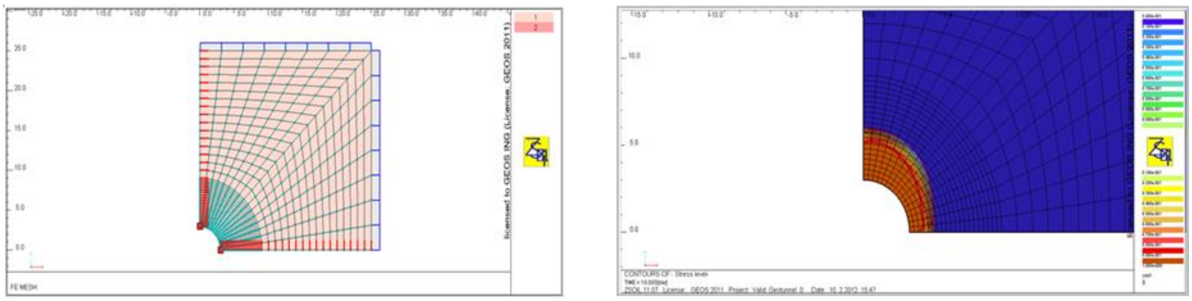


Figure 6 Finite Element Model in Z-SOIL

The following graphics represent the obtained results by varying at a time the friction angle, the ratio between the principle stresses or the cohesion of the rock mass.

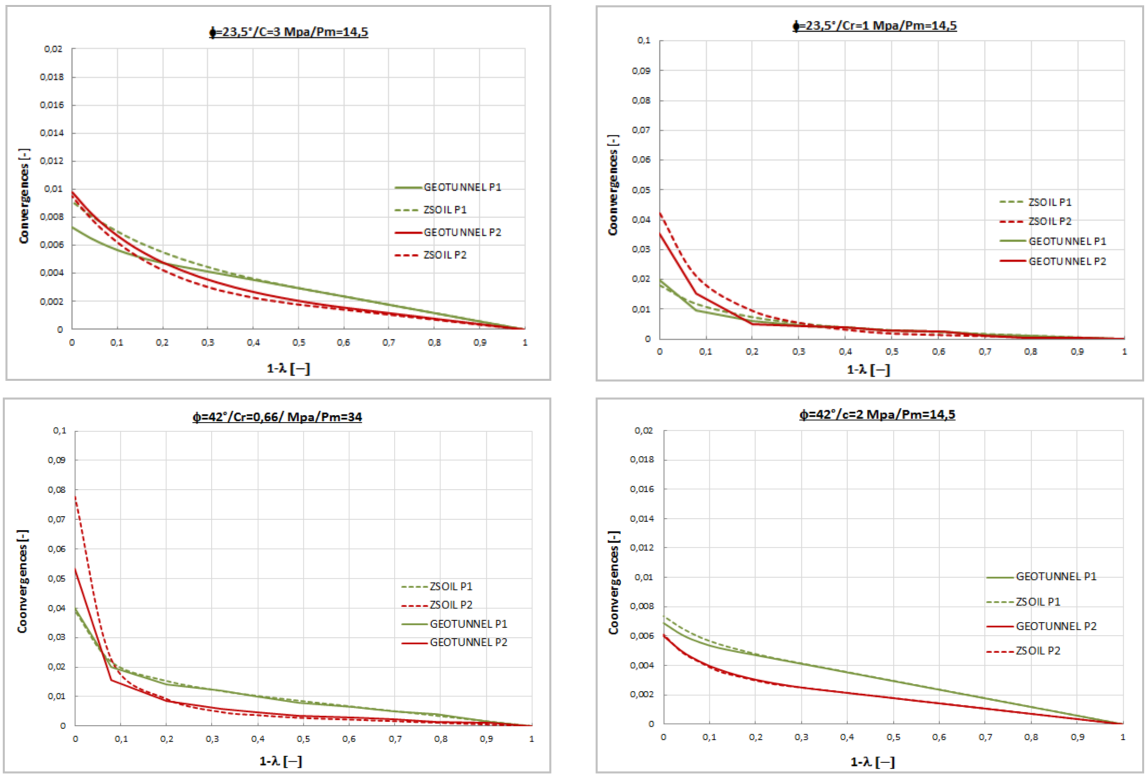


Figure 7 Comparison between the ground reaction curves obtained from the finite element model and the analytical model.

