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Access tunnel convergence prediction in longwall coal mining

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Summary

The paper describes theoretical and *in situ* studies of tunnel deformation in longwall coal mining. It develops a method to predict tunnel convergence profiles from the faceline in longwall mining. The method accounts for the effect of panel width, extracted seam height, deformation moduli of the goaf material and coal pillar, depth of cover, *in situ* structural defects, tunnel shape and tunnel size in addition to the strength characteristics of surrounding strata. The analytical technique has been validated by reference to *in-situ* deformation measurements in 26 face-access tunnels in Cape Breton Coalfield mines. Based on this method a series of vertical convergence profiles for different depths and extracted panel widths have been presented.

Keywords: Face-access tunnels; strata control; tunnel stability; longwall mining; Cape Breton Coalfield.

Introduction

In order to analyse the stresses around mining excavations, Krauland (1971) gives a comprehensive review of the methods for analysing elastic and non-elastic stresses around mining tunnels using various strata behaviour models.

The elastic solutions permit the calculation of radial, tangential and shear stresses within an elastic rock mass. When the induced stresses reach a certain critical level, failure of the rock mass occurs, resulting in the formation of a yield zone. For an effective and appropriate tunnel and support design it is essential to determine the extent of the yield zone and the distribution of stresses inside and outside this zone. Various methods exist for the determination of the induced stresses under such conditions. The concept of yield zones was originally examined by Fenner (1936). The extent of the plastic zone was studied by Kastner (1949). The work of Fenner and Kastner has been extended by many investigators and a number of theoretical and empirical solutions have been proposed for determination of the state of stress and strain around a circular opening in rock masses. For example, Bray (1967), Ladanyi (1974), Wilson (1980), Hoek and Brown (1980) and Brown *et al.* (1983) have developed theoretical design criteria to calculate the radius of a fracture zone, the state of stresses, and tunnel closure. A comparative study of these theoretical solutions with *in situ* measurements has been given by Whittaker *et al.* (1983). The latter authors showed that all of

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these solutions underestimate tunnel closure. This could be due to the effect of longwall extraction which was not taken into account. A brief discussion follows in order to highlight some of the existing theories and corresponding assumptions together with the parameters involved.

Bray's (1967) formula was based upon the theory of limiting equilibrium for a finely jointed rock mass. The stress distribution in the fractured zone has been derived from the assumption that the shear resistance on the fracture surface is purely frictional (i.e. $C_r = 1$). Landanyi (1974) in the development of a design criterion considered a long-term strength concept of the rock surrounding the tunnel excavation. Ladanyi's closed-form solution offers the advantage of taking into account a number of significant characteristics of the rock mass, such as its linear or non-linear failure criterion, its post-failure reduction of strength associated with its volume dilation, as well as its strength decrease with time. Wilson's (1980) formula was originally derived from the basis of theoretical considerations which were modified to conform to empirical observations from the British coal mining environment. Wilson assumes that the angle of internal friction of intact rock remains unchanged for the post-failure condition. Hoek and Brown (1980) and Brown *et al.* (1983) based their formulae upon an empirically derived failure criterion (Hoek and Brown, 1980) which takes the post-failure strength reduction into account.

These formulae, however, have the limitation of giving closure at only one point along the tunnel length, therefore, none of them can be used to predict the tunnel convergence profile along the tunnel. They also significantly underestimate tunnel convergence compared with *in situ* measurements (Whittaker *et al.*, 1983). An empirical solution proposed by Hobbs (1968) also suffers the same limitations.

The empirical approach proposed by Panet (1979), and extended by Guenot *et al.* (1985), Sulem *et al.* (1987) and Barlow and Kaiser (1987) gives convergence as a function of distance from face and time. However, this approach has the limitation of being applicable only to a single tunnel, hence, the effect of longwall extraction and *in situ* structural defects are not considered. The development of a new theoretical criterion to minimize and reduce the above drawbacks, when predicting yield zone radius and tunnel convergence, is thus considered important.

A theoretical approach to the yield zone

Jaeger (1979) introduced an analysis of a case in which the rock contains well-defined, parallel planes of weakness whose normals are inclined at an angle θ to the major principal stress direction (measured counter-clockwise from the σ_1 direction). Each plane of weakness has a limiting shear strength defined by the Mohr–Coulomb failure criterion

$$\tau = C + \sigma_n \tan \phi \tag{1}$$

Slip on the plane of weakness will become incipient when the shear stress on the plane, τ, becomes equal to, or greater than the shear strength. The stress transformation equations give the normal and shear stress on the plane of weakness as follows

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta \tag{2}$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta \tag{3}$$

where $\sigma_1 - \sigma_3/2 = \tau_m$ (maximum shear stress).

The relationship of the cohesion, C, and the uniaxial compressive strength, σ_c, can be written in the following form

$$C = \frac{\sigma_c(1 - \sin \phi)}{2 \cos \phi} \tag{4}$$

Substituting Equations 2, 3 and 4 into Equation 1 and simplifying results in

$$(\sigma_1 - \sigma_3)\sin(2\theta - \phi) = \sigma_c(1 - \sin \phi) + (\sigma_1 + \sigma_3)\sin \phi \tag{5}$$

If $\theta = (\pi/4 + \phi/2)$ then $\sin(2\theta - \phi)$ reduces to unity. The failure requirement can be stated as $\phi < \theta < \pi/2$, hence, failure does not take place when $\theta = \phi$ or $\theta = \pi/2$. Employing Equation 3 to a laboratory uniaxial stress condition, σ_{cl} for a solid sample where $\sigma_3 = 0$ and $\sigma_1 = \sigma_{cl}$, results in

$$\sigma_{cl} = \frac{2\tau_s}{\cos \phi_s} \tag{6}$$

Similarly, for a rock sample with a discontinuity, the stress required, σ_{cd} to produce displacement leads to the following equation

$$\sigma_{cd} = \frac{2\tau_d}{\cos \phi_d} \tag{7}$$

According to Bishop (1972), the actual or the maximum strength lies between the two limits of the peak and the residual values hence, the *in situ* uniaxial compressive strength, σ_{ci}, can be obtained if the ratio of Equation 7 to Equation 6 is used as an *in situ* structural defect index, I_d. This results in

$$\sigma_{ci} = I_d \sigma_{cl} \tag{8}$$

in which

$$I_d = \frac{\tau_d \cos \phi_s}{\tau_s \cos \phi_d} \tag{9}$$

where τ_d, ϕ_d and τ_s, ϕ_s are shear strengths and angles of internal friction of defect and solid laboratory samples respectively. An alternative approach to approximate the *in situ* structural defect index, I_d, can also be achieved if *in situ* discontinuity information is obtained from the core log. This can be proposed by the following empirical equation

$$I_d = \exp(-n_d/a)\cos \alpha \tag{10}$$

in which n_d = number of discontinuities or structural defects per metre of core log (or per metre of tunnel height and takes a value of ≤ 10); a = the slope of $\log(I_d/\cos \alpha)$ versus number of discontinuities per metre of tunnel height; α = maximum inclination of the structural defects to the horizontal in degrees. $n_d \geq 0$, whereas, $0 \leq I_d \leq 1$: $I_d = 1$ when n_d for laboratory solid samples. The relationship between RQD and the average number of discontinuities per metre was previously investigated by Priest and Hudson (1976) which also revealed a similar negative exponential function.

