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The buckling failure of tunnels induced by high horizontal stress in jointed rocks

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ABSTRACT: The buckling failure of tunnels in jointed rock by high horizontal stress is illustrated and numerically investigated. In simulating the buckling failure, the homogenous model is applied in combination with the discrete modeling of some key joints near the tunnel wall. Using the proposed procedure, a mouth-shaped, unlined tunnel driven in jointed rock with a family of horizontal joints is numerically computed.

1 INTRODUCTION

Under certain circumstances, stability problems, such as buckling and bulging, are of significance in rock engineering and mining practice. This phenomenon has been observed in situ, see e.g. Stille et al (1981). Fairhurst & Cook (1966) investigated this problem theoretically under some postulations, whereas model tests were conducted by the Australian Coal Industries Research Laboratory for simulating the structural and stress conditions in the coalfields near Sydney/Australia. In Fig. 1b), the buckling of slabs in the roof and floor of an excavation in a high horizontal stress field is illustrated, from Hoek & Brown 1982.

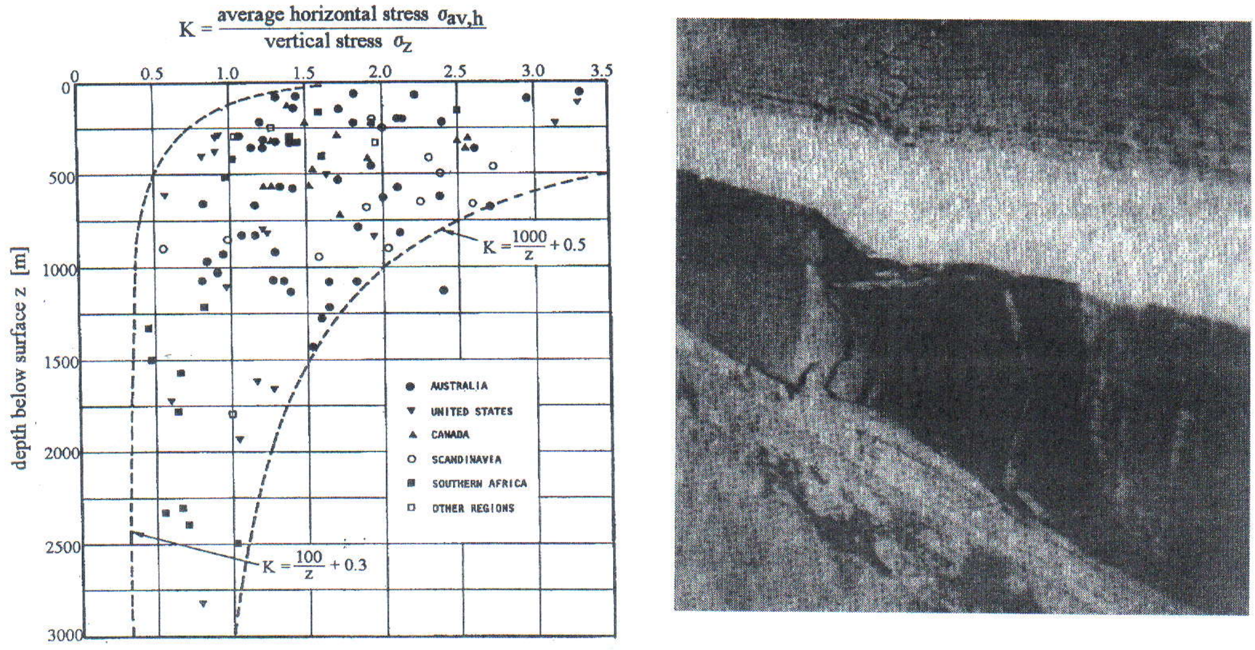
The essential loading and boundary condition for the buckling failure is the uniaxial compressive stress along to thin rock slabs containing discontinuities parallel to free boundary surface of underground openings. Until now, it is a common way to evaluate the safety against the buckling failure by using stability theory. Rock slab is seen as plate under axial compressive loading. This approach can only be applied, when the real axial stress is determined. This is, however, not always the case. The secondary stress in rock mass is dependent on the geometric profile of openings, the E-modul of rock slabs, the shear strength in rock and along discontinuities as well as the primary stress state. The strength failure in rock and on discontinuities due to underground opening lead to the further stress redistribution in rock mass and may have a significant influence on the failure mode. Therefore, it is important to consider all these factors in analyzing such problems.