It is obvious that the convergence between the results is satisfactory enough mainly for the cases where the residual cohesion of the rock mass is not taken into account, (figures 4a,d). The localized effects produced in the finite element model, due to the softening cannot be equally well simulated by an analytical model (figures 4b, c). One of the main results is the consecutive shortening of the section contour, which be conforming to the major principal stress direction then to the one of the minor stress, according to the progressive expansion of the plastic domain.

2.3 Long term ground behaviour

The viscoplastic behavior of the rock mass is adjusted via an empirical procedure, by trying to estimate the equivalent deformation modulus and the residual cohesion depending on the time. The time dependent displacements are given assuming that the convergences follow a P-Power law of the form:

$$u = a.t^b \qquad (10)$$

where *u* and *t* denote the convergence and the time, respectively.

The approach consists to investigate the curve which best approximates, using regression methods, the convergence measurements versus time. As it is shown in the figure 8a the p-power law would pretty well fit to the convergences measured in an experimental gallery drilled in the COX and then could be extrapolated.

It should be noted that a setting of a deformation law must be carried out in a sufficiently long period in order to overcome the measurement artefacts.

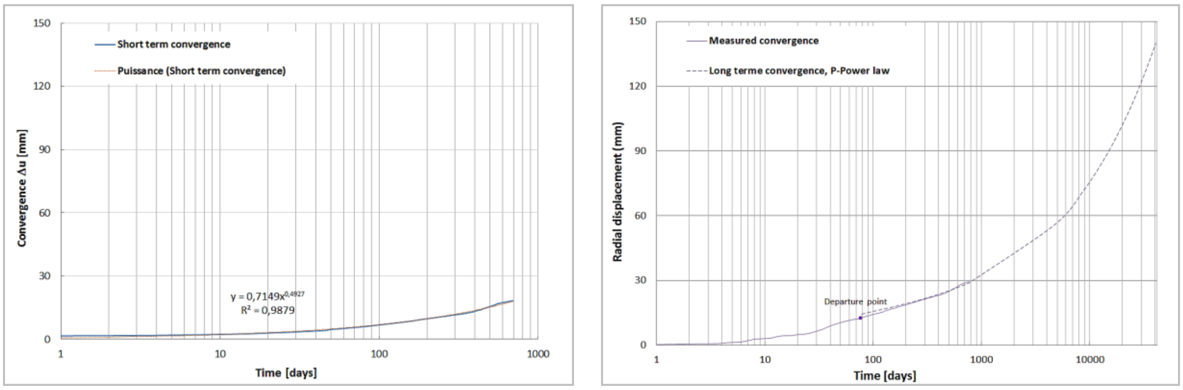
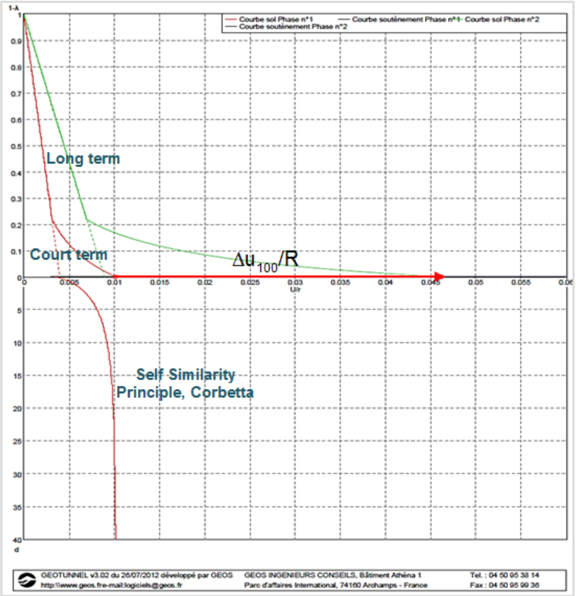


Figure 8 Fitting of P-Power law to the measured convergences and extrapolation in 100 years

Actually, the objective is to find the couples of the deformation modulus E and the residual cohesion Cr able to reproduce the convergence at a given time as estimated from the extrapolation according to the power law. The figure below shows the followed procedure in the software. The variation of the parameters is searched in the range of (E_{LT}, C_{rST}) and (E_{ST}, C_{rLT}), where the notation "LT" is for long term and "ST" for the short term, with intermediate values, by varying both parameters simultaneously, every time in order to obtain the extrapolated convergence. Because of the anisotropic stress field, the fitting of the p-power law and the search of the couples E,Cr is made for each of the three directions shown in figure 2.



