AFTES

RECOMMENDATIONS ON

THE CONVERGENCE-CONFINEMENT METHOD

AFTES welcomes comments on this paper

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Notation

Geometry

R radius of circular tunnel section
d unsupported tunnel length from working face
x distance from working face to any given tunnel cross section

Time

t time after working face has passed any given tunnel cross section
T_A characteristic time of rate of advance of working face
T_M time characterising the rate of time-dependent deformation of the ground
T_S time characterising creep behaviour of support

Displacement, Convergence

u, u_R, radial displacement of a point on tunnel wall
u_0 value of u at working face
u_d value of u at distance d from working face
u_∞ value of u at very long distance from working face
u_{ns} value of u for unsupported tunnel
C(x) convergence at section at distance x from working face

Speed

V_A rate of advance of working face

Stresses

σ, σ_R radial stress on tunnel wall massif
σ_0 natural stress tensor, natural stress value with hydrostatic tensor

Ground properties

E Young's modulus of ground
n Poisson's ratio of ground
G shear modulus of ground
σ_c uniaxial compressive strength

Support properties

E_S Young's modulus of support
v_S Poisson's ratio of support
K_S stiffness of support
K_{SN} normal stiffness of support
K_{SF} bending stiffness of support
p_S support pressure

Decompression

λ confinement loss
λ_0 confinement loss at working face
λ_d confinement loss at distance d from working face
λ_e confinement loss at boundary of elastic zone

SOMMAIRE

1 - INTRODUCTION

Designing tunnel support was for many years considered too complex for strict engineering analysis and remained an empirical art repeating techniques that had proven satisfactory under similar geological conditions in the past. This similarity approach was based on qualitative factors that were neither well-defined nor interpreted in any consistent way.

The analytical methods which are a basic tool for construction engineers were found to be unsuitable for tunnel support design. This left the way open for dogmatic claims about the universal suitability of certain methods and techniques, claims failing to stand up to quantitative analysis.

The difficulty of designing tunnel support arises mainly from

• inadequate knowledge of ground behaviour under conditions associated with tunnel driving,

• insufficient data on the natural state of stress in the ground, and

• the fact that it is a three-dimensional problem.

The last point is due to the fact that the engineer needs to analyse the interaction between the ground and the support near the working face.

Additionally, time-dependent response dictated by the rheological properties of the ground may also have to be considered.

The convergence-confinement method is a simplified method of analysing this interaction between the ground and the support. In its basic form using extreme axisymmetry assumptions, it becomes a two- or one-dimensional problem, providing a simple understanding of the ground/support interaction processes occurring near the working face.

These Recommendations describe the general principles of the convergence-confinement method, including the rules for selecting the confinement loss value, which is the keystone of the method. They also describe the field of application of the method and its relationship to other existing methods.

Contrary to what is mistakenly assumed in the usual design methods, the ground/support interaction is not generally amenable to dealing separately with actions applied to the structure and the structure's stresses and deformations. Addressing these two factors separately is the main reason for the inadequacy of the methods previously proposed.

In fact, tunnel support design methods can be classified into four types:

• Purely empirical methods indicating the most appropriate type of support for a situation defined from various geotechnical classification systems.

• Methods for determining the loads acting on the support, regardless of support type and deformation.

• Support design methods which consider loads exerted by the ground as input data but allow for support stiffness and deformation and the reactions of the surrounding ground.

• More recent methods taking full account of the ground/support interaction.

These methods, which are the subject of earlier AFTES Recommendations [1], are briefly reviewed in the following.

1.1 - Empirical methods based on geotechnical classification systems

Various rock classification systems have been proposed. The most widely used are Bieniawski's RMR [7] and N Barton's Q system [3]. They attribute a lumped score to the rock based on several quantified parameters. The overall score determines support type.

This approach must not be confused with a different use of the RMR and Q index for determining the geomechanical properties of rock (Hoek & Brown) [14].

AFTES has proposed a method entitled "Tunnel Support Type Selection" described in interim Recommendations in the September 1976 special issue of *Tunnels et Ouvrages Souterrains*. The parameters used for support selection are rock strength as measured on laboratory specimens, fragmentation (RQD index) and weathering, hydrogeology, tunnel cover and cross section, and tunnel environment. Another important feature is that the rock strength classification is to some degree dependent on tunnel driving method (presplitting or mechanical excavation). A matrix identifies, for a given situation, unsuitable support methods, feasible methods and appropriate methods. Unlike the RMR and Q classifications, there is no information on unsupported length, steel rib inertia and spacing, rock bolt density and bolt length, or shotcrete thickness.

These methods had the merit of introducing the need for a quantitative description of the ground. But since they are based on case histories whose relevance has not been addressed, they tend to repeat past mistakes. They are in standard use, mainly abroad, and can be useful aids at the project planning stage.

1.2 - Methods giving loads exerted by the ground on the support

These methods determine the extent of the failure zone. Purely static considerations then determine the reaction that needs to be exerted by the support to keep the failure zone stable.

These methods implicitly assume that severe convergence has occurred for failure mechanisms to occur; the corresponding displacements are not necessarily acceptable for the support structure.

Methods differ in the way they define the failure zone.

In the methods recommended by Terzaghi (fig. 1) and Protodiakonof, the shape of the failure zone is given. Its extent depends on the mechanical strength of the ground and tunnel cover.

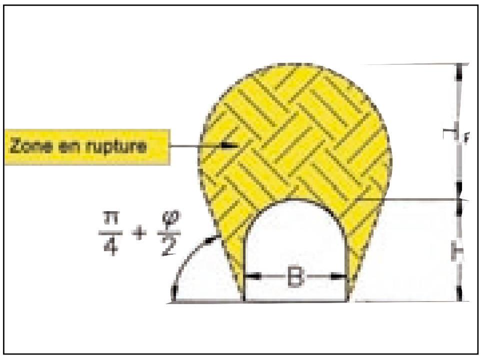

Figure 1 - Definition of failure zone above tunnel crown according to K Terzaghi

Caquot [8] defines the failure zone by analysing the extent of the plastic zone around a circular tunnel.