In this paper, a procedure is illustrated investigating the buckling failure of a mouth-shaped, unlined tunnel driven in jointed rock. The primary stress state is assumed to be anisotropy, and has a higher horizontal stress arising from geological effect and rest tectonic stress. For simulating the buckling failure of the tunnel, the homogeneous model is applied in combination with the discrete model, where the material and geometric nonlinear behavior is taken into consideration by using finite elements and joint elements.

2 DETAILS OF THE PROBLEM

The statistical results of in situ stress measurements in various parts of the world, as Hoek & Brown reported (1982), indicate that in general the vertical stresses in rock mass can be determined considering the overlying weight of rock at corresponding depths. Compared to this, the measured horizontal stresses may not be predicted using elastic theory. Furthermore, the horizontal stresses are often larger than the vertical stresses at depths of less than 500 m, see Fig. 1 a). The construction of underground openings under such primary stress state may lead to the strong stress concentrations at the roof and at the invert. On the assumption that the encountered rock mass contains a family of horizontal discontinuities, buckling failure may occur at roof and floor slabs (see Fig. 2).

Fig. 2 illustrates a mouth-shaped, unlined tunnel having a height of 10 m and a width of 14 m. The tunnel is driven in a rock mass containing a family of horizontal joints. The primary horizontal stress is 4 MN/m² and the vertical stress 2 MN/m². The joints have an average spacing of 0.3 m. Rock is assumed to be elastic and strength failure is only possible along joints.



a) b)

Figure 1. a) The ratio of average horizontal stress to vertical stress dependent on depth below surface; b) Buckling of roof and floor slabs in a coal mine model subjected to high horizontal stress (Model by Australian Coal Industries Research Laboratory). from Hoek & Brown (1980).

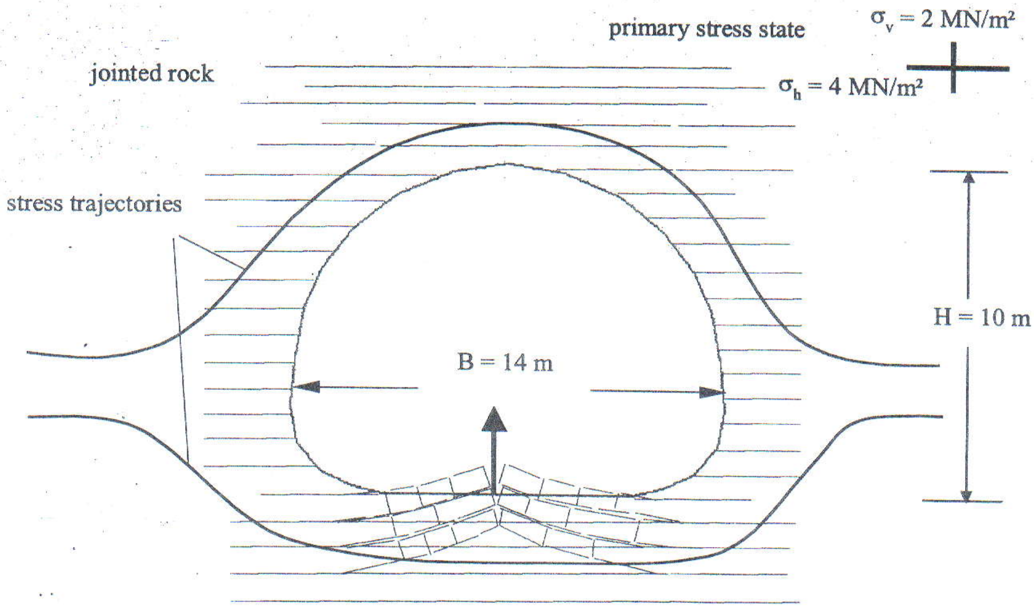


Figure 2. Buckling failure of tunnel in jointed rock under high horizontal stress.

