# Evaluation of rock bolts effectiveness in reducing tunnel convergence in squeezing rocks

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*Summary*

The interaction between fully bonded rock bolts and the ground is analyzed through the homogenized material approach. The action of each radial reinforcement is smeared throughout the bolted rock annulus around the tunnel which is then treated as a fibre-reinforced material with perfect bonding between fibres and rock matrix. In heavy squeezing rock masses the development of zones with rock and bolts in plastic conditions around the excavation is unavoidable. The analytical solution provided for the stress and strain in each zone around the excavation, characterized by elastic or plastic conditions of ground and/or rock-bolts can be utilized to calculate the convergence-confinement curve of the bolted tunnel. The influence of partial debonding can also be taken into account in a simplified way. An application of the method is shown making reference to the geotechnical conditions of a well-known case history. The proposed design method has the advantage of optimizing the main design parameters of the bolting system (bolt density and length, steel and grout quality) on the basis of detailed parametric studies. Moreover the influence of the construction process can be evaluated.

## 1. Introduction

The supporting effect of prestressed anchors is usually schematized as an internal pressure acting on the tunnel wall; the value of this stabilizing pressure is related only to the pretensioning load because it is assumed that it does not change substantially as the rock mass deformation increases. On the contrary such a simple model cannot be used in general for untensioned fully grouted rock bolts. In fact the axial stress along the bolt depends on the local strain increment experienced by the rock mass after bolt installation.

Fully grouted rock bolts act as a confining reinforcement arranged orthogonally to the direction of the principal compressive stress around the tunnel; moreover, their stabilizing action is not localized at the excavation surface but is diffused throughout the thickness of the bolted annulus.

Moving from these considerations, some authors [Stille, 1983; Sakurai 1988] have suggested that the bolted annulus be considered as an equivalent homogenized material with improved stiffness and strength.

The strength of the composite material should be characterized by a cohesion higher than that of the matrix which depends on the reinforcement ratio $\gamma$ (bolt section area $A_b$ / rock surface area $S_r$ $S_l$).

The ultimate equivalent cohesion $c_b$ afforded by the rock-bolts can be estimated by the formula

$$c_b = 0.5\,\gamma\, f_y\, \mathrm{tg}\,(45°+\phi/2) \qquad (1)$$

where $f_y$ represents the yield stress of the bolts and $\phi$ the friction angle of the rock mass. The method of the homogenized equivalent material can be utilized not only in the framework of yield design of reinforcing systems, but also in stress analyses of a more general type, where account needs to be kept of the elastic behaviour of the bolts and rock and of the partial stress relaxation occurring inside the rock mass before the rock-bolts are installed.

A more thorough analysis of the reinforced rock mass behaviour requires the formulation of a complete constitutive law for the composite fiber-reinforced material.

In order to be able to work out analytical solutions it is necessary to idealize the real situation: the usual assumption is that of a circular tunnel driven into a rock mass characterized by an isotropic in situ stress $\sigma_o$ (Fig.1). As in the convergence-confinement method, the effect of the excavation progress after the installation of the rock-bolts is simulated by a progressive reduction of the pressure $p_i=(1-\lambda)\sigma_o$ applied on the tunnel wall, controlled by the relaxation factor $\lambda$.

For the sake of simplicity, an axisymmetric bolt system consisting of equally spaced rock bolts both in the radial and longitudinal directions is considered. Hence the deformation pattern around the tunnel is characterized by plane-strain conditions.

The analytical derivations of the relationships governing the states of stress and strain in different zones around the tunnel, characterized by elastic or plastic conditions for the bolts and the ground, have been discussed in detail by Graziani [1994].

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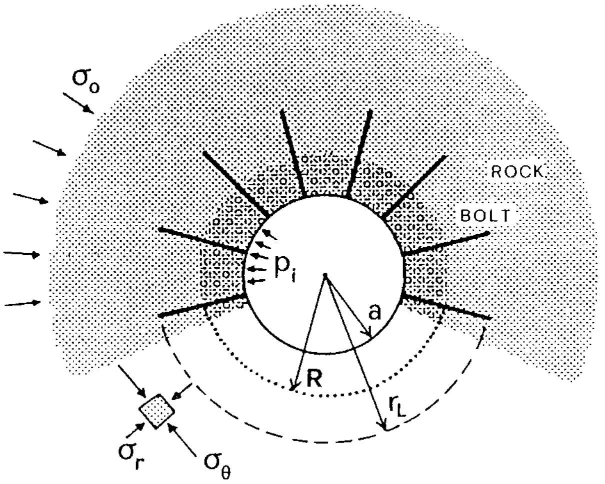


Fig. 1 – Scheme of a circular tunnel supported by radial rock-bolts in an elasto-plastic rock mass.
*Fig. 1 – Schema di galleria circolare in un ammasso roccioso elasto-plastico sostenuta mediante barre passive.*

Similar hypotheses had been made by STILLE, HOLBERG & NORD [1989], and INDRARATNA & KAISER [1990].

In the paper only the basic assumptions of the proposed method are described. More emphasis is placed on the application of the method to a real case history of a tunnel driven through squeezing rock in a fault zone (Enasan tunnel, Japan). The influence of in situ stress severity and of the delay of bolt installation behind the tunnel face has been investigated by extensive parametric analyses.

## 2. Equivalent material approach

The derivation of the constitutive law for the reinforced material is specialized for the axisymmetric plane-strain problem of the bolt-supported circular tunnel. In such conditions the radial and tangential directions r and θ, are principal directions, and displacements u are purely radial.