These methods are justified when the failure mechanism considered is independent of the support method.

This may be the case with a shallow tunnel where there is a possibility of a sinkhole appearing at ground level.

It also concerns the case of rock tunnels where the stresses around the opening are well below the uniaxial compressive strength of the rock and the deformations caused by the tunnelling are very slight. The only significant displacements that can occur are due to pre-existing discontinuities opening or causing blocks to slide. Three-dimensional structural analysis will reveal joint sets of constant direction, and allows the engineer to determine the maximum dimensions of blocks liable to be unstable for the actual size and direction of the tunnel excavation. Here, the support merely serves to prevent potentially unstable blocks from falling or sliding. Methods based on structural analysis are appropriate in such situations (fig. 2).

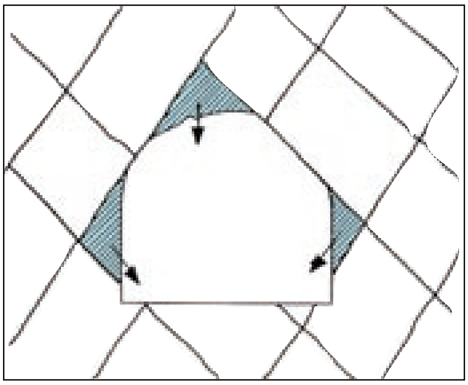

Figure 2 - Unstable rock blocks intersected by the tunnel

1.3 - Methods of analysing support exposed to predetermined loads

These methods deriving from conventional structural design based on strength of materials theory are the normal complement to the methods described above, and the same remarks apply.

However, what is known as the "hyperstatic reactions" method deserves special mention. The support is modelled as bars and the ground reaction, as springs (fig. 3). It is an attempt to address the interaction between the ground and support. The load on the support comes from the actions needed to maintain the failure zone in equilibrium and the reaction of the ground to yielding of the support.

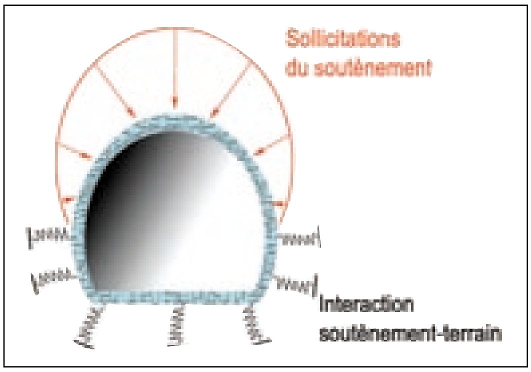

Figure 3 - Principle of hyperstatic reactions method

This method has frequently been used, despite the structural engineer's perplexity as to the distinction between the zone exposed to ground actions and the zones exposed to ground reactions to support displacements. Setting spring stiffness is critical. Results may even become very surprising when heading and benching. The method is still frequently used and may be useful, especially for shallow tunnels, because the model is simple to use.

1.4 - Methods addressing ground/support interactions

Strict engineering analysis of the ground/support interaction is feasible by three-dimensional computer modelling by various methods, lumped together under the name "composite solid methods." They use a finite element or finite difference approach, or separate elements. This type of modelling may include for

- support structure and geometry with constitutive equations for this structure,
- the geometry of the various geomechanical units identified around the tunnel with their constitutive equations, and
- unnel excavation phases and support installation.

The advantages of this type of three-dimensional modelling are incontrovertible and they will probably become commonplace in future with the inexorable progress in computer methods with in which young engineers are increasingly familiar. At present, their use and interpretation is still considered slow and complex and involves difficulties in running sensitivity analyses on parameters whose determination involves much uncertainty, especially geotechnical parameters.

Somewhat paradoxically, these models are mainly used for very complex underground openings where it is difficult to assess the true influence of the simplifications needed to make them more like more routine design problems. The LCH1 cavern currently being built for CERN is a good example of three-dimensional modelling of a highly complex underground ensemble of large, intersecting chambers and shafts (fig. 4).

The convergence-confinement method does away with the need for a complex three-dimensional model. It is based on the two-dimensional analysis of the interaction between the support and the ground. It is therefore much simpler. This type of analysis has been proposed by several authors, the first probably being Fenner [11] in 1938. The same approach was adopted by Pacher [20] in 1964. Their main shortcoming was that they failed to consider ground deformations that occur before the support is installed. With his characteristic "core" line, Lombardi introduced the concept of convergence at the working face [17]. Panet & Guellec [21, 1974] advocated including for deformations occurring prior to support installation via the confinement loss factor. This is the origin of the convergence-confinement method which was given its name at a1978 AFTES meeting in Paris.

The principal advantage of the method is that it allies ease of use with an objective approach to the more important processes involved in the ground/support interaction. It allows easy analysis of the sensitivity of parameters which can only be quantified approximately.

The first AFTES Recommendations on the convergence-confinement method were issued in 1984 [1]. Abundant research findings since that time, along with much observational data, have made it necessary to re-write the older Recommendations.

2 - TUNNEL CONVERGENCE

The loss of confinement caused by tunnel driving causes stress redistribution around the excavation and deformations. Convergence of the tunnel along line *a* is the relative displacement of a diametrically-opposed pair of points on the tunnel wall on this line as the working face advances.