Depending on the secondary stresses and the shear strength of the joints, two failure modes may be possible under the given conditions. If the shear strength on joints as well as dilatation angle are small, the high shear stress acting on the joints in the areas between the side wall of the tunnel and the roof as well as the invert (see the stress trajectories round tunnel in Fig. 2) can be transmitted to a less extent. In this case, the deformations outside the tunnel wall during the plastic stress redistribution activate the shear carrying capacity on the joints in ever more remote areas. The resulted large horizontal displacement on the tunnel wall as well as the corresponding extension of plastic zones may be seen as a strength failure mode. In addition, this plastic redistribution leads to the decrease of the horizontal stress in the bottom of the tunnel and reduces the possibility of the buckling failure of the slabs at the invert. For the case that the shear parameters on joints are relatively high and the plastic extension outside the tunnel wall very limited, the buckling failure mode becomes possible, because at the invert not only the stress condition, but also the geometric condition are fulfilled. Furthermore, the spacing of the encountered joints is relatively small compared to the width of the tunnel. For investigating the buckling failure of the tunnel instead of the strength failure, the suitable parameters were chosen and are given in Table 1.

Table 1. The assumed geometrical and mechanical parameters of the jointed rock

rock:	$E = 5000/2000/1500$ MN/m²; $\upsilon = 0.2$.
joints:	$d = 0.3$ m; $\alpha = 0°$; $\beta = 0°$; $c = 0$; $\varphi = 32°$; $\psi = 20°$.

3 CALCULATION MODEL

Under the assumption that the dimension of underground opening is much larger than that of joint spacing, jointed rock may be idealized as a continuum material (homogeneous model), see e.g. Zienkiewicz & Pande (1977). In this replacement material, the real spacing of the discontinuities exists no longer, and each point within this material behaves mechanically same, whereas the orientation (striking and dip angles of the discontinuities) remains for considering the deformation and strength anisotropy. Separate treatment of joints becomes necessary, if the opening or large sliding of joints may occur. In this case, the combination of continuum and discrete models seems to be computationally economical. That is, the discrete modeling is applied to the area where the joints must be individually simulated, whereas the other area of the jointed rock is modeled by using the continuum theory, see Hu (1997).

Compared to this approach, the distinct element method was developed for discontinuum analysis in rock mechanics about thirty years ago, see e.g. Cundall (1988). Here, jointed rock mass is represented as an assemblage discrete blocks. All joints in rock mass are individually treated and viewed as interfaces between distinct rock blocks. The corresponding contact forces and displacements at the interfaces are determined through a iterative procedure. For many years, however, it was seen as not-yet-proven numerical technique and has not been applied so extensively as conventional continuum analysis technique. In the recent years, the theoretical further refinement and the development of the software related to this method were made. More and more rock engineering projects are analyzed using this technique. In the near future, it may become a generally recognized tool in the analysis of rock engineering as the continuum theory.

In the presented paper, the homogenous model is applied with the special treatment for some key joints in rock. For the analysis of the buckling failure of rock slabs, the geometrically nonlinear theory is used. Arising from the updated Lagrangian formulation and the elasto-viscoplastic theory, the controlling equation can be expressed and converted into a finite element formulation:

$$([K_{L,t}]+[K_{N,t}])\cdot\{\Delta U\} = \{R_t(t+\Delta t)\}-\{F_t\}+\{\Delta F_t^{vp}\}$$

$[K_{L,t}]$	:	linear stiffness matrix referred to the configuration at step t;
$[K_{N,t}]$	:	nonlinear stiffness matrix referred to the configuration at step t;
$\{R_t(t+\Delta t)\}$	:	total force applied at step t+Δt referred to the configuration at step t;
$\{F_t\}$	:	equivalent internal force vector at step t;
$\{\Delta F_t^{vp}\}$	:	equivalent visco-plastic force vector at step t+Δt referred to the configuration at step t.

Upon the finite element equation, a finite element program has been developed for analyzing the deformation and stability of underground openings in jointed rock. In addition, "joint element" has also been implemented which makes the separate treatment of some key joints possible.

4 NUMERICAL CALCULATION

4.1 *Computation cross section and FE-mesh*

The problem illustrated in Fig. 2 was analyzed using the calculation procedure and program described above. Fig. 3 illustrates the chosen computational cross section and FE-mesh. For the most part of the cross section, 8-node finite element elements were used for discretization, whereas for a special part below the bottom (3 m thick, 5 m wide) the rock slabs and joints were considered using finite elements and joint elements, respectively.

The construction was computationally simulated with 6 calculation step. In the first step the primary stress state before the construction was established. The following 5 steps simulated the 5 part excavations. The designed FE-mesh consists of 881 nodes, 242 finite elements as well as 110 joint elements.

