Analysis of soft soil-structure interaction in tunnelling

Ardie Purwodihardjo, Sylviane Bernat & Bernard Cambou
Laboratoire de Tribologie et Dynamique des Systèmes, Ecole Centrale de Lyon - France

Pascal Dubois
Ministère de l'Equipement, Mission d'inspection spécialisée des ouvrages d'art - France

ABSTRACT: Numerical analysis by using the finite difference method was implemented with the aim of developing a procedure for forecasting the effects induced by tunnelling. Various parameters regarding to the soil characteristics, rigidity of the lining and pressures on the working face are analyzed in this study. Two kinds of numerical calculations have been used, i.e. axisymmetric analysis and plane strain analysis for the convergence-confinement method. The constitutive model used for the ground is an elasto-plastic model with the Tresca criterion. The main objective of this paper is to highlight the limits of 2D plane strain calculations when a rigid lining and a reinforcement of the working face of the tunnel are used.

1 INTRODUCTION

Underground transportation is one of the best alternatives for solving mass rapid transportation problems in very large cities. In urban areas, the tunnels for the underground railways are usually dug in a superficial zone which can exhibit poor geotechnical characteristics. In this case the design of the tunnel requires control of ground deformations, both to limit surface subsidence and to optimise the supports.

If one hasn't the option to use a shield tunnel boring machine, but a conventional method, it is often necessary to apply the rigid lining close to the working face, even in front of the working face (pre-supporting) and to reinforce the working face by fibreglass rockbolts. The objective of this paper is to highlight the limits of an approach using a calculation in plane strain analysis in such a case, and this while being based on an example of analysis of the principal parameters of the problem: resistance of the ground, rigidity of the lining, last lining distance from the working face, pressures to the working face simulating the effect of bolts.

Two kinds of numerical calculation have been considered:

1. axisymmetric analysis
2. plane strain analysis using the deconfinement ratio (stress release ratio) at the lining installation obtained from the previous analysis

2 INPUT DATA

In this study, basically, we employed the data obtained from the Tartaiguille Tunnel, which is located on the high speed railway line between Valence and Montélimar, in France. But we used a lower modulus of the soil and a large range of ground strengths in the parametric study.

Early works on face stability were concerned with tunnels constructed in clays (Broms and Bennermark,1967; Peck,1969). They allowed a stability criterion to be established, on the basis of the consideration of an overload factor, N:

$$N = \frac{(\sigma_0 - \sigma_f)}{c_u} \qquad (1)$$

where:

N = stability factor
σ_0 = initial stress
σ_f = pressure on the working face of the tunnel
c_u = undrained shear strength of the soil

Pressures on the working face of the tunnel are used to simulate the effect of reinforcement by bolting (Dias, 1999).

We used values of $\frac{\sigma_f}{\sigma_o}$ as a function of N and $\frac{c_u}{\sigma_o}$ which can be seen in Table 1.

The other parameters used in the input data can be seen in Table 2. *D* is the external diameter of the tunnel, *d* is the distance between the end of the lining and the working face after the excavation. The different values of *d/D* used in this study are summarized in Figure 1.

The values of *d/D* bigger than 0.1 are introduced to simulate the case of an invert installed further than the lining from the working face.

Table 1. Value of $\dfrac{\sigma_f}{\sigma_o}$

$\dfrac{c_u}{\sigma_0}$	N = 2	N = 3	N = 4	N = 5
0.1			0.6	0.5
0.2		0.4	0.2	0.0
0.3	0.4	0.1		
0.4	0.2			

Table 2. Parameters used in the input data

ITEMS	Values
External diameter of the tunnel	D = 15.0 m
Initial stress of the soils in the vicinity of the tunnel	σ_o = 2.0 MPa
Elastic Young's modulus of the Soils	Es = 200 MPa
Poisson's ratio of the soils	υ = 0.49
Thickness of the lining	e = 300 mm
Elastic Young's modulus of the lining:	
• shotcrete	Ec = 3.0 GPa
• precast concrete	Ec = 30 GPa
Poisson's ratio of the lining	υc = 0.25