Convergence depends on the distance *x* between the instrumented section and the working face, on elapsed time *t* after the working face has passed the instrumented section, on the unsupported distance *d* behind the working face, and on the stiffness of the support; this can be written in general terms:

$$C = C\,[x(t)t, d, K_S\,]$$

Convergence measurements are usually plotted versus distance to working face and time (fig. 5).

Detailed analysis of the curves yields very instructive information about the influence distance of the working face, and therefore, about the extent of the decompressed zone and whether or not time-dependent deformations will occur. It also enables a judgement to be made as to the validity of the analytical models.

Standard tunnel instrumentation methods give the convergence behind the working face but no data on the convergence which occurs ahead of it (now called 'preconvergence'). New design and construction approaches for tunnels in difficult ground provide a more refined analysis of the behaviour of the ground ahead of the working face (cf. ADECO-RS method developed by P Lunardi [18, 19]). Since it is not possible to measure preconvergence directly, he proposes measuring extrusion of the ground ahead of the face, i.e. the displacement of points on the tunnel centreline ahead of the face as the working face advances. Much can be learned from the amplitude of, and changes in extrusion with reference to the working face, especially for installing temporary support or preconfinement.

Schematically, there are three situations:

- The working face may be stable with little extrusion at the face.
- The working face may be stable but exhibits significant extrusion, due to deformations ahead of the face.
- The working face may be unstable and collapses.

The first two situations are those for which the convergence-confinement method is ideally suited.

Its extension to the third case requires prior analysis of the methods of shoring up the working face and of the deformations ahead of the stabilised face. Such analysis is beyond the scope of these Recommendations.

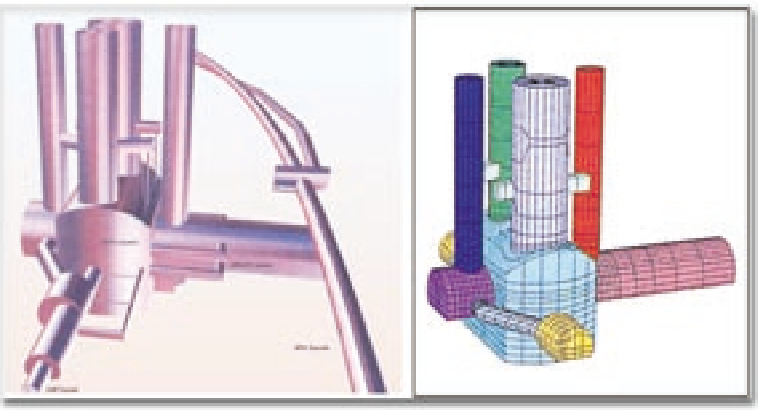

Figure 4 - Model of CERN LCH1 cavern

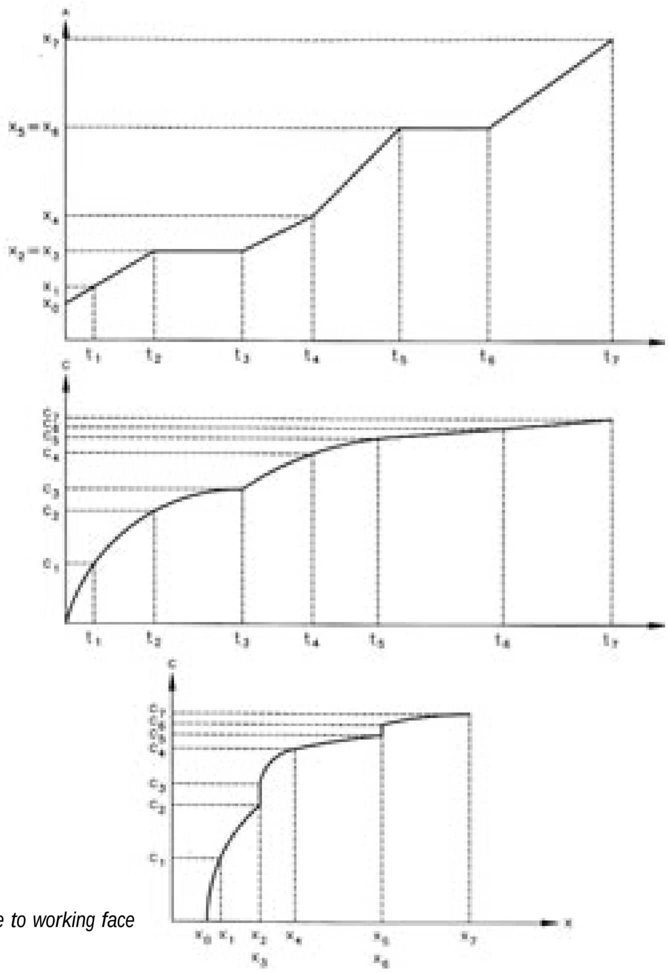

Figure 5 - Tunnel convergence vs time and distance to working face

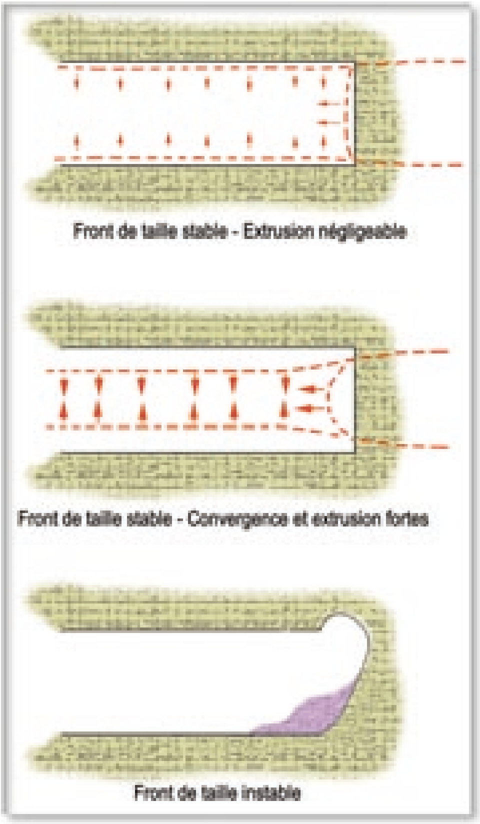

Figure 6 - Extrusion and instability of working face (aftes P. Lunardi)

3 - MECHANICAL BEHAVIOUR OF SUPPORT

Support systems oppose convergence of the tunnel walls by exerting a pressure, commonly called the support pressure. Support pressure P_S increases with support system stiffness and is limited by the strength of the support material.