<u>Données</u>

Terrain			Phase n°1	Phase n°2
Rayon du tunnel	r	m		2.6
Contrainte isotrope dans le sol	σ_0	MPa	12.3	12.3
Module d'Young du terrain	E_t	MPa	4000	1800
Coefficient du Poisson du terrain	v_t		0.3	0.3
Cohésion du terrain	c	MPa	6.2	6.2
Angle de frottement du terrain	φ	°	17.5	17.5
Angle de dilatance	ψ	°	3.5	3.5
Cohésion résiduel	c_r	MPa	1.7	0.9

Soutènement n°1 : Béton projeté			Phase n°1	Phase n°2
Distance du front de taille	d	m	0	0
Taux de déconfinement	λ_0		0.7378	0.7378
U/R	$U_{0/R}$		0.002949	0.002949
Epaisseur du béton projeté	e	m	0	0
Module d'Young du béton projeté	E_s	MPa	0	0
Coefficient de Poisson	v_s	m	0	0
Inertie du béton	I_s	m^4	0	0
Resistance à la compression	σ_c	MPa	0	0
Palier de pression	p_s	MPa	0	0
Rigidité normale	k_n	MPa	0	0
Rigidité en flexion	k_f	MPa	0	0

Coefficient de sécurité			Phase n°1	Phase n°2
Type			E.L.S.	E.L.S.
Coefficient résitance du béton			1.5	1.5
Coefficient effort Normal			1.35	1.35

<u>Résultats</u>

Limite élastique			Phase n°1	Phase n°2
Taux de déconfinement	λ_e		0.781	0.781
Pression	p_e	MPa	2.69	2.69
Convergence relative	$U_{e/r}$		0.00312	0.00694
Déplacement radial	U_e	m	0.00812	0.018

Etat final sans soutènement			Phase n°1	Phase n°2
Rayon plastique	$r_{p/r\,fin}$		1.6	2.16
Déplacement final	$U_{p/r\,fin}$		0.0102	0.0462

Équilibre terrain-soutènement			Phase n°1	Phase n°2
Taux de déconfinement équilibre	λ_e		1	1
Pression moyenne équilibre	p_e	MPa	0	0
Convergence équilibre	$U_{e/R}$		0.0102	0.0462
Déplacement équilibre	U_e	m	0.0264	0.12
Effort normal	N	MN	0	0
Effort normal sout. n°1	N_s	MN	-1.#J	-1.#J
Contr. admissible sout. n°1	σ_{ad}	MPa	-1.#J	-1.#J

Figure 9 GEOTUNNEL outcome representing the extrapolated convergences

The simultaneous reduction of the deformation modulus *E* and the residual cohesion *Cr* simulates a viscoplastic behaviour and is linked with an expansion of the plastic domain within the rock mass. On the other hand, the reduction of the deformation modulus only simulates mainly a viscoelastic behaviour of the rock mass

As it is shown in the picture below, for the same convergence at the long term and the same support confinement curve, the point of equilibrium between the ground and the structure is obtained for higher pressures as the couple (E,Cr) tends to (E_{LT}, Cr_{ST}) i.e. in case of creep of the viscoelastic type.

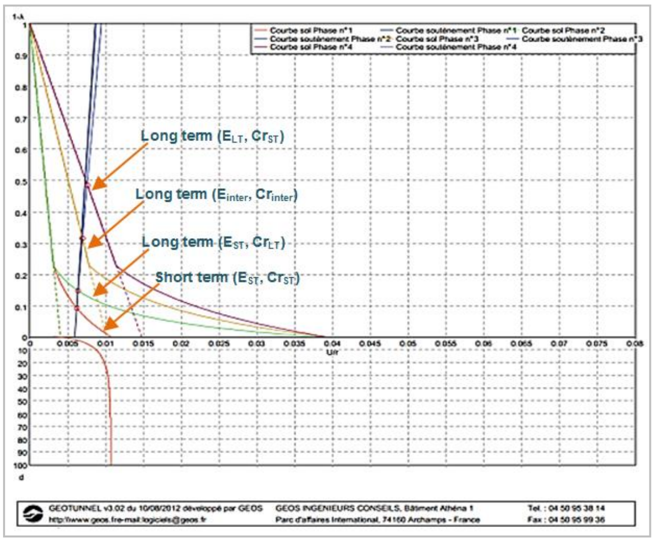


Figure 10 Ground reaction curves representing the long term behaviour of the rock for different couples (E_{LT}, Cr_{LT})

2.4 Support Confinement Curve

The SCC is the graphical representation of the relationship between the pressure that develops gradually on the structural ring until the point of equilibrium be reached. It depends on

- the displacement at the time of support installation;
- the rigidity of the structure;
- the available strength of the system

Due to the anisotropic initial stress field, the design of the structure should take into account the coupling of the bending moment with the normal force resulting from the reaction of the ground to the ovalisation of the ring. The solution implemented into the software is based on the formulation from Panet [3] for the coupling of bending moment with the normal force, via the two rigidities, normal and flexural.