By taking the above assumption and replacing σ_c with σ_{ci} from Equation 8 we may now further reduce Equation 5 to obtain a modified Mohr–Coulomb failure criterion

$$\sigma_1 - \sigma_3 = I_d \sigma_{cl} + \sigma_3 (k_s + 1) \sin \phi_s \tag{11}$$

It is assumed that the failure of rock mass due to excavation takes place gradually and progressively from the tunnel wall within the original rock mass, hence, I_d becomes smaller moving from the plastic–elastic boundary towards the tunnel wall. I_d is therefore replaced by a weakening index, I_w, in the plastic zone to account for a post-failure strength reduction. This assumption permits the failure criterion of broken material in the triaxial stress condition to be written as

$$\sigma_1 - \sigma_3 = I_w \sigma_{cl} + \sigma_3 (k_b + 1) \sin \phi_b \tag{12}$$

in which

$$I_w = \frac{\tau_b \cos \phi_i}{\tau_i \cos \phi_b} \tag{13}$$

Where τ_b, ϕ_b, and τ_i, ϕ_i are shear strengths and angles of friction of broken and original *in situ* rock masses respectively, and $0 \leq I_w \leq I_d$.

Both Equations 11 and 12 are useful expressions of modified failure criteria applicable to both the original rock mass and broken materials.

Such failure criteria in triaxial compression are used for the yield zone formulation and the derivation of the governing equations of the radial and tangential stresses. Furthermore, they can also be used to derive theoretical formulae for predicting a tunnel closure profile from the faceline in access tunnels of longwall mining with the following assumptions

1. Virgin stress field is hydrostatic
2. Plane strain conditions exist with the major principal stress circumferential (σ_θ), the minor principal stress radial (σ_r), and the intermediate principal stress (σ_Z) longitudinal
3. The tunnel has a circular cross-section
4. It is driven in a homogeneous, isotropic rock mass

There are several parameters involved in the formulation of a convergence equation and the corresponding yield zone and stress determinations. These parameters and their corresponding symbols are

q = overburden pressure
σ_{cl} = laboratory-determined, uniaxial compressive strength of intact rock
k_s = triaxial stress factor of intact rock
k_b = triaxial stress factor of broken rock
ϕ_s = angle of internal friction of intact rock
ϕ_b = angle of internal friction of broken rock
P_s = support resistance
r_i = initial radius of the tunnel at undeformed state
r_p = radius of the plastic zone

For the case of cylindrical symmetry, the differential equation of equilibrium is

$$\frac{d\sigma_r}{dr}+\frac{(\sigma_r-\sigma_\theta)}{r}=0 \tag{14}$$

assuming that $\sigma_1=\sigma_\theta$ and $\sigma_3=\sigma_r$.

In the plastic zone, the failure criterion applicable to the broken rock must be satisfied. Hence, substituting Equation 12 into Equation 14 and satisfying the boundary conditions $\sigma_r=P_s$ at $r=r_i$ and $\sigma_r=\sigma_{rp}$ at $r=r_p$ after integration yields the following equation for radial stress in the plastic zone, i.e. at $r\leq r_p$

$$\sigma_{rp}=\frac{[P_s(k_b+1)\sin\phi_b+I_w\sigma_{cl}]\left(\frac{r_b}{r_i}\right)^{(k_b+1)\sin\phi_b}-I_w\sigma_{cl}}{(k_b+1)\sin\phi_b} \tag{15}$$

Satisfying Equation 14 for linear–elastic behaviour and the boundary condition $\sigma_r=\sigma_{rp}$ at $r=r_p$ and $\sigma_r=q$ at $r=\infty$ also yields the following equations for the stresses in the plastic zone

$$\sigma_r=q-(q-\sigma_{rp})\,(r_p/r)^2 \tag{16}$$

$$\sigma\theta=q+(q-\sigma_{rp})\,(r_p/r)^2 \tag{17}$$

From Equations 16 and 17 the principal stress difference can be written as follows

$$\sigma_{tp}-\sigma_{rp}=2(q-\sigma_{rp}) \tag{18}$$

Substituting for $\sigma_1=\sigma_p$ and $\sigma_3=\sigma_{rp}$ in Equation 11 and equating the right hand sides of Equations 18 and 11 after simplifications leads to the equation of radial stress at the plastic–elastic boundary

$$\sigma_{rp}=(q-1/2I_d\sigma_{cl})\,(1-\sin\phi_s) \tag{19}$$

Based on finite element stress analysis it has been shown that the shape effect of the opening vanishes after 1.1*D* (*D* = tunnel diameter) from the tunnel wall. It has also been shown that there is no significant difference between arch-shaped tunnels and circular tunnels in terms of stresses and displacements around the immediate opening, except at the tunnel invert. The deformation at the floor of an arch-shaped tunnel is more pronounced than that in a circular tunnel. This is due to higher stress concentration at the corners and also the bending of the floor because of its beam-like behaviour. This causes 10% to 15% more vertical convergence in the arch-shaped tunnel compared to that in the circular tunnel (Majdi, 1988). The stress concentration also increases as the panel is extracted.

Majdi (1988) derived the following empirical equation as a stress concentration criterion in longwall mining. It is based on finite element stress analysis and the *in situ* deformation measurements

$$M_c=\{(0.08(h_s)^{0.55}+0.7)\,[0.002W_0(E_p/E_g)]^{0.4}+1\} \tag{20}$$

where h_s is the extracted seam height, E_p and E_g are the moduli of elasticity of coal pillar and goaf materials respectively and W_0 is the width of extracted panel.