With a circular tunnel of radius *R*, we define the normal stiffness modulus of the support K_{SN} as :

$$p_s = K_{SN} \frac{u_R}{R}$$

Only the linear portion of the support strain curve is considered.

With an axisymmetric tunnel, this modulus alone determines support stiffness, but under non-axisymmetric conditions, we also need the stiffness modulus in bending K_{SF}.

Typical stiffness moduli for different types of support are given below for an axisymmetric model.

3.1 - Circular ring of constant thickness e (e<<R)

$$K_{SN} = \frac{E_s}{1-\nu_s^2} \frac{e}{R}$$

$$K_{SF} = \frac{E_s}{1-\nu_s^2} \frac{I}{R^3}$$

in which

$$I = \frac{e^3}{12}$$

3.2 - Circular ring of n segments of thickness e

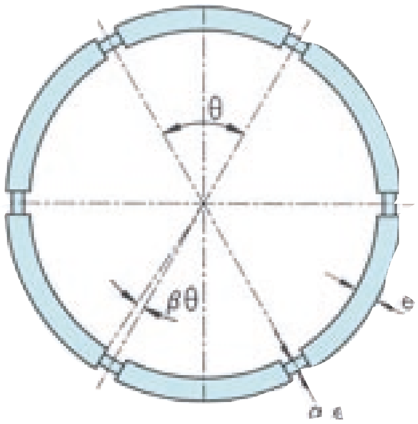

Figure 7 - Model of segmental lining ring

The above expressions are used, with

$$E_s = \frac{\alpha}{\alpha(1-\beta)+\beta} E_s$$

$$I = I_j + \left(\frac{4}{n}\right)^2 \frac{e^3}{12}$$

in which (fig. 7)

ae is the equivalent thickness at joints

θ is the angle subtended at centre by a lining segment

$\beta\theta$ is the angle subtended at centre by a joint

Ij is the inertia of the section at a joint

$$I_j = \frac{\alpha^3 e^3}{12}$$

3.3 - Circular steel ribs at spacing s in intimate contact with ground

$$K_{SN} = \frac{E_a A}{sR} \quad ; \quad K_{SF} = \frac{E_a I}{sR^3}$$

in which

A is the area of the rib cross section
E_a is the Young's modulus of steel
I is the moment of inertia of the rolled steel section.

These expressions assume the ribs are in near-continuous contact with the ground.

3.4 - Rock bolts

The following brief discussion of rock bolt support concerns only the stiffness of this type of support. The subject is dealt with much more fully in the AFTES Recommendations on rock bolting.

- Mechanically anchored rock bolts, longitudinal spacing s_l, transverse spacing s_t

$$\frac{1}{K_{SN}} = \frac{s_t s_l}{R}\left[\frac{4L}{\pi \Phi^2 E_b} + Q\right]$$

$$K_{SF} = 0$$

in which

L is bolt length

Φ is bolt diameter

E_b is the Young's modulus of the bar material

Q is a factor for deformation at the bolt head and anchorage, determined from pull-out tests.

- Rock bolts anchored over their whole length

Rock bolts anchored over their whole length and Swellex or Split Set type dowels are considered as rock reinforcing members and their effect is modelled by assuming improved geomechanical properties of the bolted zone.

In this way, thick shell theory can be applied to the bolted zone, although shell thickness must be taken as less than bolt length. "Homogenisation theory' [11] can be used to determine the mechanical properties of this ring. It is considered as an equivalent isotropic material with enhanced cohesion.

Two dimensionless parameters characterise the role of rock bolts. One describes bolt contribution to the stiffness of the homogenised zone:

$$\frac{D_b S_b E_b}{E_s}$$

The other parameter describes bolt contribution to the improved strength of the homogenised zone:

$$\frac{D_b S_b \sigma_{yb}}{\sigma_c}$$

in which D_b, S_b, E_b, s_{yb}, are respectively bolt pattern density, bolt cross section, and Young's modulus and yield point of the bolt material.

It is usually found that rock bolts contribute little support if only linear strain is considered. But in fact, bolting plays a particularly important role under high convergence conditions. This can only be understood if rock mass dilatancy is considered, especially in terms of brittle or strain softening behaviour beyond peak strength. Analytical methods which do not consider such behaviour underestimate the effect of rock bolting.

3.5 - Shotcrete

Shotcrete is widely used for tunnel support. The stiffness to be introduced in analysing ground/support interaction must be based on (i) shotcrete age and (ii) whether or not the shotcrete forms a continuous shell. The developing stiffness of green shotcrete allows it to adapt to convergence (cf. AFTES Recommendations on shotcrete).

3.6 - Stiffness of some standard support types

The following table presents typical order-of-magnitude stiffness and strain moduli for a few types of support routinely used, in a tunnel of 5m radius.

3.7 - Combinations of several types of support

In most cases, a combination of support systems is used. If all the types are installed at the same time at the same distance behind the working face, they are assumed to be exposed to the same displacement field, and stiffness moduli are the sum of the stiffness moduli of the constituent systems.