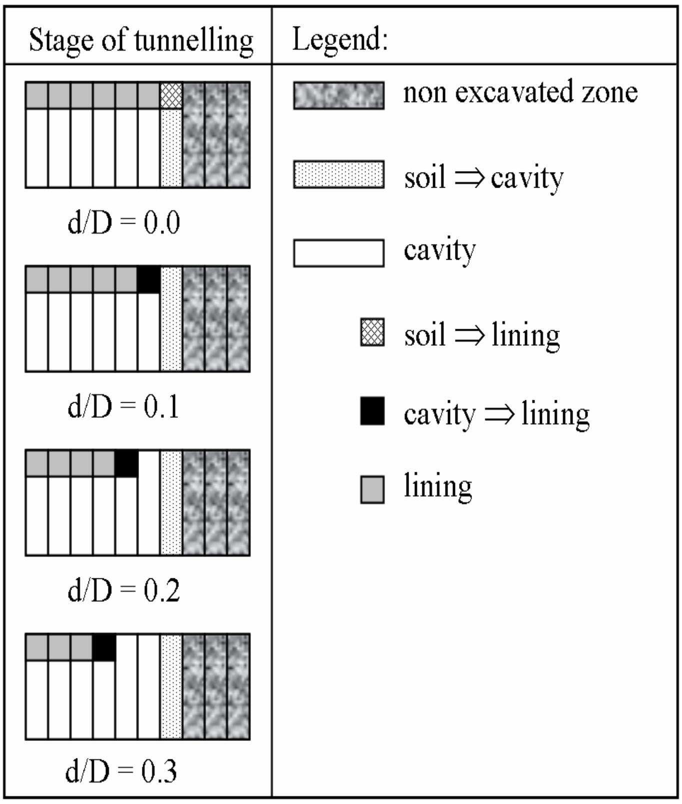


Figure 1. Sequential excavation method (SEM) in axisymmetric condition with different cases of d/D

3 COMPUTATION METHODS

3.1 *Axisymmetric calculations*

This computation is done by using the sequential excavation method (SEM) in the axisymmetric condition. The sequences for applying SEM are summarized as follows:

1. Apply initial isotropic stress conditions and deactivate all lining elements.
2. Excavate (remove) one row of ground elements along a designated length (*p*).
3. Apply pressures on the working face of the tunnel. At the same time, active one row of lining elements at the particular distance from the face of the tunnel.
4. Repeat steps 2, 3 and 4 until the excavation is finished. It can be noted that at every step of the excavation, old pressures at the previous face of the tunnel should be removed.

Figure 1. shows the application of SEM in a stage of the advancement of tunnelling with different cases of d/D. In this study the designated length used (*p/D*) for the advancement of tunnelling is 0.1.

3.2 *Two dimensional plane strain calculations*

We derived the convergence curve of the unsupported tunnel by using the plane-strain model. Figure 2. shows the principal assumption in this method (Panet et al, 1982, AFTES, 2002). The zero value of the radial stress represents the condition along the tunnel wall excavation, with no support, the tunnel being assumed to be of infinite length. The stress applied on the boundary of the tunnel near the front face can be defined as $(1-\lambda)\sigma_o$. The initial state far enough ahead of the front face corresponds to λ=0.

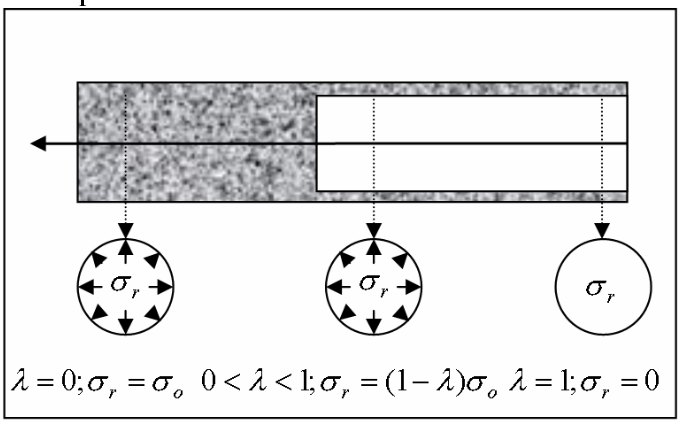


Figure 2. Virtual support pressures

The definition of λ which is needed in such 2D calculations, is usually given by an empirical expression derived from an elastic calculation or an elasto-plastic calculation in axisymmetric condition which only takes into account the distance from the working face. Different authors have proposed methods of calculation which can take into account the influence of the last lining distance from the working face and the rigidity of the lining (Bernaud & Rousset, 1996; Nguyen Minh Duc & Guo, 1993)