The fundamental assumption is made that during ground deformation the perfect bonding between bolt shaft and the ground is preserved. Using $\Delta\varepsilon_r$ $\Delta\varepsilon_\theta$ to indicate the rock strain (compression positive) occurring after the placing of the bolts and $\varepsilon_b$ for the axial strains of the bolts (tension positive), the perfect bonding condition is

$$-\varepsilon_b(r) = \Delta\varepsilon_r(r) \tag{2}$$

The radial stress $\sigma_r$ in the homogenized medium is obtained by adding together the contribution afforded by the rock $\sigma_r^a$ and the smeared equivalent contribution $\sigma_r^b$ afforded by the bolts, while $\sigma_\theta$ only the rock contributes to the hoop stress

$$\sigma_r = \sigma_r^{a+b} = \sigma_r^a + \sigma_r^b \qquad \sigma_\theta = \sigma_\theta^a \tag{3}$$

The equivalent contribution by the bolt system $\sigma_r^b$ is equal to the effective bolt-axial stress $\sigma_b$ smeared over the section area of the rock wedge influenced by the bolt

$$\sigma_r^b = -\gamma \cdot r_i \cdot \frac{\sigma_b\, r}{r} \tag{4}$$

An ideal elasto-plastic behaviour is assumed for the bolts characterized by the elastic modulus $E_b$ and yield stress $f_y$.

For the rock a brittle elasto-plastic behaviour is assumed by using Mohr-Coulomb, peak and residual strength criteria.

Once a point reaches the peak strength curve (identified by parameters $c_p$ and $\phi_p$), the state of stress drops sharply to the residual strength curve ($c_r$, $\phi_r$). The development of plastic deformation in the ground is governed by a flow rule generally of the non-associated type (dilatancy angle ψ). To complete the mathematical formulation of the problem, besides the constitutive relationships and the kinematic compatibility relationships, the equilibrium equations of the homogenized material are needed too.

The equilibrium condition for the composite medium expressed in cylindrical coordinates is

$$\frac{d(\sigma_r^a + \sigma_r^b)}{dr} = \frac{\sigma_\theta - (\sigma_r^a + \sigma_r^b)}{r} \tag{5}$$

By combining the set of fundamental equations it is found that each annulus around the tunnel, with the ground and bolts in elastic or plastic conditions, is governed by a second order differential equation in the unknown variable u.

In most cases, closed-form solutions can be worked out. Hence the global problem can be solved by inter-relating the results obtained for the individual zones through the boundary conditions which express the continuity of displacements and radial stresses at each interface between adjacent zones.

As an example, the equation governing the zone characterized by elastic conditions for both rock and bolts is

$$\left(1+\rho\frac{a}{r}\right)\cdot\frac{d^2\Delta u}{dr^2} + \frac{1}{r}\cdot\frac{d\Delta u}{dr^2} - \frac{1}{r^2}\cdot\Delta u = 0 \tag{6}$$

The general solution to this equation depends on a dimensionless parameter ρ (bolting coefficient), related also to the elastic moduli ratio $E_b/E$, and on two arbitrary constants A, B to be determined by the boundary conditions

$$\Delta u = A \cdot r + B \cdot r \left[ \ln\left(1 + \frac{\rho a}{r}\right) - \frac{\rho a}{r} \right] \qquad (7)$$

with $$\rho = \gamma \frac{E_b}{E} \frac{(1+\nu)(1-2\nu)}{1-\nu}$$

An example of stress distribution around the tunnel in elastic conditions is illustrated in Fig. 2. The rock mass and bolt parameters utilized in all the example calculations are those of the Enasan tunnel which will be later described in detail (Tab. I), while the in situ stress $\sigma_o$ has been reduced ($\sigma_o$ = 2 MPa) in the example of Fig. 2 to maintain purely elastic conditions.

Except for cases of very high bolt density, the influence of rock-bolts on tunnel behaviour appears to be negligible inside the elastic field. This general result is the consequence of the low value of the bolting coefficient ($\rho$ < 0.1 ) for the bolt density commonly used in practice. Rock bolting is much more effective in reducing rock mass deformations after yielding of the rock mass.

Fig 3 illustrates the state of stress around the tunnel obtained when an in situ stress $\sigma_o$ = 6 MPa is assumed. In this case four different zones can be identified starting from the tunnel wall: a first annulus (a < r < R) with both rock and bolts in plastic conditions; a second annulus (R < r < $R_b$) with elastic rock but bolts still in plastic conditions; a third annulus ($R_b$ < r < $r_L$) with elastic bolts; and finally, the ground without bolts.

The governing equation of the zone with elastic bolt and plastic rock has the same structure as Eq. 2.6, but it is more complex, mainly because the ground conditions (elastic and/or plastic) inside the bolted annulus prior to the placing of the bolt system, need to be taken into account

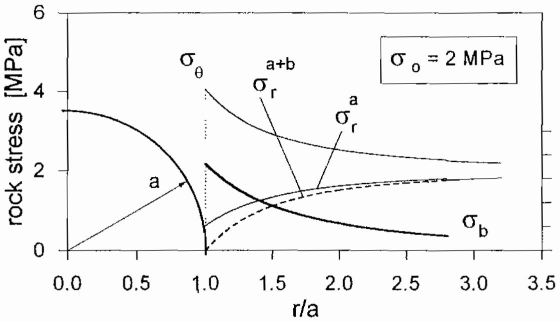


Fig. 2 – Stress around a circular tunnel in elastic condition: rock stress (a), homogenized material stress (a+b) and bolt axial stress $\sigma_b$.