If they are not all installed at the same time, support stiffness varies with the distance to the face and this must be allowed for in the convergence-confinement analysis (fig. 8).

4 - PRINCIPLE OF CONVERGENCE-CONFINEMENT METHOD

Instead of the three-dimensional problem, the convergence-confinement method [23] addresses a two-dimensional plane strain problem of the ground/support interaction.

It consists of applying to the walls of the opening a stress

$$\sigma = (1 - \lambda)\sigma_0 \qquad (1)$$

σ_0 is the natural stress in the ground; λ is a parameter simulating the excavation as it increases from 0 to 1. It is called the *confinement loss* (fig. 9).

As this parameter decreases in value, the ground loses its confinement, and this loss of confinement causes a displacement u of the walls of the opening such that

$$f_m(\sigma, u) = 0 \qquad (2)$$

This is the *convergence equation for the ground.*

The equation for support behaviour relates the stresses exerted at the wall to the corresponding displacement:

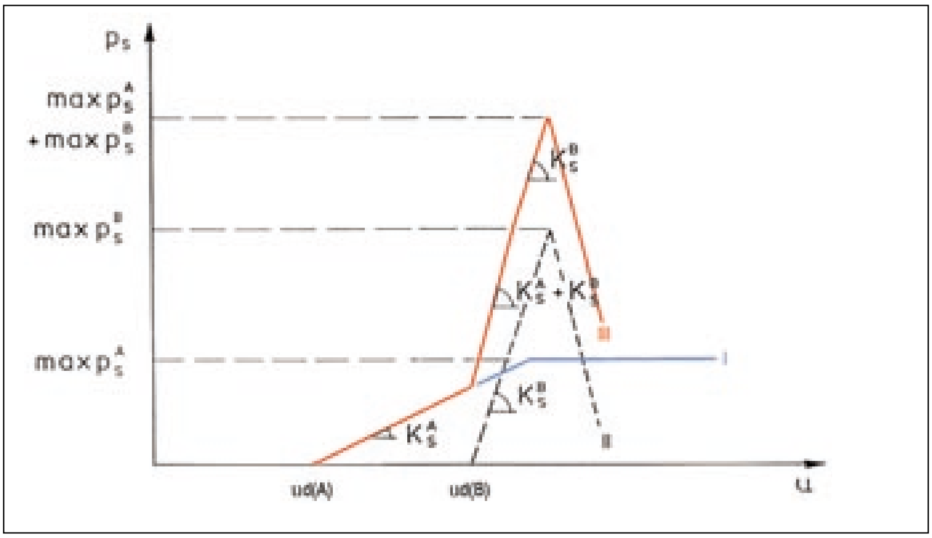

Figure 8 - Confinement curve for combination support system

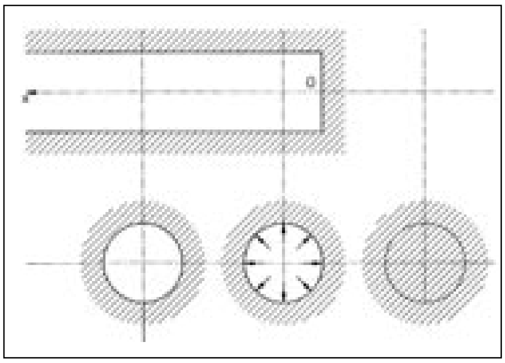

Figure 9 - Change in confinement loss on tunnel centreline

Support	40cm thick *in situ* concrete ring	ring of six 30 cm thick concrete segments	Continuous shell of 10 cm thick shotcrete	Circular HEB 140 steel ribs on 1m centres	Mechanically anchored bolts, 18mm dia. ; $s_T s_L = 1$ m²
K_{SN} (MPa)	850	750	210	150	60
K_{SF} (MPa)	0,45	0,13	$5.\ 10^{-3}$	$7.\ 10^{-3}$	0
Max u_R/R	$2{,}5.\ 10^{-3}$	$1{,}5\ .\ 10^{-3}$	$3{,}3.\ 10^{-3}$	$1\ .\ 10^{-3}$	

$$f_s(\sigma, u) = 0 \qquad (3)$$

This is the *convergence equation for the support.*

The support is usually installed some distance *d* behind the working face, called the *unsupported distance.* Displacements u_d will have occurred ahead of the face and in the unsupported zone behind it.

Displacement u_d has a corresponding confinement loss λ_d.

Therefore the preceding equation can be written

$$f[\sigma, (u - u_d)] = 0 \qquad (4).$$

The equilibrium which eventually results from the interaction between the ground and the support is found by solving the system of equations (2) and (4).

The confinement loss concept is the key to the method and determining its value λ_d at the time the support is installed is the main challenge.

It should be mentioned that other methods have been proposed to take account of the proximity of the working face limiting convergence at the time of support installation. Svoboda suggests simulating this effect by progressive softening of the ground waiting to be excavated. But these methods have been found to be less practical than the confinement loss method. He did not offer any clear rules for finding the amount of softening from unsupported distance and ground behaviour. Setting it at 90% for deep tunnels and 30% for shallow tunnels appears completely arbitrary.

In the simplest case where there is complete axisymmetry about the centreline and there is no time-dependent deformation of the ground, the convergence-confinement method is amenable to very simple graphic plotting (fig. 10).