In this paper we propose to use the results obtained from the axisymmetric calculation to derive the value of λ at a given stage of the work. In particular, to analyse the interaction between the soil and the support, we use the value of λ_i (in a 2-D calculation) when the lining is installed. This value is determined by using the initial radial displacement of the ground (U_i) while loads begin to be applied to the lining, where U_i is defined at a distance y_i from the working face of the tunnel as:

$$y_i = d - \frac{p}{2} \qquad (2)$$

where:

y_i = mean length of the non-supported zone

d = the last lining distance from the working face

p = the advancement of tunnelling

This approach can be done by using the curve giving the convergence all along the tunnel obtained by the axisymmetric calculation, as shown in Figure 3. Furthermore the stabilized deformation and the corresponding λ value for the axisymmetric calculation are defined at distance y_s from the working face, where y_s is equal to 4 D.

In this study we assume that the shape of the lining is circular with a constant thickness.

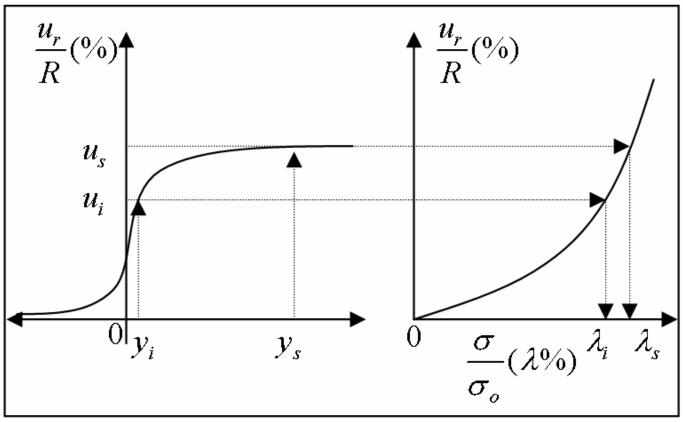


Figure 3. Virtual support pressures

4 RESULTS

Among the results, we show those corresponding to N=3 and N=5, because the other two results (N=2 and N=4) are similar to the results of N=3 and N=5.

Figures 4 and 5 show the relations between the initial value of U (U_i) or λ (λ_i) and the distance between the installation point of the lining and the working face, for different values of the other parameters (c_u, and N). We remind that N depends on c_u and σ_f (Table 1). As mentioned in paragraph 3 the value of U_i has been obtained from the axisymmetric calculation and the value of λ_i is derived from U_i using the convergence curve of the tunnel. These figures point out that for a same distance from the working face the value of λ_i depends on the various parameters: plastic characteristic of the soil in the vicinity of the tunnel, stiffness of the lining and pressures applied to the working face.

When the lining is installed close to the working face (d/D = 0.0 or 0.1) the values of U_i/R are quite similar for a same value of the load factor N.

Figure 4 shows that when the lining is installed relatively far from the working face (d/D = 0.2; d/D = 0.3) U_i/R is significantly greater for small value c_u (c_u=0.1 σ_o). Figure 5 shows that the value of λ_i depends significantly on the applied pressure (σ_f) because for small value of d/D (0.0 or 0.1) for the same value of c_u and different values of N, we can observe significantly different value of λ_i. For example, compare the case (N = 3, c_u = 0.2 σ_o , σ_f = 0.4σ_o) with the case (N = 5, c_u = 0.2 σ_o , σ_f = 0).

One can also notice that the role of the stiffness of the lining on the values of U_i and λ_i is quite significant in the same cases (d/D = 0.0 or 0.1). For higher values of d/D and for soils with poor characteristics (c_u=0.1σ_o), the reinforcement of the working face cannot prevent the occurrence of large deformations behind the face. That gives information on the role played by an invert installed according to its distance from the face. On this point one can also refer to calculations made by Dias (1999) with a 3D model. Of course one has to keep in mind that deformations larger than 2% (and often 1%) cannot be accepted in most cases. The present study has been deliberately enlarged to point out clearly the influence of the various parameters

Figures 6 to 9 show the convergence curve of the unsupported tunnel and the characteristic curve of the lining loading obtained from the initial value of U (U_i) computed as explained in paragraph 3. and shown on Figures 4 and 5. In the classical two dimensional plane strain analysis the final state is located at the intersection between these two curves. On these figures, the final state obtained directly from an axisymmetric calculation is also drawn. It can be noted that the two final balance states are very close except in the cases of larger distances between the face and the lining and of very rigid linings.

