VOUSSOIR BEAM RESPONSE OF BEDDED LIMESTONE ROOFS IN GREEK UNDERGROUND MINING EXCAVATIONS

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ABSTRACT: When an underground opening is driven into well bedded rock formations, the lower layers of the immediate roof, will almost always be intersected by pre-existing or induced cracks, thus allowing a discontinuous beam arching action to take place. The stability of such roofs is evaluated through a compiled computer code which simulates them as voussoir beams. Characteristic correlations between important parameters such as the modulus of deformation of the rock mass, the span of the opening, the strata layer thickness and the modes of failure involved are demonstrated. A typical chart for the theoretical prediction of stable and unstable voussoir beam roofs is checked against numerous observed failures in Greek bauxite mines.

1 INTRODUCTION

Roof failures and consequent falls have been always considered to be the major and most important source of fatalities and serious injuries in underground mining and civil engineering works. Although safer excavation procedures, modern production methods, sophisticated techniques and efficient training have significantly contributed to the general goal of reducing such serious accidents, it is also essential that engineers, foremen and even underground workers increase their understanding and theoretical knowledge of the basic mechanics of roof failure.

The introduction of complicated mining equipment and the lack of shallow deposits have necessitated mining in larger working spans and at increasing depths. The main aim of modern mining activities is to establish operations characterized by optimum ore recovery, fast production rates, improved safety and minimum support costs. Modern standards in the construction of tunnels and large storage caverns exhibit the general requirement of reliable structural design. Ground control for underground workings in bedded formations has become increasingly complex and requires a rather more rational scientific approach.

The engineering significance of "bedded rock" is defined as applying to those rocks in which the bed thickness is small compared to the roof span and the bond between the layers is weak, so that bed separation and slip may occur (Adler and Sun, 1976). These rocks are usually involved in the construction of underground openings within layered formations i.e. limestones, sandstones, shales, conglomerates etc., as well as in mining of rather flat dipping, bedded deposits of coal, potash, trona, bauxite, uranium, copper, iron ore etc.

Outcrops of well bedded rock formations are quite often in the Greek mainland and numerous underground workings are completely or partially driven into such rocks. Almost all Greek bauxite and nickel mines as well as a great part of numerous civil tunnels are surrounded by clearly stratified rock formations.

When an underground opening is driven into bedded rock formations, the main effects due to the overburden loads can be estimated as follows : At remote distances, subsidence may occur on surface. At intermediate distances, a pressure dome or arch structure is developed, and at close-in distances, the immediate roof will deflect downward in beam or plate action (Adler and Sun, 1976).

In the past, ground control design was based mainly on observations, practice and experience and sometimes a rock mass classification system was used, especially for the estimation of support requirements of tunnels. Ground control was based on concepts of continuous elastic solid mechanics. Linear elastic continuous beam theory assumes that the rock of the immediate roof behaves as a series of continuous beams or plates loaded by their own weight. The roof span was then designed for a specified allowable tensile stress of the rock. Nevertheless, observations showed that roof strata, although cracked transversely, resist bending moments which are larger than evaluated by linear elastic analysis. Such beams are formed in the roof of excavations driven within well bedded rock. There, the rock mass is separated at the bedding planes due to sagging and tensile cracks occur at the exposed surface of the roof. It is necessary for the engineer, before starting any theoretical analyses, to go back to the rock and find out what actually happens. This approach may not be very fascinating to him because it comprises dirty experimental work and rather dangerous observations, but that's what is really needed.

The existing openings contain the real conditions on a full scale, just asking to be observed, measured and analyzed. These beds cannot be simulated adequately by continuous, linear, elastic beams. The lower strata which will almost always be intersected by pre-existing or induced cracks, will allow a discontinuous beam arching action to take place. No one discontinuous beam model may simulate accurately roof response but it will approach the discrete phase comprehensively. A simple such model is the voussoir beam.

2 VOUSSOIR BEAM MECHANICS

Voussoir beam theory came on stream in order to throw a new light on some of the many factors involved in roof control. It recognizes that in a confined situation the ultimate strength of a beam is larger than its evaluated elastic strength and that pre-existing cross fractures may not allow tensile stresses. Frictional resistance of the joint surfaces should provide adequate shear strength for the stability of the beams. Thus, the beam which consists of a no-tension material, is assumed to carry its own weight by arching. The term "arching" does not denote exclusively the formation of an arch or a dome-shaped opening. It mainly refers to the process by which fractured rocks may become partially self-supporting by the development of a compression zone above an opening, which transfers vertical load to the abutments on each side (Adler and Sun, 1976).