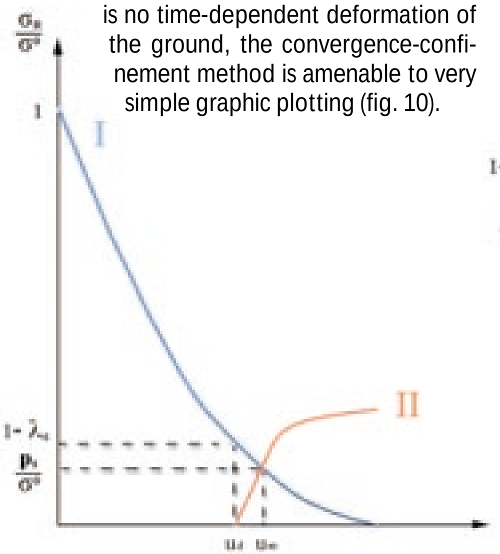

Figure 10 - CAxisymmetric case. Graphic representation of convergence-confinement method

4.1 - Axisymmetric case: linear elastic ground and support

In the simplest case, displacements and stresses are radial, equations (2) and (4) are linear and are written

$$\sigma_R + \frac{E}{1+\nu}\frac{u_R}{R} - \sigma_0 = 0$$

$$\sigma_R - K_{SN}\frac{u_R - u_d}{R} = 0$$

K_{SN} is the stiffness modulus of the support.

Solving this system determines the support pressure ps and radial displacement at equilibrium u_R:

$$p_S = \frac{K_{SN}}{2G + K_{SN}}(1 - \lambda_d)\sigma_0$$

$$\frac{u_R}{R} = \frac{2G + \lambda_d K_{SN}}{2G + K_{SN}}\frac{\sigma_0}{2G}$$

in which

$$2G = \frac{E}{1+\nu}$$

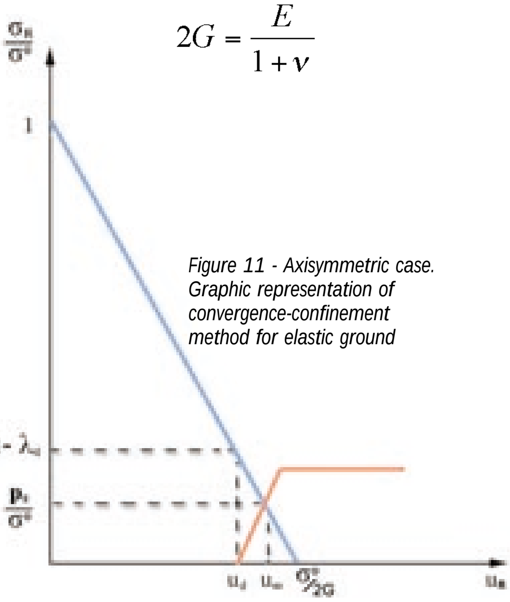

Figure 11 - Axisymmetric case. Graphic representation of convergence-confinement method for elastic ground

4.2 - Axisymmetric case: elastic-plastic ground

When confinement loss approaches unity, the boundary of the elastic response zone is reached at the tunnel wall with a confinement loss value λ_e.

If the plasticity criterion is given by an intrinsic curve whose equation is

$$F(\sigma_1, \sigma_3) = 0$$

then the value of λ_e is given by

$$F[(1+\lambda_e)\sigma_0, (1-\lambda_e)\sigma_0] = 0$$

With $\lambda > \lambda_e$, , there appears a plastic zone of radius R_p and the convergence curve of the ground ceases to be a straight line because of the plastic strains.

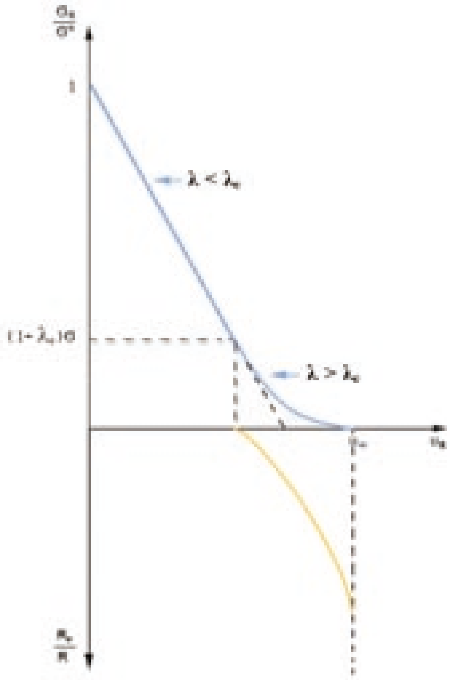

Figure 12 - Axisymmetric case. Convergence curve of ground and change in plastic radius for elastic-plastic ground

The equations for the convergence curves under axisymmetric conditions have been established by various authors for various types of elastic-plastic behaviour laws [23].

5 - DETERMINATION OF CONFINEMENT LOSS

Selecting the confinement loss λ_d corresponding to the convergence occurring before the support starts interacting with the ground is the most critical point in the convergence-confinement method.

λ_d is determined from the convergence equation:

$$f[(1 - \lambda_d)\sigma_0, u_d] = 0$$

The value of λ_d is therefore chosen by determining the radial displacement u_d at the unsupported distance d behind the working face.

High u_d displacement values correspond to higher confinement loss values as λ_d approaches unity. This parameter is governed mainly by the length of the unsupported distance behind the face *d* but it is also dependent on the constitutive equation for the ground and to a lesser extent, on the stiffness of the support.

Accuracy in calculating the support pressure is closely linked to the accuracy of determining λ_d. It is dependent on the slope of the convergence curve near its intersection with the confinement curve. The impact of uncertainty as to the precise value of λ_d on the value of the support pressure should be assessed in all cases.

The radial displacement u_d can generally be written

$$u_d = u_0 + a_d\,(u_\infty - u_0)$$

in which

a_d is given approximately by

$$a_d = 1 - \left[\frac{mR}{mR + \xi d}\right]^2$$

m and ξ are coefficients dependent on the constitutive equation for the ground.