*Fig. 2 – Stato di sforzo intorno ad una galleria circolare in condizioni elastiche: sforzi roccia (a), materiale omogeneizzato (a+b) e sforzo assiale nelle barre $\sigma_b$.*

## 3. Simulation of the excavation process

When the bolts are installed in a tunnel section, the surrounding rock has already undergone some stress relief which is more or less pronounced depending on the distance from the tunnel face.

To take into account the delay between excavation and installation of the rock bolts, the relief of the internal pressure $p_i = (1-\lambda)\sigma_o$ must be separated into two phases: a first phase (from $\lambda$=0 to $\lambda=\lambda_B$) simulating the amount of relaxation before bolt installation, a second phase (from $\lambda=\lambda_B$ to $\lambda$=1) corresponding to the loading of the bolts due to further progress of the tunnel face.

Of course this relaxation process is only a fictitious tool to schematize the support effect related to the real 3D conditions existing in the vicinity of the tunnel face [PANET & GUENOT, 1982].

A relaxation factor at bolt installation of $\lambda_B$=0.6 was assumed for the example calculation in Figs. 2 and 3; and the stress conditions represented in the figures correspond to $\lambda$=0 (plane-strain conditions away from the face).

From the design point of view it would be very useful to have a relationship between the relaxation factor $\lambda_B$ and the distance $x_B$ between the face and the tunnel section where the rock bolts are installed. BERNAUD, CORBETTA & MINH [1991] found that the deformation pattern in the vicinity of the face (i.e. tunnel-wall displacement $u_a$ as a function of the distance x from the face) is quite insensitive to the change of rock strength parameters in a wide range, and proposed the following relationship

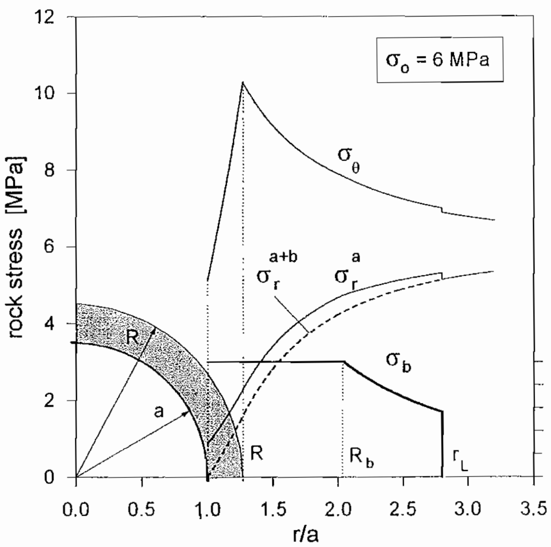


Fig. 3 – Stress around a bolt-supported tunnel with elasto-plastic behaviour of both rock mass and bolts.

*Fig. 3 – Stato di sforzo intorno ad una galleria sostenuta con barre passive (analisi elastica).*

$$u_a(x)=u_{a\infty}\left\{0.29+0.71\left[1-e^{-1.5\left(x/\xi a\right)^{0.7}}\right]\right\} \quad (8)$$

where $\xi$ is the ratio between the displacement of the tunnel wall $u_{a\infty}$ for an elasto-plastic behaviour of the rock (plane strain condition for $x \rightarrow \infty$), and the same displacement calculated for a purely elastic behaviour $u_{a\infty}^{el}$. According to the homogenized material approach, it is deemed reasonable to apply rel. (7) also to the bolt-reinforced tunnel. While the elastic displacement $u_{a\infty}^{el}$ is almost independent of the bolt installation, the elasto-plastic displacement $u_{a\infty}$ should be a function of the relaxation factor $\lambda_B$ which in turn is dependent on the distance $x_B$ through the associated displacement $u_a(x_B)$.

The relationship between $\lambda_B$ and $x_B$ therefore cannot be resolved explicitly by Eq. 7.

This difficulty can be overcome by repeating the calculations for different values of $\lambda_B$. More precisely the calculation steps are the following:

- for a given value of $\lambda_B$ , the wall displacement before bolt installation $u_a(x_B)$ and the final displacement $u_{a\infty}$ are calculated by the solutions obtained for the zones present around the excavation;
- the displacement $u_a(x_B)$ now obtained is substituted on the left side of rel. (7), which can be seen as an equation in the unknown distance $x_B$ to be solved by trial and error.

## 4. Influence of debonding between bolt and rock

In the vicinity of the tunnel wall, where the loosening of the rock mass may be particularly intense and the grouting operation less accurate, also the grouting material can reach plastic conditions. In this case the bonding stress (tangential stress) between the bolt shaft and the rock mass is assumed to be equal to a constant limit value $\tau_b$, which can be estimated on the basis of the residual cohesion of the rock mass.

The effectiveness of the rock bolts in the immediate vicinity of the excavation boundary strongly depends on the bearing capacity of the rock mass under the end plate of the bolt and on the ability of the nut-plate blocking system to undergo high plastic deformation without failure. JOHN [1979] reports that during the construction of the Arlberg tunnel in a heavy squeezing rock mass, 21% of the total number of rock bolts (71) instrumented with strain gauges, failed as a result of end plate breakage, 54% failed along the shaft and only 25% preserved service ability. Following STILLE, HOLBERG & NORD [1989], two static assumptions can be envisaged in these particularly exacting conditions. According to a first scheme (model M1), the end plate is assumed to be still efficient and to apply a uniform load on the rock surface.