Equation 20 indicates that the pre-mining stress, q, in Equation 19 has been increased by a factor M_c due to panel extraction. Taking this into account, equating the right hand side of Equations 15 and 19 and simplifying results in

$$r_p = r_i\left[\frac{(M_c q - 1/2 I_d \sigma_{cl})(1-\sin\phi_s)(k_b+1)\sin\phi_b + I_w\sigma_{cl}}{P_s(k_b+1)\sin\phi_b + I_w\sigma_{cl}}\right]^{[1/(k_b+1)\sin\phi_b]} \tag{21}$$

Equation 21 can be used to predict the radius of the plastic zone around an access tunnel in longwall mining.

For the case where the cohesion of the materials in the elastic zone is zero the term related to I_w in Equation 21 vanishes. The advantage of Equation 21 is that many parameters, including post-failure behaviour and an *in situ* strength reduction factor, have been taken into account. Fig. 1 illustrates the variation of predicted radius of the plastic zone for different support resistance with depth of cover based on Equation 21.

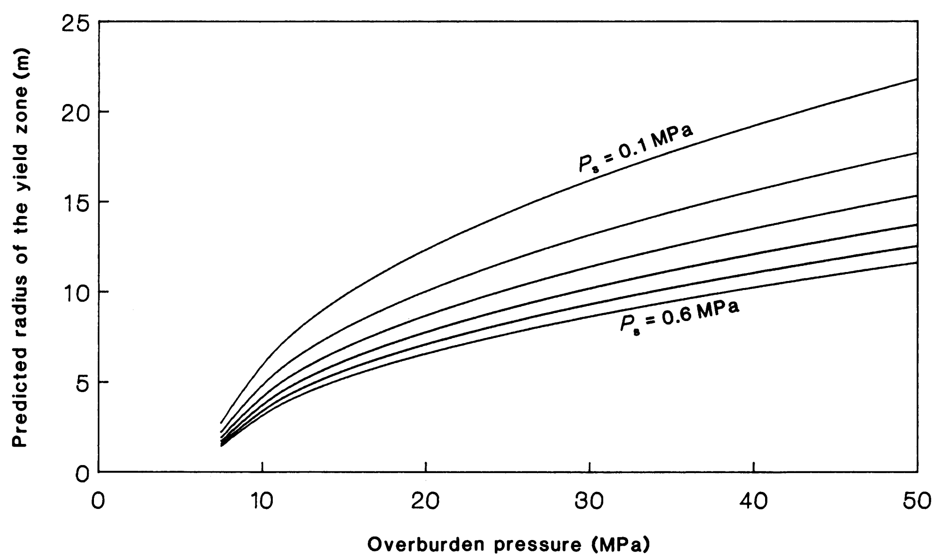

Fig. 1. Variations of radius of the yield zone for different support resistance with depth of cover in longwall mining

To derive equations predicting radial and tangential stresses in the elastic zone, recall Equations 16 and 17. Substituting for σ_{rp} and r_p from Equations 19 and 21 respectively leads to

$$\sigma_r = q - (R_f L_f)(r_i/r)^2 \tag{22}$$

$$\sigma_t = q + (R_f L_f)(r_i/r)^2 \tag{23}$$

where

$$R_f = q\sin\phi_s + 1/2 I_d \sigma_{cl}(1-\sin\phi_s) \tag{24}$$

$$L_f = \left[\frac{M_c q - 1/2 I_d \sigma_{cl})(1-\sin\phi_s)(k_b+1)\sin\phi_b + I_w\sigma_{cl}}{P_s(k_b+1)\sin\phi_b + I_w\sigma_{cl}}\right]^{[2/(k_b+1)\sin\phi_b]} \tag{25}$$

Theoretical deformation analysis

According to Hooke's law the stress–strain relationship in polar coordinates for plane strain conditions can be written as follows

$$\varepsilon_r = [\sigma_r - v\sigma_\theta - v^2(\sigma_r + \sigma_\theta)]/E \tag{26}$$

$$\varepsilon_\theta = [\sigma_\theta - v\sigma_r - v^2(\sigma_r + \sigma_\theta)]/E \tag{27}$$

The strain–displacement relationship can be written as

$$\varepsilon_r = \frac{\partial u_r}{\partial r} \tag{28}$$

$$\varepsilon_\theta = \frac{u_r}{r} \tag{29}$$

where r = radial distance from the centre of the tunnel and u_r = radial displacement at the plastic–elastic boundary, i.e. at $r = r_p$, $u_r = u_{rp}$.

Hence, the tangential strain at the plastic–elastic boundary can be written as

$$\varepsilon_{\theta p} = \frac{u_{rp}}{r_p} \tag{30}$$

Substituting for ε_p from Equation 27 leads to

$$\frac{u_{rp}}{r_p} = [\sigma_\theta - v\sigma_r - v^2(\sigma_r + \sigma_\theta)]/E \tag{31}$$

The radial displacement, u_{rp}, of the plastic–elastic boundary due to excavation, when q decreases to σ_{rp} at $r = r_p$ will be that induced by the change in the stress field. From Equations 22 and 23 the changes of the radial and tangential stresses at the plastic–elastic boundary are given as

$$\Delta\sigma_r = -(R_f L_f)(r_i/r_p)^2 \tag{32}$$

$$\Delta\sigma_\theta = (R_f L_f)(r_i/r_p)^2 \tag{33}$$

where R_f and L_f are given in Equations 24 and 25 respectively. Substituting the changes of these stresses for tangential and radial stresses in Equation 31 and simplifying results in

$$u_r = \frac{1+v}{E_i}(R_f L_f)(r_i/r)^2 \tag{34}$$

where E_i is the *in situ* determined equivalent vertical modulus of elasticity of the stratified rock strata (Muir-Wood, 1979). This is also a function of *in situ* confining pressure (Hobbs, 1968; Santarelli and Brown, 1987), and *in situ* structural defects (Kulhawy and Rose, 1979). In this paper, E_i has been formulated to account for the above factors. This leads to the following

$$E_{\mathrm{i}} = \frac{(1+v)^2(1-2v)}{I_{\mathrm{d}}\mathrm{E}_{\mathrm{l}}(1-v)} \tag{35}$$

in which E_{l} is the laboratory-determined modulus of elasticity and I_{d} was given by Equations 9 and 10.