Therefore u_0, u_∞, *m* and ξ have to be determined.

Common errors are to assign u_0 and u_∞, values for an unsupported tunnel; but then, u_∞, is not the equilibrium radial displacement for a supported tunnel and u_0 is overestimated.

What are called implicit methods using values for a supported tunnel have been developed more recently.

5.1 - Methods based on convergence in an unsupported tunnel

5.1.1 - Elastic ground behaviour

$$u_\infty = \frac{\sigma_0 R}{2G}$$

$$u_0 = \alpha_0 u_\infty$$

$\alpha_0 = 0{,}25 \quad ; \quad m = 0{,}75 \quad ; \quad \xi = 1$

Therefore

$$\lambda_d = 1 - 0{,}75\left[\frac{0{,}75R}{0{,}75R + d}\right]^2$$

It will be seen that this equation yields a confinement loss at the face (*d* = 0) of 0.25, but in fact, the true value depends on the Poisson's ratio. With 0.2<ν<0.5, it ranges more or less linearly from 0.20 to 0.3. But with *d/R* > 0.25, confinement loss is almost entirely independent of the Poisson's ratio.

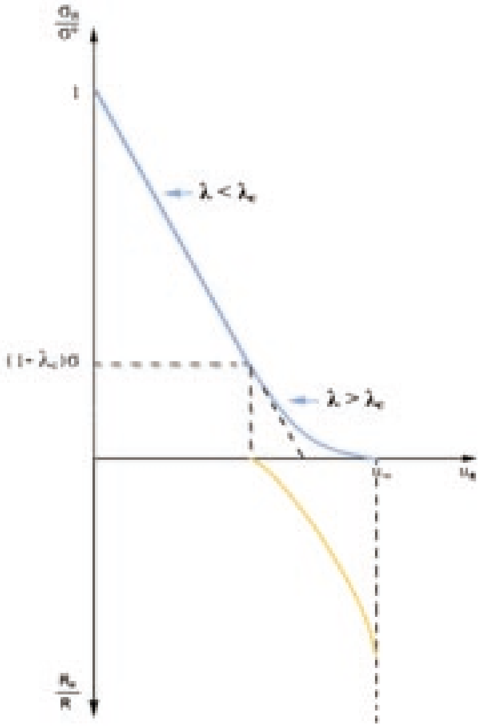

Figure 13 - Axisymmetric case. Radial displacement vs x for different Poisson's ratios.

5.1.2 - Elastic-plastic ground behaviour

The value of u_d is determined by assuming similarity with elastic ground conditions, as proposed by Bernaud, Corbetta & Nguyen Minh [5]. This approach consists of obtaining the $U_r = f(x)$ curve for elastic-plastic conditions as a homothetic transform of the corresponding elasticity curve of centre o and ratio 1/ξ (fig. 14).

The final radial displacement of the unsupported tunnel is written as

$$u_\infty = \frac{1}{\xi}\frac{\sigma_0 R}{2G}$$

Then

$$a_d = 1 - \left[\frac{0{,}75R}{0{,}75R + \xi d}\right]^2$$

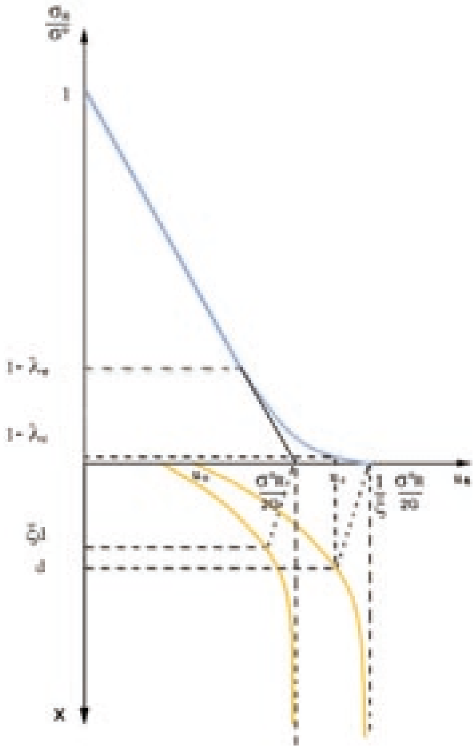

Figure 14 - Application of similarity principle (after Bernaud, Corbetta & Nguyen Minh)

From this:

$$u_d = u_\infty\left[1 - 0{,}75\left(\frac{0{,}75R}{0{,}75R + \xi d}\right)^2\right]$$

5.2 - Methods based on convergence of supported tunnel

These methods allow for the fact that the stiffness of the support limits convergence ahead of and behind the face, so that coefficient λ_d has a lower value than in the methods described above. The effect is obviously more marked when the support is stiff and installed close behind the face.

What are called 'implicit' methods have been developed by Bernaud & Rousset [6] and by Nguyen Minh & Guo [14]. These methods give similar results.

Nguyen Minh & Guo define two parameters:

$$A = 1 - \frac{u_\infty}{u_{ns\infty}}$$

and

$$B = 1 - \frac{u_d}{u_{nsd}}$$

in which $u_{ns\infty}$ and u_{nsd} are respectively values of u_∞ and u_{nsd} for an unsupported tunnel.

They thus arrive at the general equation

$$B = A\left[0{,}45 + 0{,}42A^2\right]$$

This formula and the equations for the convergence and confinement curves make it possible to determine u_∞ and u_0.