4.2 *Exemplary calculation results*

For finding the critical point related to the buckling failure of the invert, parameter study was carried out by varying the E-modul of the jointed rock. In Fig. 4, the distributions of the displacements on the tun-

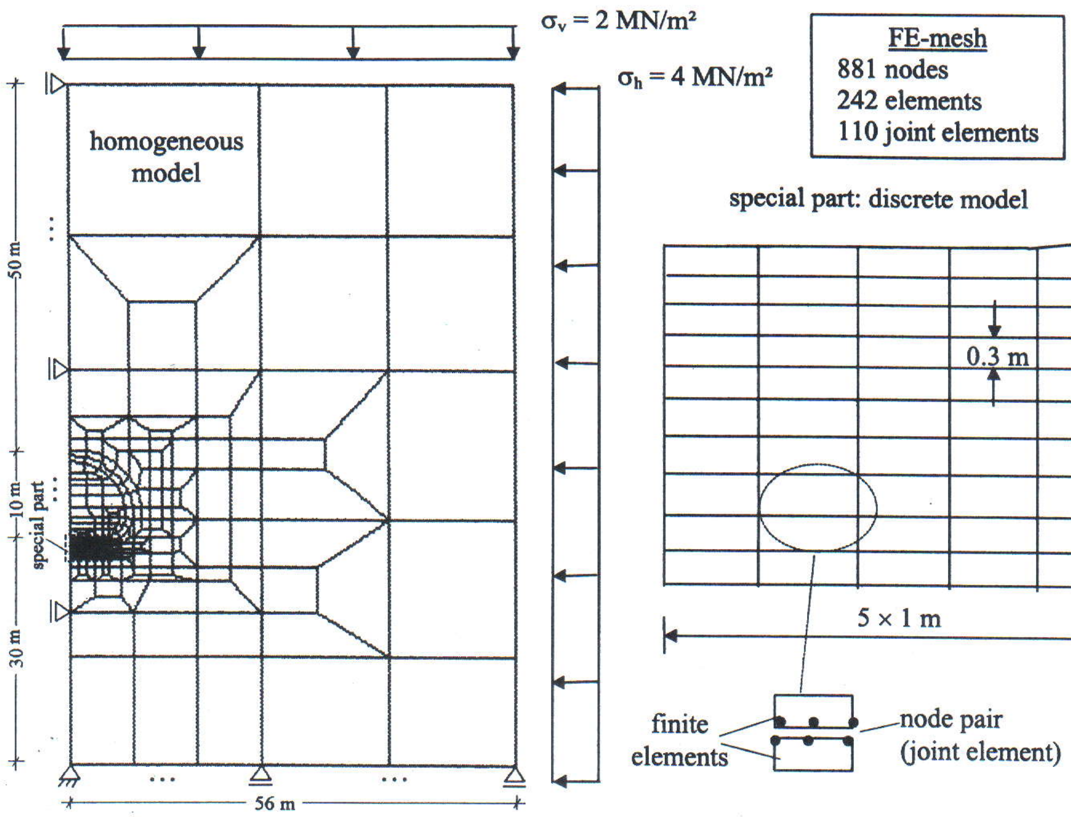


Figure 3. Computational cross-section and FE-mesh.