Equation 34 can be used to predict the elastic displacement of the elastic–plastic boundary. The displacement of the access tunnel boundary at $r = r_{\mathrm{i}}$ is larger than that at the elastic–plastic boundary. This is due to volume expansion and associated with the passage of the rock from a solid state, or a densely interlocked state, to a broken state (Ladanyi, 1974). Ladanyi (1974) and Wilson (1980) derived an expression that can be used to approximate this expansion. Wilson suggested that the immediate expansion can be added to the elastic displacement, hence Wilson's expression can be rewritten

$$u = u_{\mathrm{rp}}(r_{\mathrm{p}}/r)^{(1+e_{\mathrm{f}})} \tag{36}$$

in which u is the radial displacement at radius $r = r_{\mathrm{i}}$ and e_{f} is an expansion factor lying between 0.0 an 0.50 depending upon the type of rock mass (Wilson, 1980). Substituting Equation 34 for u_{rp} in Equation 36 leads to

$$u = d_{\mathrm{i}} \frac{(1+v)^2(1-2v)}{I_{\mathrm{d}}E_{\mathrm{l}}(1-v)} R_{\mathrm{f}} J_{\mathrm{f}} \tag{37}$$

in which d_{i} is the initial diameter of the tunnel and

$$J_{\mathrm{f}} = \left[\frac{M_{\mathrm{c}}q - 1/2I_{d}\sigma_{\mathrm{cl}})(1-\sin\phi_{\mathrm{s}})(k_{\mathrm{b}}+1)\sin\phi_{\mathrm{b}} + I_{\mathrm{w}}\sigma_{\mathrm{cl}}}{P_{\mathrm{s}}(k_{\mathrm{b}}+1)\sin\phi_{\mathrm{b}} + I_{\mathrm{w}}\sigma_{\mathrm{cl}}}\right]^{[2+e_{\mathrm{f}}/(k_{\mathrm{b}}+1)\sin\phi_{\mathrm{b}}]} \tag{38}$$

Equation 37 can be used to predict the diametric closure of the circular tunnels in longwall mining.

Variations of the diametral tunnel closure for different support resistance and depth of cover based on Equation 37 is illustrated in Fig. 2.

The above derivations are based on plane strain conditions, therefore, Equation 37 represents tunnel displacement far away from the faceline. Majdi *et al.* (1986) have also shown that the vertical convergence profile, VC (%) from the faceline can be obtained by employing the following exponential function

$$\mathrm{VC} = a[1 - \exp(-x/b)] + c \tag{39}$$

in which a, b and c are constants and x is the distance from the faceline. The constant a represents the total vertical convergence which may be taken as equal to the calculated displacement based on plane strain conditions, by Equation 37. The constant b which was calculated based on the least squares technique, yields an average value of 2.3 D for single tunnels and (2.3 $D + W_0/4$) for access tunnels in the longwall mining, in which W_0 = the panel width. The constant c represents two components of vertical displacement, i.e. $c = c_1 + c_2$. One component represents the time-dependent displacement which may be written in the following form (Panet, 1979)

$$c_1 = G[1 - \exp(-t/T)] \tag{40}$$

in which G is a constant representing the maximum time-dependent deformation when t approaches infinity, and T is a time factor.

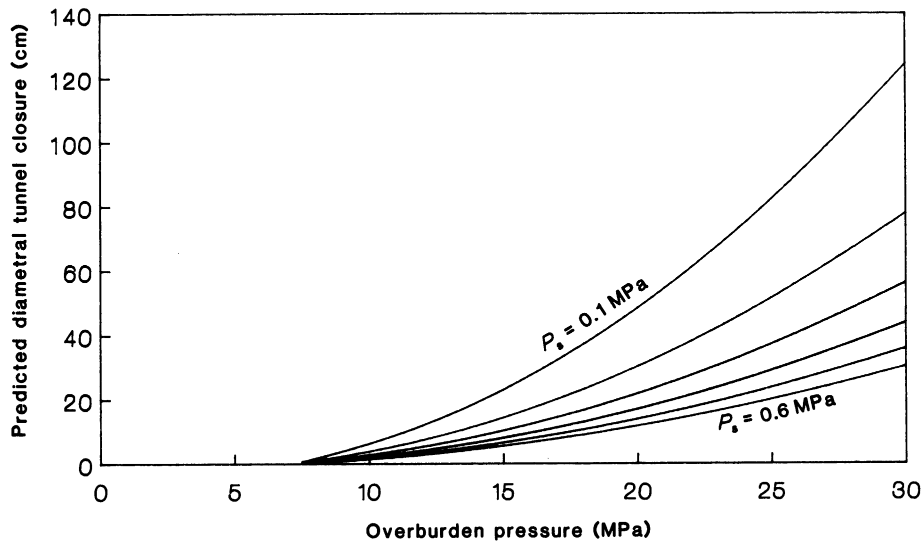

Fig. 2. Variations of diametral tunnel closure for different support resistance with depth of cover in longwall mining

The second component of c, i.e. c_2, represents the vertical displacement which may be induced due to face–end design (Blades and Whittaker, 1972; Anon, 1982).

Substituting Equations 37 and $c=c_1+c_2$ into Equation 39 after required simplifications yields

$$\mathrm{VC}=\left\{\frac{(1+v)^2(1-2v)}{I_\mathrm{d}E_\mathrm{l}(1-v)}[R_\mathrm{f}J_\mathrm{f}]\left[1-\exp\left(\frac{-4x}{9.2D+W_0}\right)\right]+c_1+c_2\right\}(100) \quad (41)$$

Equation 41 is a new equation which can be used to predict tunnel convergence profiles from the faceline in both single and multiple circular openings in longwall mining environments.

Based on a finite element analysis, Equation 41 can also be used for convergence prediction in non-circular tunnels by introducing a new term as a coefficient of compensation, C_c, to take the shape effect into account (Majdi, 1988). This coefficient depends upon the tunnel shape, lying between 0.1 to 0.15 for arch-shaped, 0.2 to 0.3 for rectangle-shaped and zero for circle-shaped. The tunnel diameter also should be replaced by tunnel width, therefore, Equation 41 can be rewritten in the general form

$$\mathrm{VC}=\left\{\frac{(1+v)^2(1-2v)}{I_\mathrm{d}E_\mathrm{l}(1-v)}[R_\mathrm{f}J_\mathrm{f}]\left[1-\exp\left(\frac{-4x}{9.2D+W_0}\right)\right]+c_1+c_2\right\}x(1+C_\mathrm{c})x(100) \quad (42)$$

in which R_f, E_l, J_f and c_1 have been given by Equations 24, 35, 38 and 40, respectively. The term W_0 does not exist where there is no longwall panel. Practical application of the convergence equation developed herein, Equation 42, with illustrated examples are presented in the following section.