The following table shows correction factors to be applied to the λ_d value for an unsupported tunnel in elastic ground for different values of *d/R* and normal stiffness modulus of the support, referred to the shear modulus of an elastic ground:

$$k_{sn} = \frac{K_{sn}}{2G}$$

k_{SN} \ *d/R*	0,25	0,50	0,75	1	2
0,25	0,97	0,97	0,98	0,99	0,99
0,50	0,93	0,96	0,96	0,98	0,99
1	0,88	0,92	0,95	0,97	0,98
2	0,83	0,89	0,93	0,96	0,97
5	0,76	0,85	0,89	0,94	0,97
10	0,71	0,82	0,86	0,93	0,96
∞	0,69	0,81	0,85	0,92	0,96

In non-linear ground, iterative methods must be used too determine λ_d. The starting point is the value obtained for an unsupported tunnel, then proceeding to iterate for A and B until the problem converges with little change between successive steps.

5.3 - Using convergence-confinement method with two-dimensional computer models

The most common way of using the convergence-confinement method is to investigate a two-dimensional cross section of the tunnel in a finite element model reproducing the exact tunnel geometry and construction phases, with one or more confinement loss values, as discussed above.

Values for λ_d are usually found from axisymmetric models, i.e. for circular tunnels, with hydrostatic natural stresses and uniform ground. Any significant departure from these basic assumptions - tunnel cross section inscribed within a highly eccentric ellipse, highly anisotropic natural stresses, highly heterogeneous ground near the tunnel, shallow ground cover - requires that λ_d should be determined with simplified models. This approach is all the more necessary when support stiffness is high compared to ground deformation characteristics and support is installed close to the working face.

For example, when designing a large underground railway station opening, λ_d was selected on the basis of the convergence data from a three-dimensional model of the unsupported excavation and a two-dimensional model simulating the phases of excavation and support installation.

These adaptations of the method require engineering judgement and cannot be readily codified in guidelines.

6 - EXTENSION OF CONVERGENCE-CONFINEMENT METHOD TO SUPPORT AHEAD OF THE FACE

The growth of tunnelling in soil and similar difficult ground has produced techniques for shoring up the working face and controlling displacement ahead of it.

Slurry and earth pressure balance shields are being rivalled by forepoling and consolidation by bolting and drainage. These techniques are used alone or in combination.

They control extrusion of the ground and convergence behind the working face. Extending the convergence-confinement method to forepoling situations has been studied by many researchers. A method for determining confinement loss behind the face has been proposed for elastic conditions [13], but there is as yet no method for elastic-plastic conditions. Nevertheless, in many cases where displacements caused by tunnel driving need to be very tightly controlled, the elasticity approach may be sufficient if plastic deformation is insignificant. This may be the case with shallow tunnels.

P Aristaghes & P Autuori [2] have made an in-depth analysis of convergence behind the working face of tunnels driven with pressurised shields and have shown the difficulty of using the convergence-confinement method in its strictest form with a single confinement loss value. They introduce three efficiency coefficients, for the working face support pressure, the radial pressure around the shield tail, and the grouting pressure behind the segmental concrete lining. The object is to predict displacements more accurately, especially surface settlement above shallow tunnels driven with a closed-face shield machine.

7 - TIME-DEPENDENT DEFORMATION

Convergence of a tunnel section is found to be due to the working face receding but often also, to deformations which continue to occur when the distance to the working face is much greater than the distance of influence of the face.

The processes causing these time-dependent deformations are

• the rheological behaviour of the ground, and

• creep in the support system.

When considering the rheological behaviour of the ground, one must, as far as possible, distinguish between the rheological behaviour of the soil skeleton and the development of steady-state pore water flow around the tunnel. It is frequently difficult to make this distinction through lack of data on the hydraulic properties of the ground (permeability coefficient, storage coefficient) and on how the water table is fed.

Support creep is an important factor, especially with concrete support systems.

Including for these time-dependent deformations when analysing ground/support interactions raises many difficulties which have not yet all been resolved.

In the very great majority of practical cases, simplifying assumptions have to be made [22].

The validity of these assumptions must be checked by assessing the characteristic times of the various time-dependent processes:

• Characteristic time of excavation rate:

$$T_A = \frac{d}{V_A}$$

• Relaxation time characterising the time-dependent deformation rate of the ground T_M

• Relaxation time characterising the creep behaviour of the support T_S.

Order-of-magnitude values for these characteristic times may be very different and justify the simplifying assumptions introduced.

The most common practice is to take a convergence equation with short-term characteristics for the driving and support installation stages, and another convergence equation with long-term characteristics to determine final equilibrium ($T_A << T_M$).

When the permanent lining is installed a long time after the temporary support, intermediate characteristics between the short- and long-term characteristics may be used.

However, theoretical developments for viscoplastic axisymmetric conditions [4] show that this is not always justified. It is only acceptable with support whose relative stiffness modulus meets the following condition:

$$k_{SN} < \frac{1}{1-2\nu}$$

This condition cannot be satisfied in soil

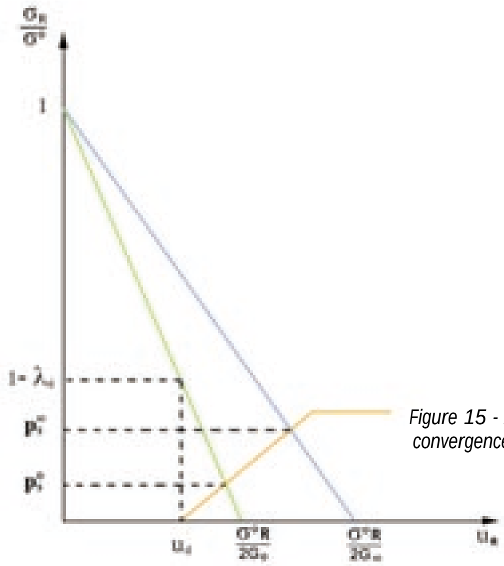

Figure 15 - Axisymmetric case. Graphic representation of convergence-confinement method for viscoelastic ground

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