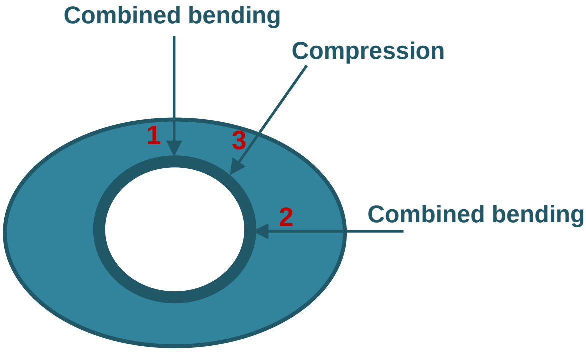
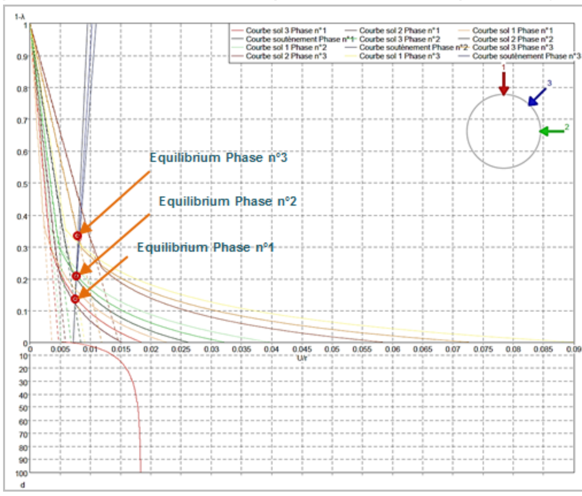


Figure 11 Ring's ovalization due to the anisotropic stress field

The applied efforts on the ring are function of its convergence, resulting from the equilibrium between the massif and the support. The software is capable to simulate the installation of a structural lining at the long term, even if prefabricated segments are already placed, as temporary support.

Although the average normal force, resulting from the equilibrium between the ground and the structure, could be calculated at the point 3, maximum bending moments depend on the differential displacement between the points 1 & 2. It is assumed that those displacements are imposed by the coupled GR Curves at the points 1 & 2 respectively.



Soutènement n°1 : Voussoirs			*Phase n°1*	*Phase n°2*	*Phase n°3*
Distance du front de taille	d	m	0.6396	0.6396	0.6396
Taux de déconfinement	λ_s		0.85	0.85	0.85
U/R	U_{sm}		0.007134	0.007134	0.007134
Epaisseur du béton projeté	e	m	0.65	0.65	0.65
Module d'Young	E_b	MPa	36000	21400	16000
Coefficient de Poisson	ν_b	m	0.2	0.2	0.2
Inertie du béton	I_b	m^4	0.0228	0.0228	0.0228
Nombre de voussoirs/anneaux	j		7	7	7
Coeffcient réducteur aux joints	α		0.8	0.8	0.8
Resistance à la compression	σ_e	MPa	45	45	45
Palier de pression	p_b	MPa	0	0	0
Rigidité normale	k_n	MPa	6.04e+03	3.59e+03	2.68e+03
Rigidité en flexion	k_f	MPa	7.9	4.69	3.51
Soutènement n°2 : Béton projeté			*Phase n°1*	*Phase n°2*	*Phase n°3*
Distance du front de taille	d	m	0.2843	0.2843	0.2843
Taux de déconfinement	λ_s		0.854	0.854	0.854
U/R	U_{sm}		0.09	0.09	0.007751
Epaisseur du béton projeté	e	m	0	0.5	0.5
Module d'Young du béton projeté	E_b	MPa	0	36000	15500
Coefficient de Poisson	ν_b	m	0	0.2	0.2
Inertie du béton	I_b	m^4	0	0.0104	0.0104
Resistance à la compression	σ_e	MPa	0	45	45
Palier de pression	p_b	MPa	0	0	0
Rigidité normale	k_n	MPa	0	4.53e+03	1.95e+03
Rigidité en flexion	k_f	MPa	0	4.29	1.85