Practical applications

Example for an arch-shaped access tunnel

Consider the following data from the B4W coal road of the Lingan mine, Nova Scotia, to investigate access tunnel behaviour in terms of vertical convergence after support installation and longwall extraction

Average laboratory determined uniaxial compressive strength of surrounding rock strata, $\sigma_{cl}=30$ MPa
Average laboratory determined modulus of elasticity of the first 10 m of roof and the first 5 m of floor strata, $E_r=E_f=15$ GPa
Average laboratory determined modulus of elasticity of coal seam, $E_c=4$ GPa
Average laboratory determined modulus of elasticity of compacted broken rock in the goaf area, $E_g=1$ GPa
Average Poisson's ratio of surrounding materials, $v=0.3$
Assumed support resistance, $P_s=0.1$ MPa
Assumed *in situ* structural defect index of the surrounding rock strata, $I_d=0.3$
Assumed weakening index of the broken rocks, $I_w=0.01$
Depth of cover, $D_c=420$ m
Initial tunnel width, $W_t=5$ m
Arch-shaped tunnel with initial height, $h_t=3.5$ m
Average unit weight of the roof strata, $\gamma=25$ KN m^{-3}
Average extracted panel width, $W_0=220$ m
Average extracted seam height, $h_s=2$ m
Assumed average expansion factor of yielded materials in the plastic zone, $e_f=0.2$
Average angle of internal friction of the solid rock strata, $\phi_s=37°$
Average angle of friction of broken rocks, $\phi_b=30°$
Assumed coefficient of compensation for tunnel shape, $C_c=0.15$

Solution

To calculate the vertical convergence by Equation 42, the equivalent vertical modulus of elasticity must be calculated. This can be achieved either by assuming a global average value or by calculating the equivalent elastic moduli of ribside and goafside separately, then to be averaged. In the latter case, the following equation is recommended for stratified strata by Muir-Wood (1979)

$$1/E= (\Sigma t/E)/(\Sigma t) \tag{43}$$

On this basis, $E_{\text{ribside}}=11$ GPa and $E_{\text{goafside}}=3$ GPa. Back substitution of the above data in Equations 20, 24 and 38 for determination of M_c, R_f and J_f, respectively, results in values of $M_c=2.112$, $R_f=8.10$ and $J_f=43.97$. Substituting the required data in Equation 4 yields a maximum total vertical convergence, VC, of 25.16% as compared with 27.19% obtained by the least squares method. The variation of this vertical convergence from the tunnel faceline is illustrated in Fig. 3 and is compared with both *in situ* measurements and the best-fit curve obtained from the non-linear least squares method.

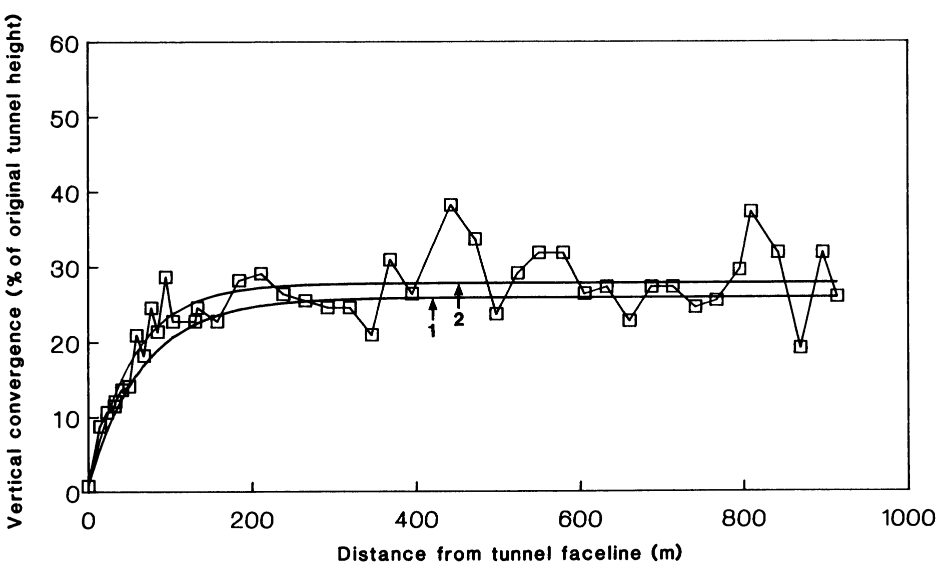

Fig. 3. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for B2W coal road at Lingan mine. (□) *in situ* measurements; (1) theoretical prediction; (2) best-fit curve obtained by least-squares method

Example for circular access tunnel

Consider the calculation of the vertical convergence of a circular main access tunnel at 7.6 m in diameter, and at 204 m depth as in the Donkin-Morien mine, Nova Scotia. The following data, based on the average values for the surrounding rock strata and support resistance were also available

$$E_l = 12\ \text{GPa},\ \sigma_{cl} = 40\ \text{MPa},\ \phi_s = 37°,\ \phi_b = 30°,\ \nu = 0.25,\ e_f = 0.2,$$
$$\gamma = 0.25\ \text{KN m}^{-3},\ I_d = 0.3,\ I_w = 0.01\ \text{and}\ P_s = 0.5\ \text{MPa}$$

Solution

In this case, since there is no longwall operation, the influence of longwall extraction on the access tunnel convergence is zero, therefore in Equation 20 $M_c = 1$. The access tunnel excavated is circular, hence the coefficient of compensation, C_c, in Equation 42 due to the shape effect is also zero. Now, following steps similar to Example 1 for calculating R_f and J_f, gives $R_f = 4.85$ and $J_f = 0.61$. Finally, substituting the given data in Equation 42 yields a maximum total vertical closure of 17.23 mm as compared with 15.9 mm from *in situ* measurement by extensometers. The profile of calculated vertical closure from the faceline compared to that from the *in situ* deformation measurements is shown in Fig. 4.