The local deflection $\delta$ of the tunnel wall caused by the load applied by the end plate is

$$\delta = \frac{d_b^{\,2}}{d_p} \cdot \frac{1-\nu^2}{E_r} \cdot \sigma_b(a) \quad (9)$$

where $d_b$ and $d_p$ are the diameter of the bolt and of the plate, $E_r$ is an elastic modulus representative of the rock condition under the plate ($E_r$ = 0.01÷0.04 E due to loosening of the rock material and unmatched contact between steel plate and rock surface). The bolt stress at the tunnel wall $\sigma_b(a)$ and the external radius $R_c$ of the annulus with plastic grout can be calculated through the condition that the total axial deformation of the debonded bolt segment must be equal to the total radial deformation of the rock annulus minus the local deflection $\delta$; moreover the continuity condition of bolt stress at the external boundary $R_c$ must be satisfied.

In a second scheme (model M2) it is assumed that the end plate has either not been installed or is completely out of working order. The bolt load $\sigma_b(a)$ is therefore null, while the extent of the zone with grout in shearing conditions is determined by imposing stress continuity at the external boundary.

In Fig. 4 the trend of the axial stress along the bolt calculated by the perfect bonding model (model M0) is compared with the stress obtained by the model M1 and M2. In the calculations the limit bonding strength $\tau_b$ has been assumed equal to the cohesion (0.5 MPa) estimated for the rock formation of the Enasan tunnel.

The impact of the debonding phenomena on the overall behaviour of the tunnel will be discussed by making reference to a real case.

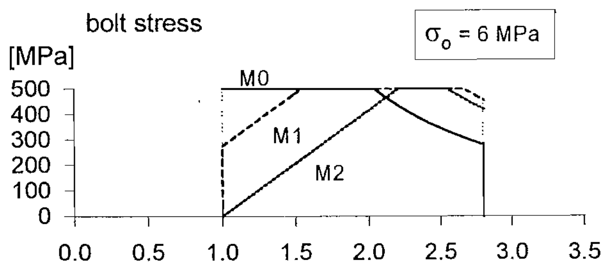


Fig. 4 – Bolt axial stress calculated by model M0 with perfect bonding and by models M1, M2 which take into account a partial debonding ($\tau_b$=0.5MPa).

*Fig. 4 – Sforzo assiale nelle barre calcolato con il modello M0 (aderenza ideale) e con i modelli M1 e M2 (sforzo limite di aderenza barra-roccia $\tau_b$=0.5MPa).*

## 5. Application of the analytical model

In the design of the support system for a tunnel driven through squeezing rock, the main parameter to be checked is the convergence of the excavation. Indeed, around a deep tunnel, the rock yield strength is inevitably exceeded and the stability of the excavation can only be assessed in terms of maximum admissible deformation.

$$\varepsilon_{\theta\,adm} = u_{a\,adm}/a \qquad (10)$$

On the basis of the data collected by SAKURAI [1988] on the effective behaviour of many tunnels, the value of $\varepsilon_{\theta adm}$ is roughly 1% and tends to increase with the reinforcement density $\gamma$, which can significantly improve the ductile behaviour of the rock mass.

The importance of a design tool allowing extensive parametric analyses is clearly understood if ones considers that most of the geotechnical parameters which govern the behaviour of the rock mass are affected by great uncertainty and variability along the alignment of the tunnel to be excavated.

The calculation of the characteristic curve of the bolted tunnel by the proposed analytical method can therefore be a valuable design tool.

The geotechnical situation encountered during the construction of the Enasan tunnel [KIMURA, OKABAYASHI & KAWAMOTO, 1987] has been chosen as a typical case of a tunnel driven through a heavy squeezing rock mass, especially for the section driven through the Chobeizawa fault zone, which is mainly composed of altered granite with abundant residual clay (Table I).

The tunnel (radius a = 5m) was excavated by the so-called NATM method. The following construction stages were planned: head excavation (1m/blast), immediate application of a steel-fibre-reinforced shotcrete layer (20 cm thick), installation of a regular rock bolt pattern ($d_b$ =24mm, length 9m, $n_b$ =1bolt/m$^2$), bench excavation and completion of the primary support system at the invert.

Due to the expected occurrence of large deformations, it was anticipated that the insertion of six 25-cm-wide longitudinal slots in the shotcrete lining could accommodate a convergence of the rock of up to 50 cm without collapse of the support system. As when the initially expected maximum deformation was exceeded, the design density of the bolts was increased fourfold (Fig. 10) up to $\gamma$ = 0.17%, and the slots were sealed by means of shotcrete.

The characteristic curve $\lambda$, $u_a$ of the bolted tunnel ($\lambda_B$=0.6, $\gamma$=0.17%) calculated by the different models proposed are represented in Fig. 5. The tunnel wall convergence $u_a$ as a function of face distance x, calculated by rel. (7) is also shown.

With a bolt density $\gamma$ of 0.17%, the final convergence at great distance from the face can be reduced by more than 50% if perfect bonding between ground and bolt is preserved.