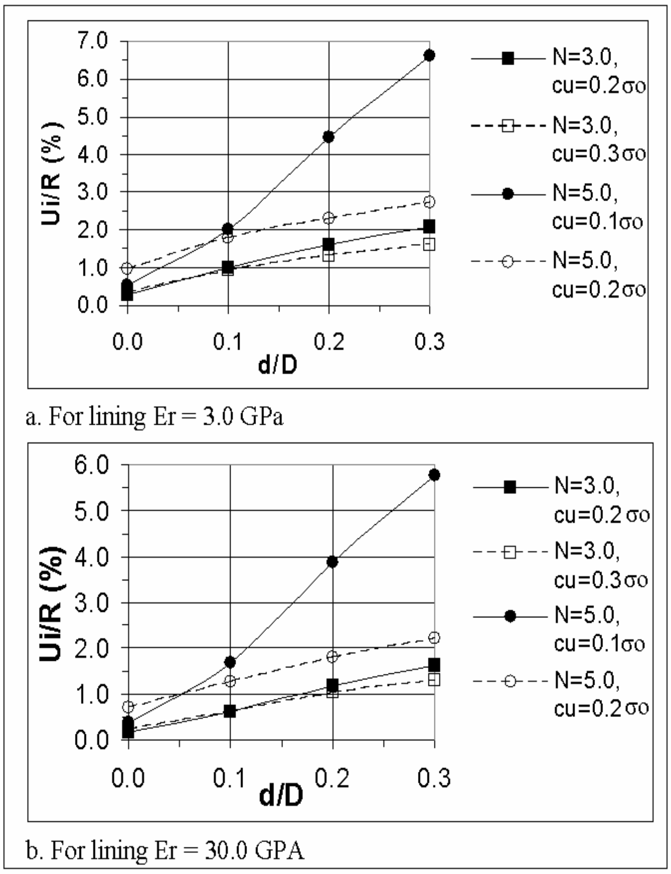
It can also be noticed that the results obtained from an axisymmetric calculation always give lower values for the final deconfinement rate than the 2D calculation, the difference can reach 10%. On the other hand, the strains obtained in the two calculations are very similar in every case.

Figure 10 shows convergence-confinement curves and axisymmetric calculation results for different values of σ_f. We can clearly see in this figure that the pressures applied on the working face play a quite significant role in the final deformations of the tunnel. The results are given for d/D=0.1, the other cases displayed the same phenomenon.

We do not discuss here how to choose the pressure to be applied on the working face according to the level of reinforcement by bolting. Specifications on this subject can be found in (Clouterre, 2002).