Verification of the convergence equation by field measurements

The convergence equation developed in this paper is further investigated with regard to its validity and applicability when applied to arch-shaped tunnels. *In situ* deformation

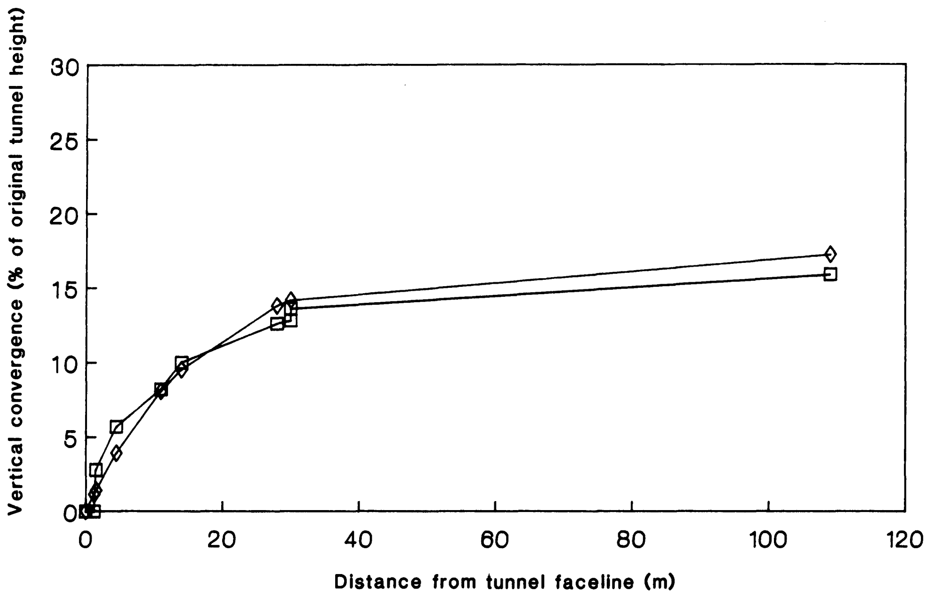

Fig. 4. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for main access tunnel at Donkin-Morien. (□) *in situ* measurements; (◇) theoretical prediction

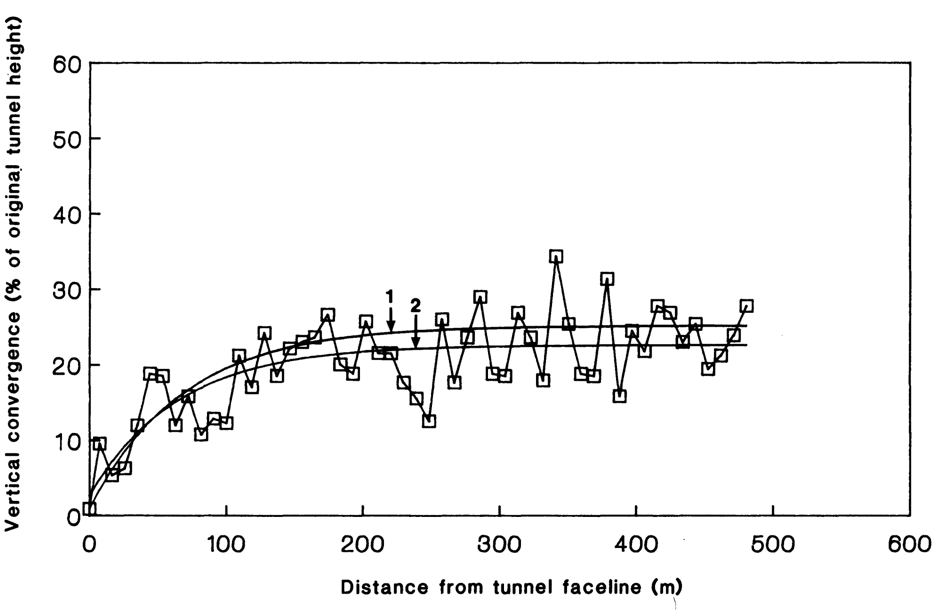

Fig. 5. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for B4W coal road at Lingan mine. (□) *in situ* measurements; (1) theoretical prediction; (2) best-fit curve obtained by least-squares method

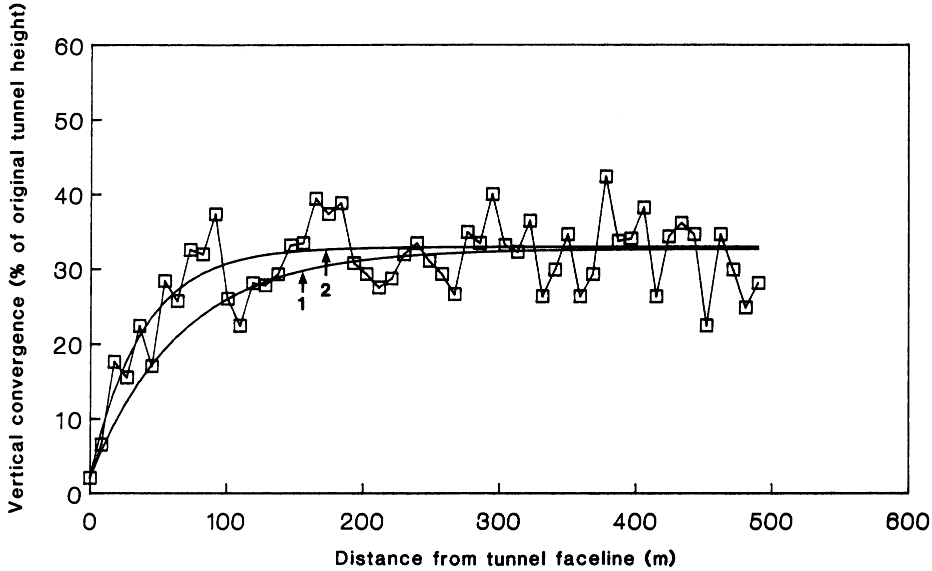

Fig. 6. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for B6E coal road at Lingan mine. (□) *in situ* measurements; (1) theoretical prediction; (2) best-fit curve obtained by least-squares method