The influence of the in situ stress value $\sigma_o$ over the tunnel wall convergence, and the corresponding extension of the different plastic zones around the excavation are illustrated in Fig. 6. For the maximum value of $\sigma_o$ considered here ($\sigma_o$=10MPa), cor-

Tab. I – Rock mass parameters for the Chobeizawa fault zone encountered during the construction of the Enasan tunnel.
*Tab. I – Parametri dell'ammasso roccioso in corrispondenza della faglia di Chobeizawa (Enasan tunnel).*

| | | | | | |
|---|---|---|---|---|---|
| E | = | 350 MPa | n | = | 0.4 |
| $c_p$, $c_r$ | = | 0.5 MPa | H | = | 400 m |
| $\phi_p$, $\phi_r$ | = | 35° | y | = | 10° |
| RMR | = | 20 | $\gamma_v$ | = | 25 KN/m$^3$ |

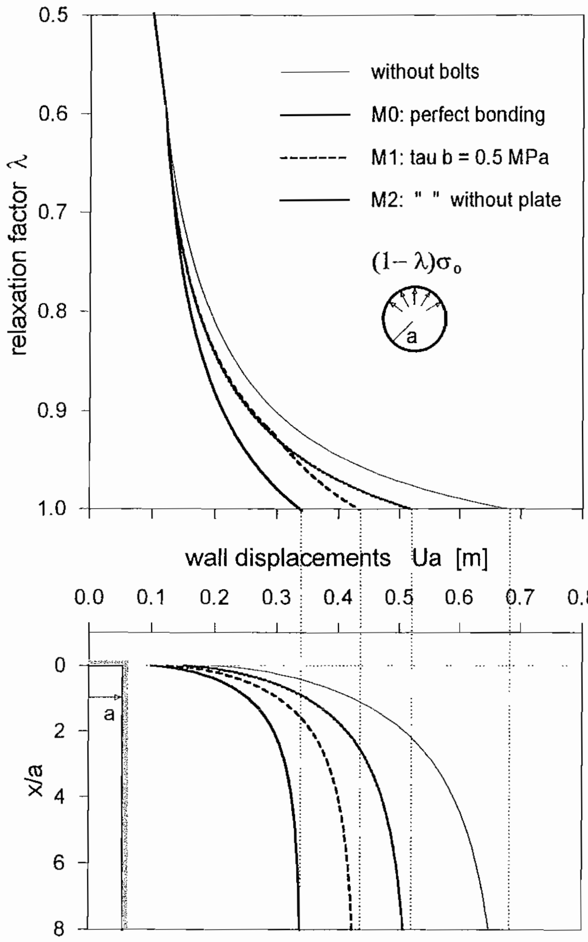


Fig. 5 – Characteristic curves of the bolted tunnel (above) and wall displacements along the tunnel axis (below).
*Fig. 5 – Curve caratteristiche della galleria bullonata (in alto) e spostamento della parete galleria in prossimità del fronte (in basso).*

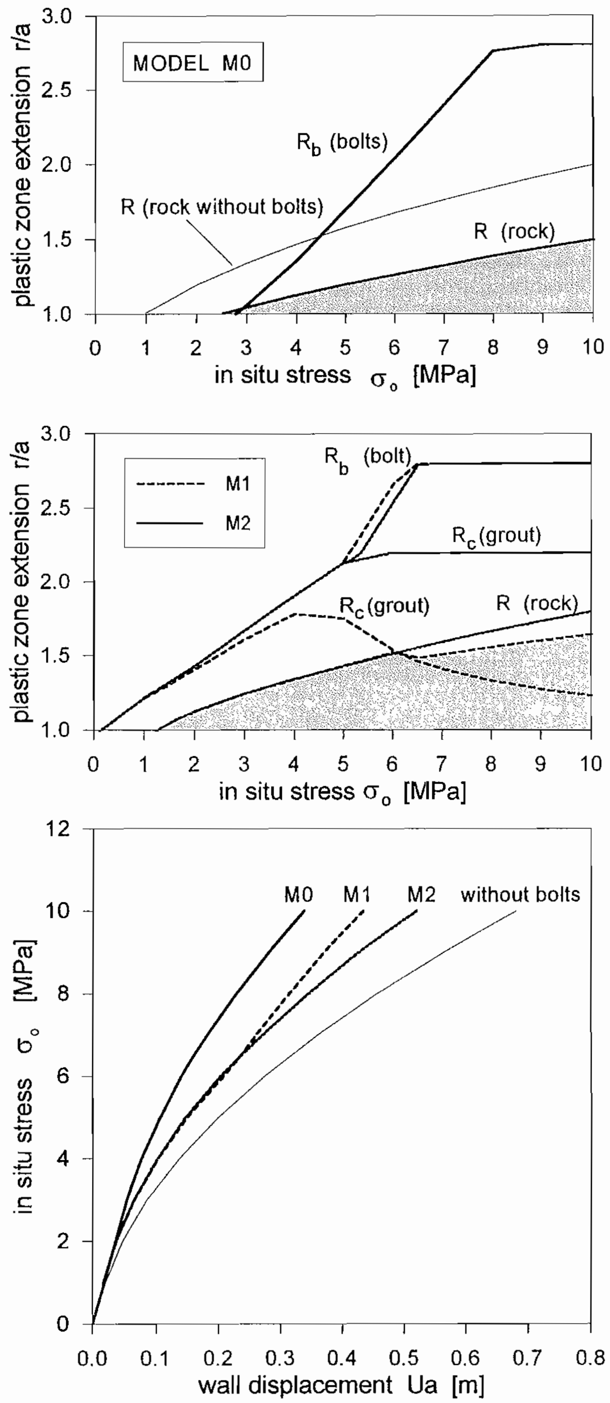


Fig. 6 – Tunnel wall displacements as a function of the in situ stress $\sigma_o$ (below) and corresponding extensions of the zones with rock, bolts and grout in plastic conditions (above), calculated by different models.