5 CONCLUSION

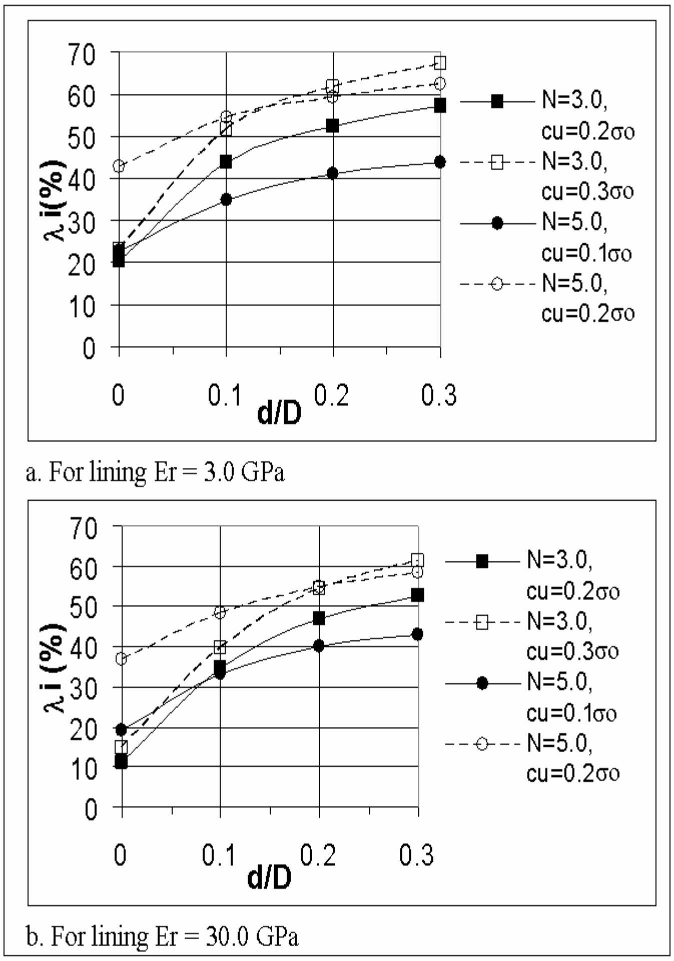
The proposed analysis shows that the plane strain calculations, like by using the convergence-confinement method, give results in good agreement with axisymmetric calculations even in the case where a significant pressure is applied to the working face of the tunnel to simulate the reinforcement of this face, subject to an appropriate choice for the stress release coefficient (λ_i) used while placing the lining. As expected, that is less true if the lining is placed at a certain distance from the face and very rigid, but this hypothesis does not fit with the strict ground motion control which is mostly required. The choice of stress release coefficient (λ_i) depends significantly on the pressure on the working face and on the stiffness of the lining. The use of the common evaluation of λ_i , which only takes into account the distance between the installation point of the lining and the working face, does not suit. In our opinion it seems difficult to give rules for choosing the appropriate value of λ_i without carrying out very large parametric studies. These conclusions need also further calculations to be eventually generalized to the other strength criteria of the ground.



a. For lining Er = 3.0 GPa

b. For lining Er = 30.0 GPA

Figure 4. Initial deformation of the tunnel when the linings were installed



a. For lining Er = 3.0 GPa

b. For lining Er = 30.0 GPa

Figure 5. Initial value of λ of the tunnel when the linings were installed

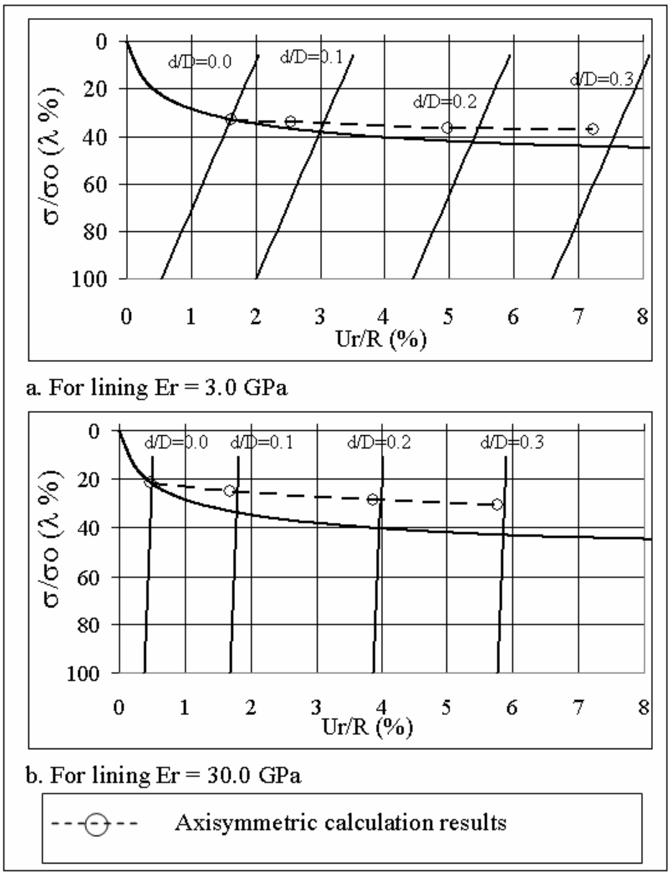


Figure 6. Convergence-confinement curves and axisymmetric calculation results for cu=0.1 σo; σ_f=0.5 σo and N=5