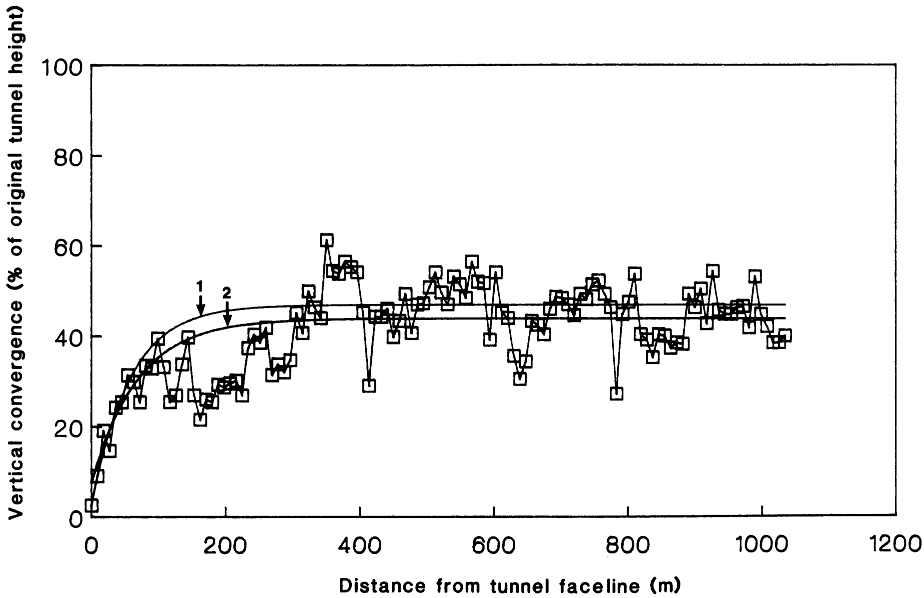

Fig. 7. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for T13S material road at Lingan mine. (□) *in situ* measurements; (1) theoretical prediction; (2) best-fit curve obtained by least-squares method

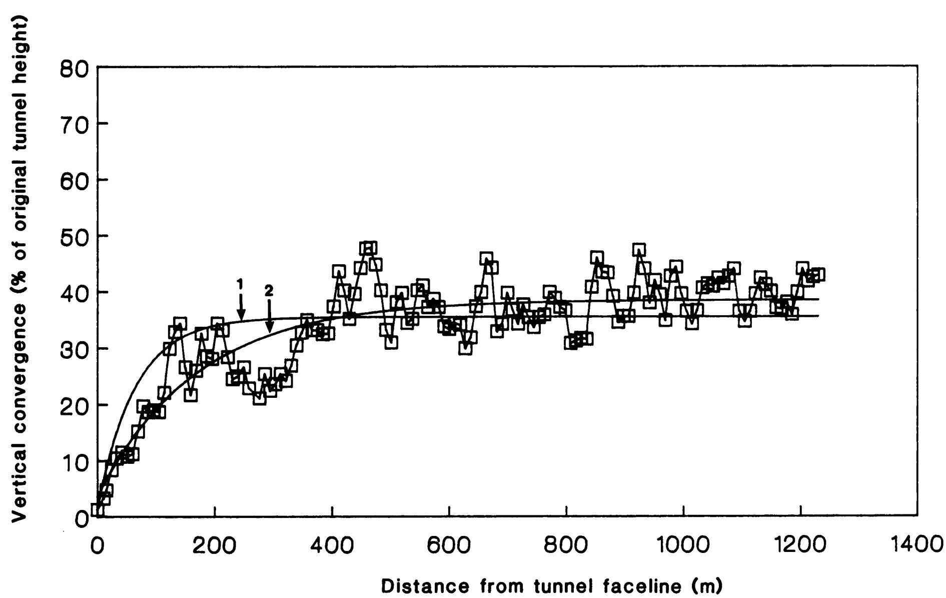

Fig. 8. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for T13S material road at No. 26 colliery. (□) *in situ* measurements; (1) theoretical prediction; (2) best-fit curve obtained by least-squares method

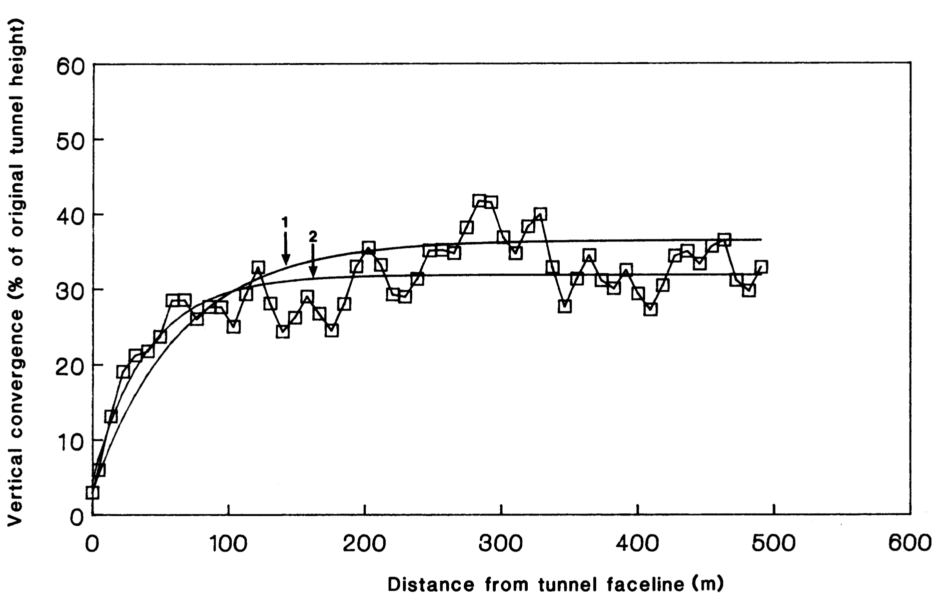

Fig. 9. Comparison of theoretical prediction of vertical convergence with *in situ* measurements for B13S coal road at No. 26 colliery. (□) *in situ* measurements; (1) theoretical prediction; (2) best-fit curve obtained by least-squares method

measurements from both the Lingan and No. 26 mines, Nova Scotia, have been compared to theoretical predictions. Figs 5 and 6 represent the face-distance vertical convergence profiles of the B4W and B6E coal roads in the Lingan mine. Additionally, the corresponding theoretical predictions and the best-fit curves obtained by the non-linear least squares technique are also superimposed. It can be seen that the theoretical predictions are in good accord with *in situ* measurements. Similarly, Figs 7, 8 and 9 illustrate the vertical convergence profiles from the access tunnels facelines for B12S and B13S coal roads, and T13S material road in the No. 26 mine. Comparison of the data indicates the validity and the range of applicability of the convergence equation. The theoretical prediction shows a good agreement with the *in situ* data with a maximum difference up to 5% as compared with the best-fit curve. This indicates the versatility of the developed equation being applicable to both circle- and arch-shaped tunnels. Finally, based on Equation 42, Figs 10(a) to (c) have been produced for different mining conditions to reflect the variation in vertical convergence from the tunnel faceline with respect to the major controlling factors.