*Fig. 6 – Spostamento parete galleria in funzione dello sforzo in situ $\sigma_0$ (in basso) e corrispondente estensione delle zone con roccia, barre e materiale di cementazione in condizioni plastiche (in alto), calcolati con diversi modelli.*

responding to the overburden load in the Chobeizawa fault, the steel reinforcements are completely plasticized. In fact, the external radius $R_b$ of the annulus with yielded rock-bolts (Fig. 6) becomes equal to $r_L$ when $\sigma_o$ reaches 6.5MPa if a limited strength of the grouting material is assumed (model M1 and M2), while a higher in situ stress $\sigma_o$ = 8MPa is needed if perfect bonding is preserved (model M0).

In the extreme working conditions of the Chobeizawa fault zone, the bolt action tends to be similar to that of a system of prestressed anchors with a pretension load equal to the yield limit of the steel bar. In this situation the stabilizing action of rock bolts is almost equivalent to a pressure $p_i = \gamma f_y$ applied on the tunnel wall. Another simplified approach to evaluate the effectiveness of passive reinforcement is given by rel. (1) which represents the equivalent cohesion due to the confinement effect of yielded rock-bolts.

The comparison of the convergence predicted by the homogenized material method (model M0) and by the two aforementioned simplified calculation schemes is illustrated in Fig.7. While the internal pressure approach largely overestimates the efficacy of the bolts in reducing the tunnel convergence, especially for low overburden loads, the equivalent cohesion approach seems to give more reliable results at least for a heavy squeezing rock mass.

One of the crucial points in the application of the confinement-convergence method as well as in the 2D numerical calculation of the stress and strain condition around bolted or lined tunnels is a correct evaluation of the relaxation factor $\lambda_B$ when the support system is installed. The procedure described in paragraph 3 may be a useful aid for evaluating $\lambda_B$ as a function of the distance $x_B$ between the face and the section where the bolts are installed.

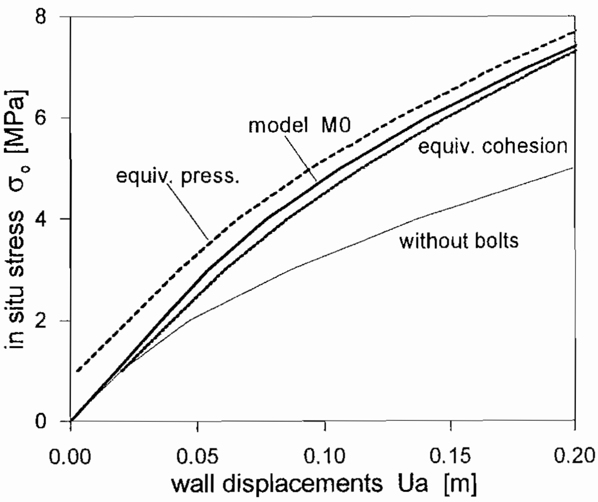


Fig. 7 – Tunnel wall displacement calculated by different methods for increasing values of in situ stress.

*Fig. 7 – Spostamento parete galleria calcolato con diversi modelli per valori crescenti dello sforzo in situ.*

The tunnel wall displacement $u_a$ as a function of the face distance $x_B$ at the time of bolt installation calculated for the Chobeizawa fault conditions is illustrated in Fig. 8. It is worth noting that the curves calculated with model M0 show a rather flat trend for low values of $x_B$ followed by a sharp increase which is particularly steep in the case of perfect bonding. This means that in a heavy squeezing rock the effectiveness of the bolts remains almost unchanged even if there is a rather large "delay" ($x_B/a$ = 2÷4) in installing the rock-bolts. If the end plate is not efficient (model M2), the performance of the bolting system is strongly undermined. This reduction of efficacy due to debonding near the tunnel wall is found to be proportionately higher for bolts installed very close to the face.

In Fig. 9 (upper part) the final displacement $u_a$ of the tunnel wall scaled to the final displacement $u_a^*$ of the unsupported tunnel is represented as a function of the bolting "delay" $x_B$ for three different values of the in situ stress.

In the lower part of the same figure, the corresponding value of the relaxation factor $\lambda_B$ to be introduced in numerical or analytical calculations is indicated. Even if the value of $\lambda_B$ increases very sharply for short distances $x_B$ from the tunnel face, the influence of the $\lambda_B$ value over the final tunnel wall displacement for this range of values is not very large.

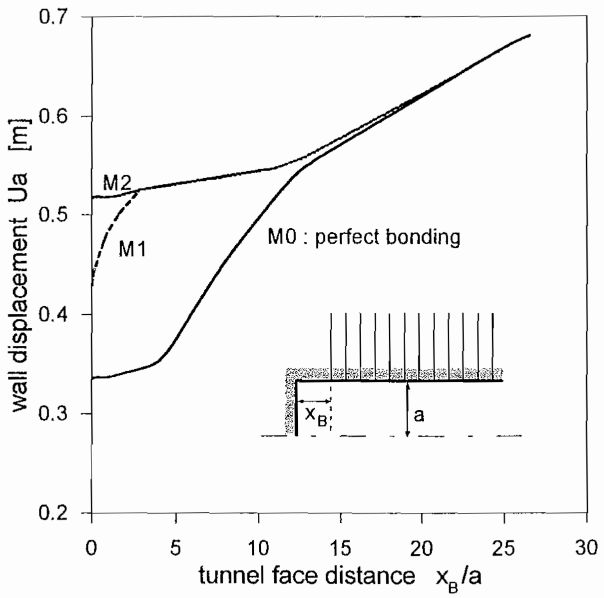


Fig. 8 – Final convergence (for a face distance x → ∞) as a function of face distance at rock bolt placing.