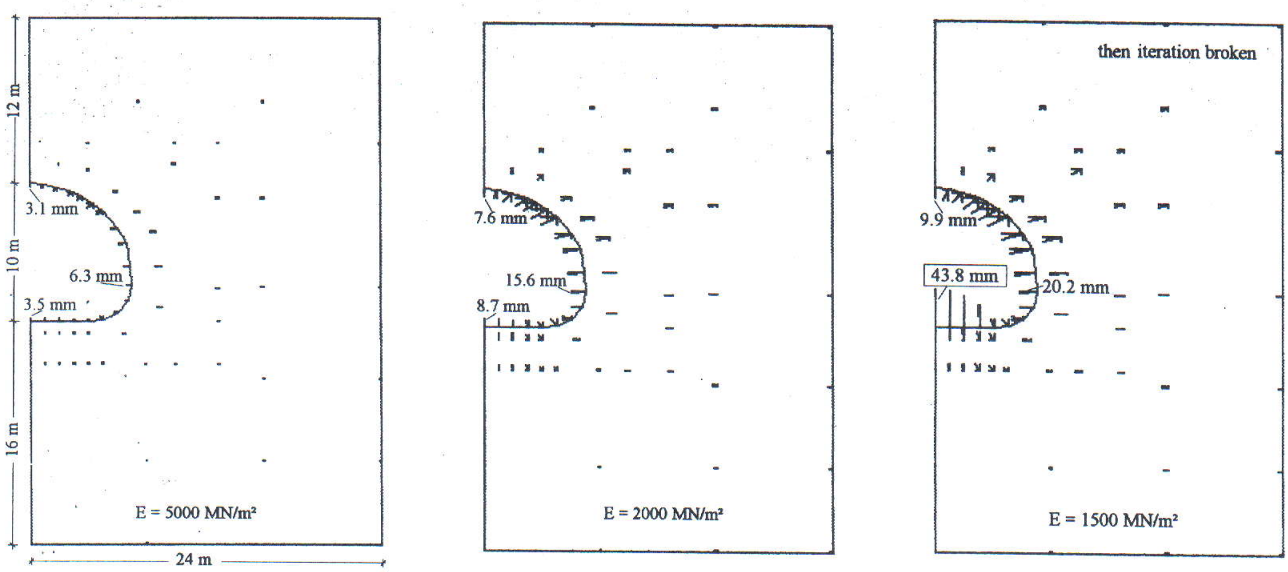


Figure 4. Comparison of the displacement on the surface of tunnel, E = 5000/2000/1500 MN/m².

nel surface as well as in the rock mass are compared for E = 5000/2000/1500 MN/m². It can be clearly seen that the change of the displacements on the roof and the side wall is not drastic for the three cases. However, an overproportional increase of the heaving at the bottom of the tunnel appears, when E = 1500 MN/m² is approached. This means the failure state in relation to the buckling failure is reached. The stress distribution before the interruption of the iteration is presented in Fig. 5 for this case. The plastic zones at the roof and the bottom are not very extended outside as expected. In this state, the joints below the bottom are mostly under small compressive stress and experience some relative shear displacements. The buckling takes place suddenly, when the contacts between the rock slabs are lost

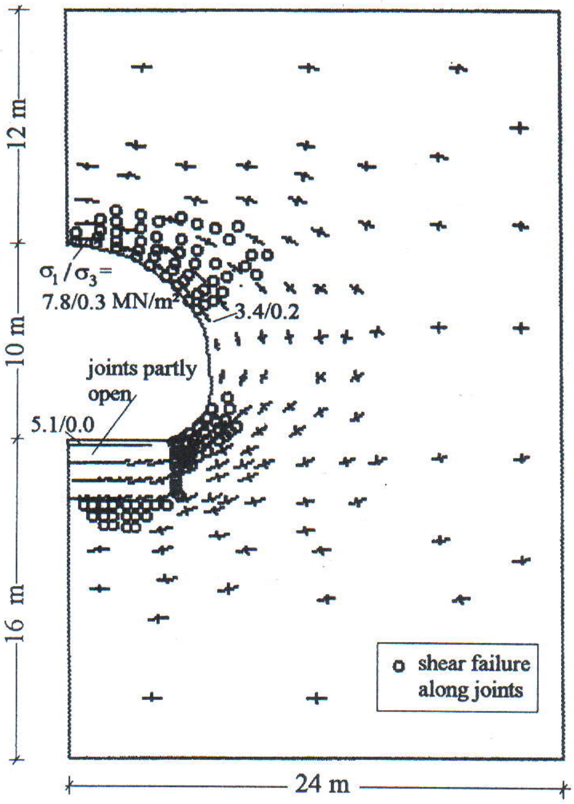


Figure 5. Stress distribution shortly before the interruption of the iteration, E = 1500 MN/m².

through minor disturbance. In the calculation, the iteration is interrupted due to the very small values in some elements of the stiffness matrix.

5 CONCLUSIONS

This investigation related to the buckling failure of underground openings is basically upon the continuum theory considering the material and geometrical nonlinear behavior of jointed rocks. For the area where buckling failure may appear, however, the joints and rock were separately simulated using joint elements and finite elements (discrete model). This combination makes the computation effectively and economically. Depending on the strength, orientation and spacing of joints, the failure mode of tunnel may be different, in some cases in combined form. The computational example presented is a mouth-shaped, unlined tunnel in a rock mass containing a family of horizontal joints. The primary stress state shows a ratio of horizontal stress to vertical stress being 2. Under the assumption of the relatively high shear parameters along joints, the buckling failure mode at the invert appears, when the E-modul of the rock is reduced down to 1500 MN/². This is illustrated by the overproportional increase of the heaving at the bottom. The numerical iteration was then interrupted due to the drastic decrease in the stiffness of some elements. In this state the calculation is not stabile indicating the buckling failure of the tunnel.

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