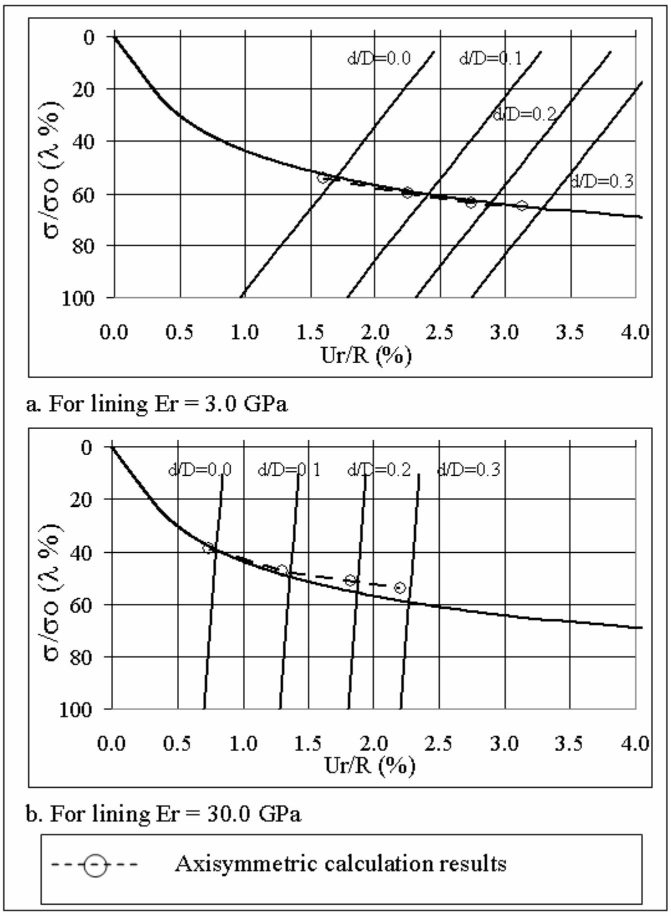


Figure 7. Convergence-confinement curves and axisymmetric calculation results for cu=0.2 σo; σ_f=0.0 σo and N=5

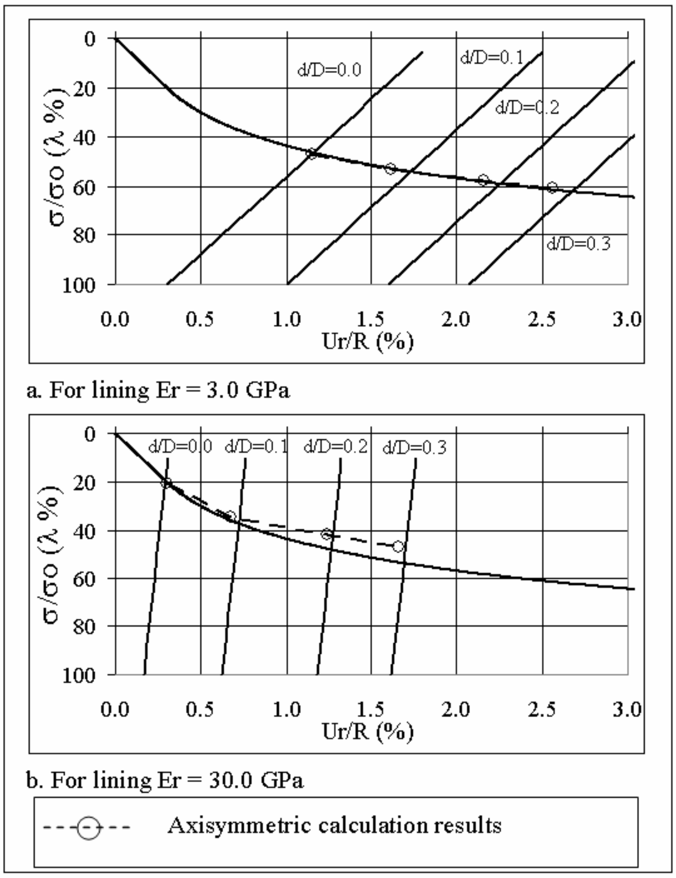


Figure 8. Convergence-confinement curves and axisymmetric calculation results for cu=0.2 σo; σ_f=0.4 σo and N=3