*Fig. 8 – Convergenza finale (a grande distanza dal fronte x → ∞) in funzione della distanza del fronte alla messa in opera delle barre.*

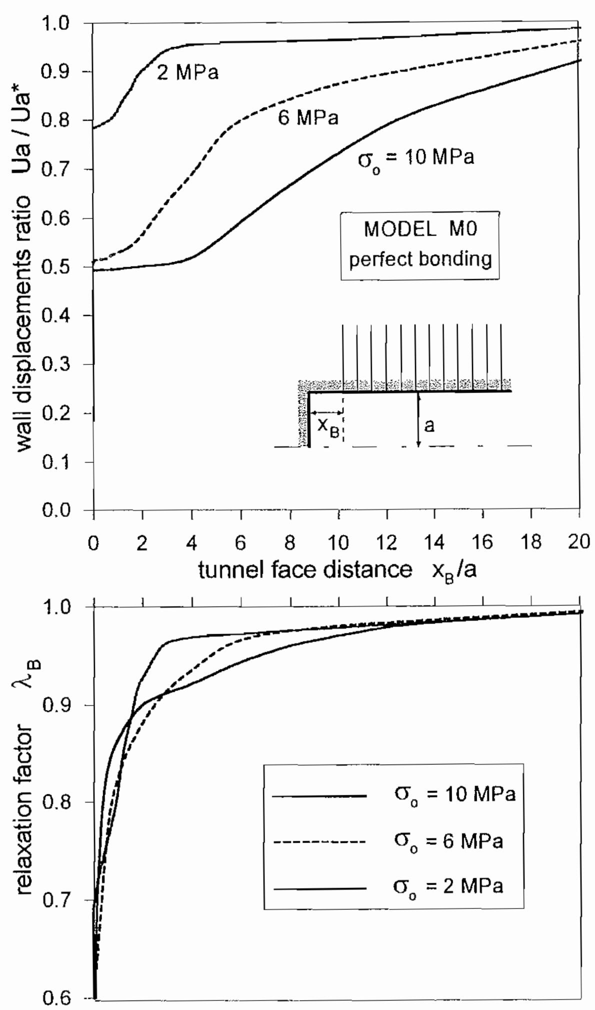


Fig. 9 – Final convergence (scaled to convergence $u_a^*$ calculated without bolts) as a function of bolting "delay" $x_B$; the relaxation factor $\lambda_B$ corresponding to the different values of $x_B$ is also shown (below).

*Fig. 9 – Convergenza finale (scalata rispetto alla convergenza $u_a^*$ in assenza di barre) in funzione del "ritardo" d'installazione delle barre $x_B$ (in alto) e corrispondente valore del fattore di rilascio $\lambda_B$ (in basso).*

While in conditions of "high" in situ stress ($\sigma_o$= 10MPa, heavy-squeezing case) the effectiveness of rock bolts starts to decrease only for $x_B/a > 4$, in conditions of "moderate" in situ stress ($\sigma_o$= 6MPa) the decrease begins earlier and shows a sharp knee around $x_B/a > 2$ Finally for "low" stress conditions ($\sigma_o$= 2MPa), the bending points in the $u_a$, $x_B$ curve, shifts to $x_B/a = 1$.

Convergence measurements and face progress recorded at Station A in the Chobeizawa fault plotted against time [BARLOW & KAISER, 1987] are illustrated in Fig. 10 together with the final convergence δ computed by the proposed models M0 and M2. As the measurements started very close to the face, the predicted value of convergence is $\delta = 2*0.71 u_{a\infty}$ (from rel. 7).