Coefficient de sécurité			*Phase n°1*	*Phase n°2*	*Phase n°3*
Type			E.L.S.	E.L.S.	E.L.S.
Coefficient résitance du béton			1.67	1.67	1.67
Coefficient effort Normal			1.35	1.35	1.35

Figure 12 GEOTUNNEL outcome for a design where the provisional support is made of prefabricated segments. A deferred lining structure is being placed after the end of the excavation

Finally, it should be noted that the implemented approach takes into account the age of the concrete via the decrease of its deformation modulus due to creep effects, simulated by the slope change of the SCC. Moreover, a decrease of the flexural rigidity for non-continuous shells as it is proposed by A. Muir-Wood [4], is also implemented.

2.4.1 Longitudinal Deformation Profile

The most delicate point concerning the design of the tunnel structures is the definition of the ground convergence at the time of their installation. The complexity of the problem is related both to the softening and to the creep of the rock mass. The figure 10 represents the influence of the softening on the LDP.

A comparison between a Finite Difference Model realized with FLAC by ITASCA Inc. and the LDP from Corbetta [1] implemented in GEOTUNNEL illustrates that the curve resulting from the FDM is framed between the curves resulting from the approximate analytical approach, taking in account or not a softening behaviour.

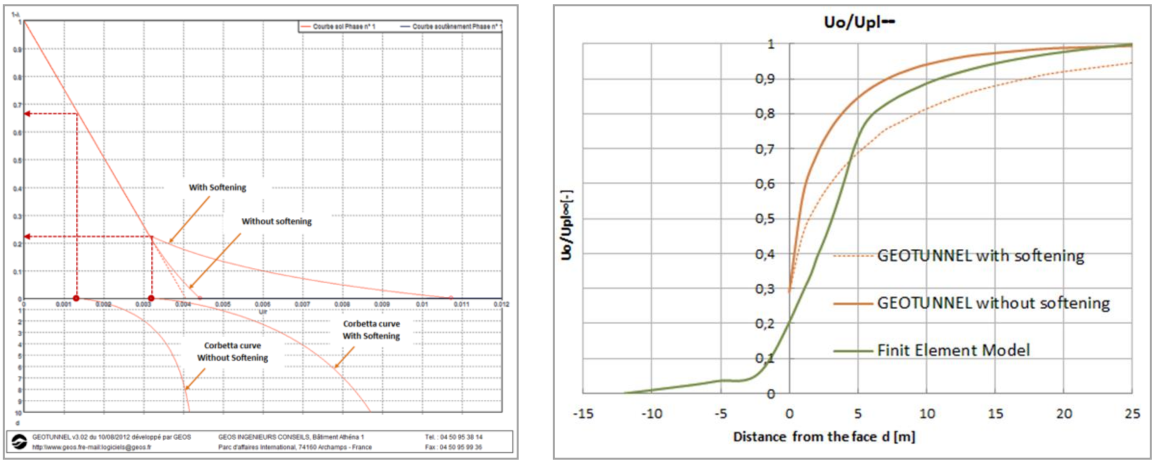


Figure 13 Convergence versus distance from the excavation face with and without softening

As described above, the creep effects in the analytical approach implemented in GEOTUNNEL © are represented via a decrease of the deformation modulus, and of the residual cohesion. The main issue of the last comparison is that the time dependent behaviour should be considered to apply the LDP curve in order to determine the initial convergence before placing of the support. As a practical result, the only way to take into account the long term effects of the creep is by an adjustment of the fictive internal pressure at the time that the structure is placed, as if the delayed behaviour would induce an additional pressure to the 3D effect of the excavation face.

3 Conclusion

In order to be applied to the 1st design stage of the project CIGEO for the french underground nuclear wastes repository, the present article described the approximate generalization of the convergence confinement method in an anisotropic initial stress field by taking into account at the time of the creep phenomena associated with the softening of the rock mass. It could be shown that an approximate analytical solution for the elasto-plastic equilibrium based on a closed-form implicit solution from Detournay & Fairhurst, could well approached a complete numerical model in anisotropic stresses condition and that an empirical law of the time dependent reduction of the modulus and cohesion, simulates correctly a creep behaviour when adjustment of the Longitudinal Displacement Profile was taking in account.

4 Acknowledgements

We are grateful to the members of the ANDRA team in charge of the CIGEO project, for their constructive comments and for the high quality data provided from the underground laboratory of Bure (CMHM) and for the permission to use these data to this development.

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