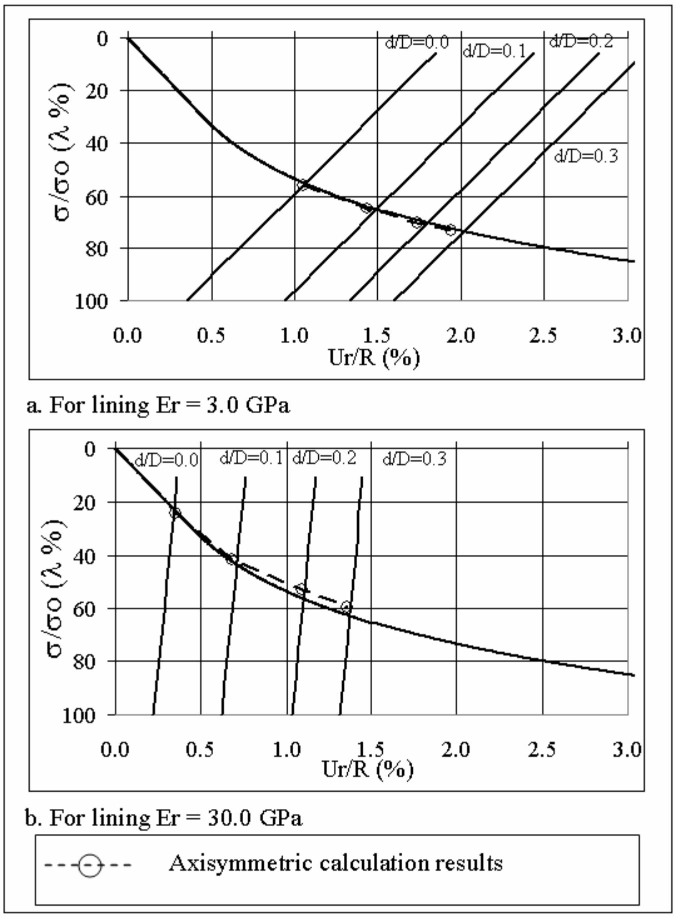


Figure 9. Convergence-confinement curves and axisymmetric calculation results for cu=0.3 σo; σ_f=0.1 σo and N=3

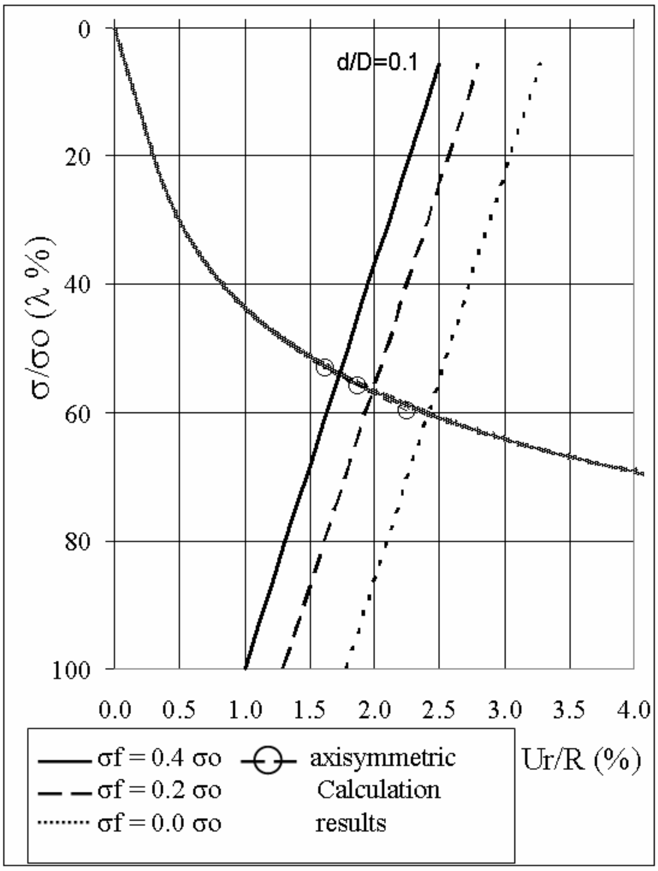


Figure 10. Convergence-confinement curves and axisymmetric calculation results for Cu = 0.2 σ_o, Er = 3.0 GPa and with different values of σ_f

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APPENDIX I. NOTATION

The following symbols were used :

N = stability factor

σ_0 = initial stress

σ_f = pressure on the working face of the tunnel

c_u = undrained shear strength of the soil

y_i = mean length of the non-supported zone

d = the last lining distance from the working face

p = the advancement of tunnelling