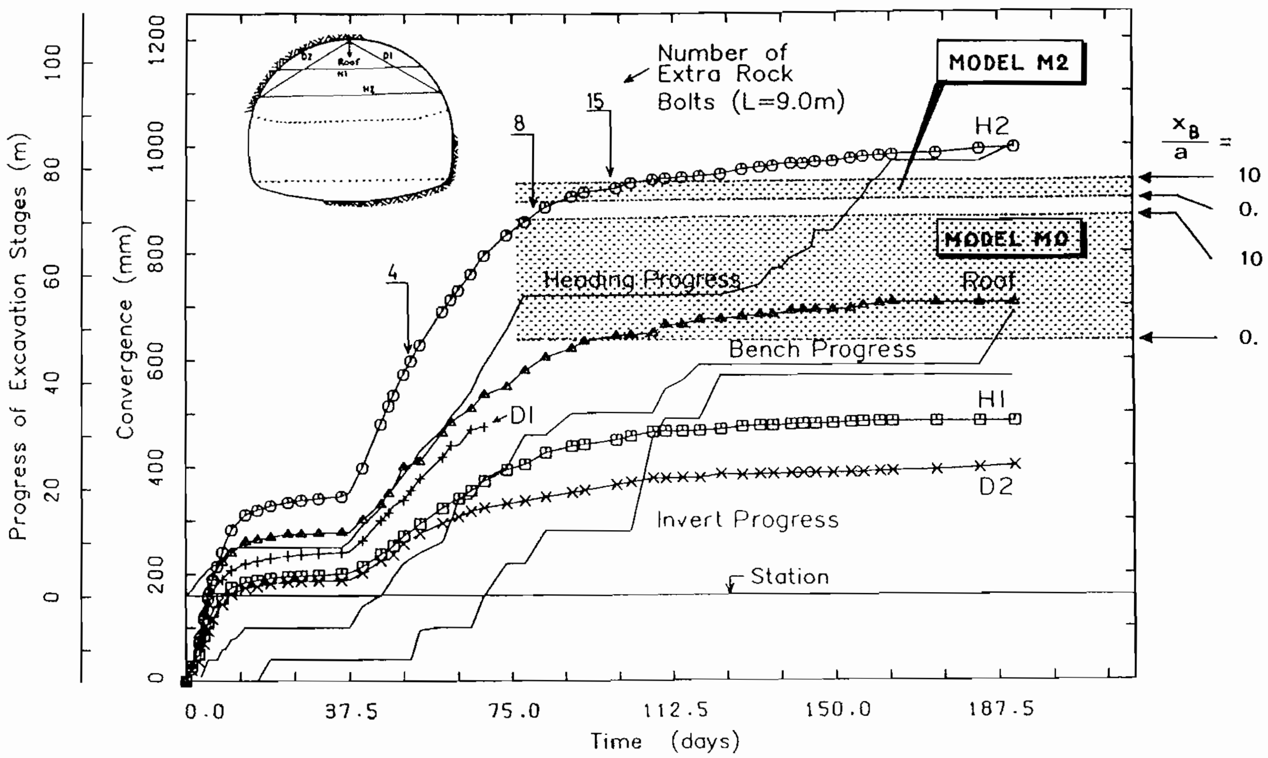


Fig. 10 – Excavation progress and convergence measurements at Station A in the Enasan tunnel [after BARLOW & KAISER, 1987], compared with the convergences predicted by model M0 and M2 for different $x_B$ values (shaded area).
*Fig. 10 – Fasi di avanzamento dello scavo e misure di convergenza relative alla Stazione A del tunnel di Enasan [da BARLOW e KAISER, 1987]; confronto con le convergenze previste dai modelli di calcolo M0 e M2 per diversi valori di $x_B$ (aree puntinate).*

The comparison between measured and computed values of convergence is made more difficult by the following reasons:

- the sequential excavation process applied (head and bench method) cannot be modelled in detail;
- a large number of rock-bolts have been installed far away from the face, therefore $x_B$ is variable.

The impact of these specific features of the construction process on the tunnel convergence can only be evaluated on average by a sensitivity analysis in which the distance $x_B$ is made to vary within a wide range.

The shaded areas in Fig. 10 indicate the final convergence predicted by models M0 and M2 for $x_B$ varying between 0 and 10a. The computed values compare well on average with the convergence measured along different chords.

It is worth noting that for the assumed range of face distances $x_B$, the relaxation factor $\lambda_B$ is comprised between 0.65 and 0.97, values that are much higher than those derived from a linear-elastic analysis ($\lambda_B \approx 0.3$).

As already mentioned about Fig. 8, the wall displacements predicted by the limit bonding model M2 are much less sensitive to the increase of support "delay" $x_B$ than the displacements predicted by model M0.

## 6. Conclusions

A simplified analytical model for the calculation of the states of stress and strain around a circular tunnel supported by a ring of radial rock-bolts has been developed.

The predictions afforded by the model have been compared with the measurements of convergence carried out during tunnelling through a fault zone (Enasan Tunnel, Japan).

Some of the points the paper wants to convey are:

- the influence of yielding of the rock mass and/or the bolts, as well as the effect of a partial debonding between the bolts and the rock surrounding the tunnel wall can be simply analysed by the proposed method, which can therefore be utilised for extensive parametric studies in the baseline design stage;
- a general relationship between the support "delay" $x_B$ (distance between the face and the tunnel section where rock-bolts are installed) and the relaxation factor $\lambda_B$ to be applied in plane-strain analyses of bolt-supported tunnels can be identified;
- the effectiveness of rock-bolts in controlling tunnel convergence is less sensitive to the support

"delay" $x_B$ in heavy squeezing rock masses where high in situ stress and large plastic deformations are expected, but is strongly reduced if the end-plates are not efficient and debonding phenomena are widespread in the vicinity of the excavation surface.

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## Valutazione dell'efficienza della bullonatura nella riduzione della convergenza in galleria in rocce spingenti

### *Sommario*

*Viene studiata l'interazione tra un sistema di barre passive e il terreno seguendo l'approccio del mezzo equivalente omogeneizzato. L'azione esercitata da ciascun rinforzo radiale intorno alla galleria si considera quindi diffusa all'interno dell'anello di roccia rinforzata, che viene quindi modellato come un mezzo fibro-rinforzato, ipotizzando condizioni di perfetta aderenza tra "fibre" e matrice rocciosa.*

*In un ammasso roccioso con forte tendenza al "rifluimento", è inevitabile la presenza di una fascia in prossimità dello scavo con roccia e barre in condizioni plastiche.*

*Le soluzioni analitiche ottenute per gli sforzi e le deformazioni nelle diverse zone presenti intorno allo scavo, caratterizzate da condizioni elastiche o plastiche della roccia e/o dei rinforzi possono essere utilizzate per il calcolo della curva convergenza-confinamento della galleria bullonata. È anche possibile tener conto in modo semplice dell'effetto di una limitata perdita di aderenza tra rinforzi e terreno.*

*Infine viene illustrata un'applicazione del metodo per lo studio di una situazione geotecnica reale. Il metodo proposto rappresenta uno strumento utile per il progetto del sistema di rinforzo di una galleria, in quanto consente di ottimizzare i principali parametri dell'intervento di bullonamento (densità e lunghezza delle barre, caratteristiche dell'acciaio e del materiale di cementazione) sulla base di dettagliate analisi parametriche. È possibile inoltre valutare l'effetto delle varie fasi di scavo e messa in opera dei rinforzi.*