Conclusions

A theoretical formulation for the prediction of vertical convergence profiles from the faceline in the longwall mining access tunnels has been developed. The method presented takes into account the effect of panel width, extracted seam height, elastic moduli of the goaf material and coal pillar, tunnel shape, tunnel size and depth of cover in addition to strength characteristics of surrounding strata. Furthermore, the method has been validated with *in situ* deformation data from 26 face-access tunnels in mines of the Cape Breton coalfield. The results predicted by this method have shown good accord with the *in situ* deformation data.

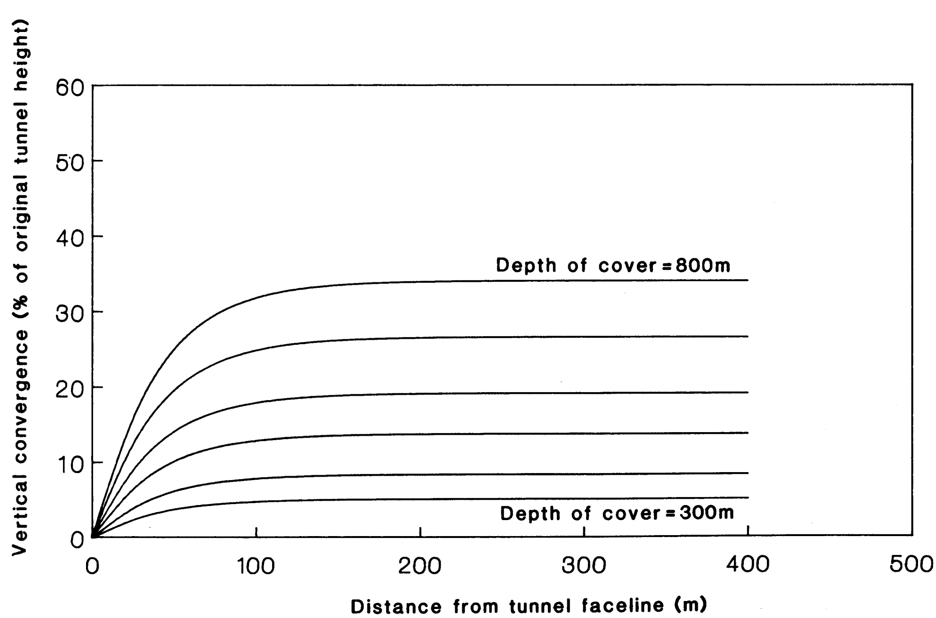

Fig. 10(a)

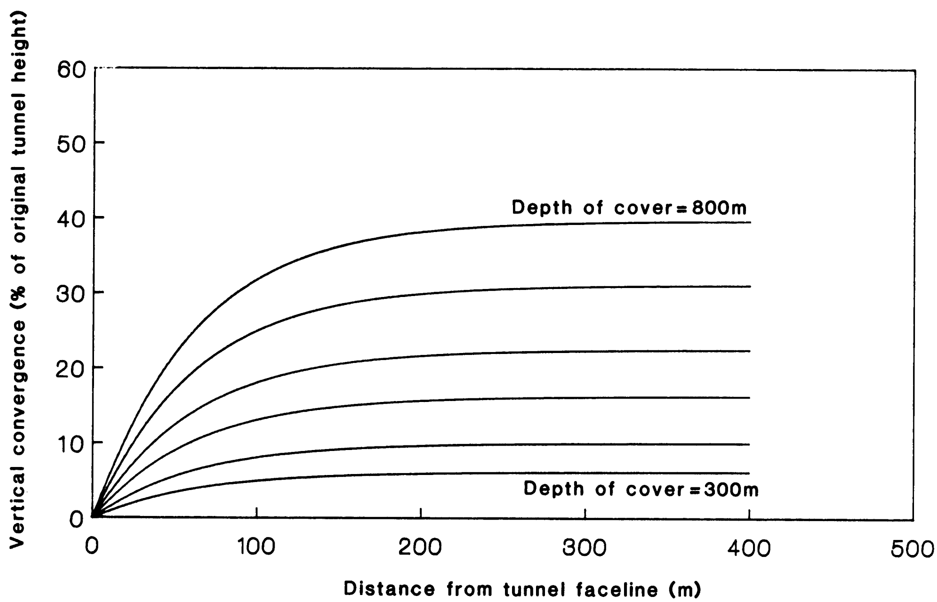

Fig. 10(b)

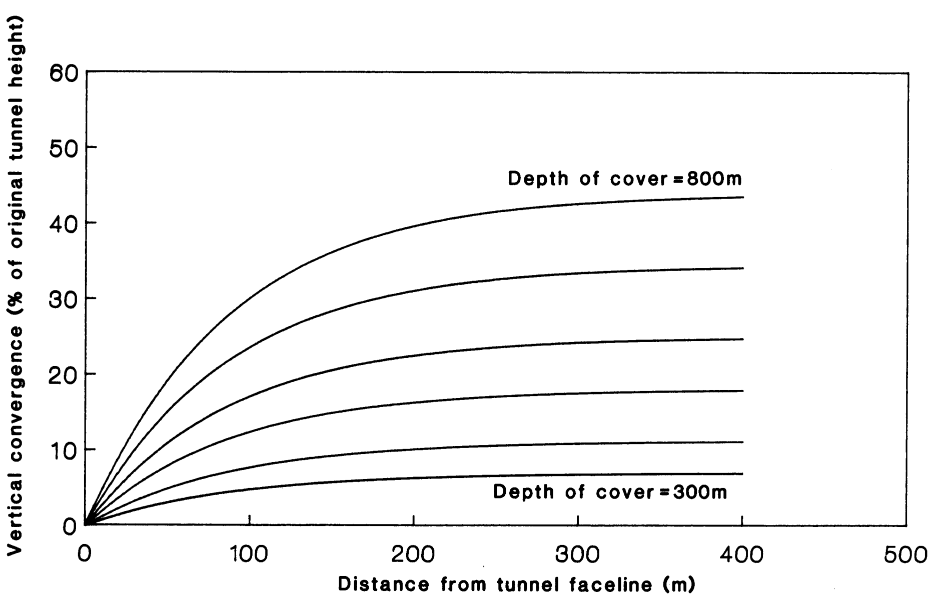

Fig. 10(c)

Fig. 10. Theoretical prediction of vertical convergence profile of access tunnels at different depth of cover for extracted panel of (a) 100 m, (b) 200 m and (c) 300 m. $\sigma_{cl}=200$ MPa; $P_s=0.1$ MPa; $h_s=2$ m; $W_t=5$ m

Based on this method, a series of vertical convergence profiles for different depths of cover has been produced.

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