A voussoir beam is a beam composed of individual blocks (voussoirs). This beam may be considered as a limiting condition of a voussoir arch of zero curvature. Such arches are used in structures such as masonry bridges. In Fig. 1 the in situ conditions of an opening driven into bedded rock formations are illustrated. An idealized opening with a roof beam, crossed by vertical joints which separate the voussoirs, is shown in Fig.2. These joints may carry negligible tension and have a low shear resistance. This renders them inappropriate for ordinary beam analysis. Vertical loading of such horizontal (or subhorizontal) beams will develop cracks underneath at midspan and over the abutments, which in turn will cause vertical movement at midspan and rotation at the abutments.

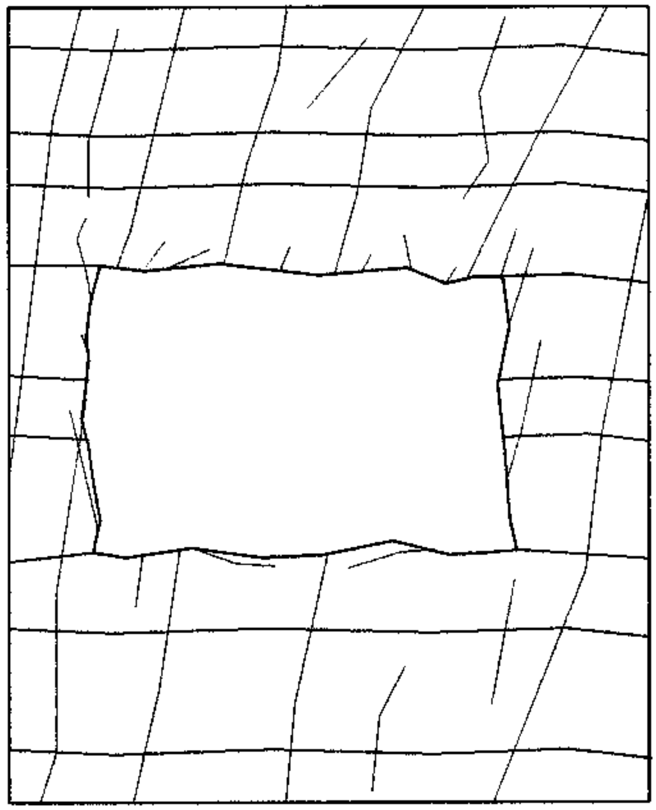

Figure 1. Prototype underground excavation in layered jointed rock.

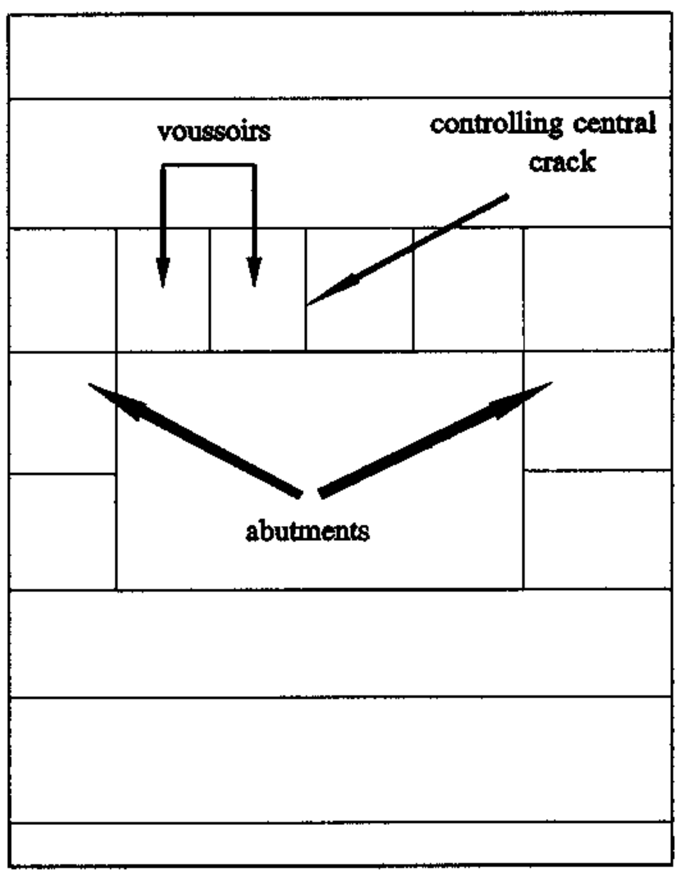

Figure 2. Idealized opening with Voussoir beam roof.

This, will squeeze the lower section of the beam at the abutments and the upper section at midspan, which will develop progressively an arch shaped thrust line (Fig.3). Movement will continue until the formed arch will be capable of withstanding the acting load.

γ : is the unit weight of the rock,
s : is the clear distance between the abutments,
n : is the ratio of the thickness of the arch, at midspan or at the abutments, to the thickness t of the beam,
z_f : is the moment arm of the thrust line after movement,
z_o : is the moment arm of the thrust line before movement.

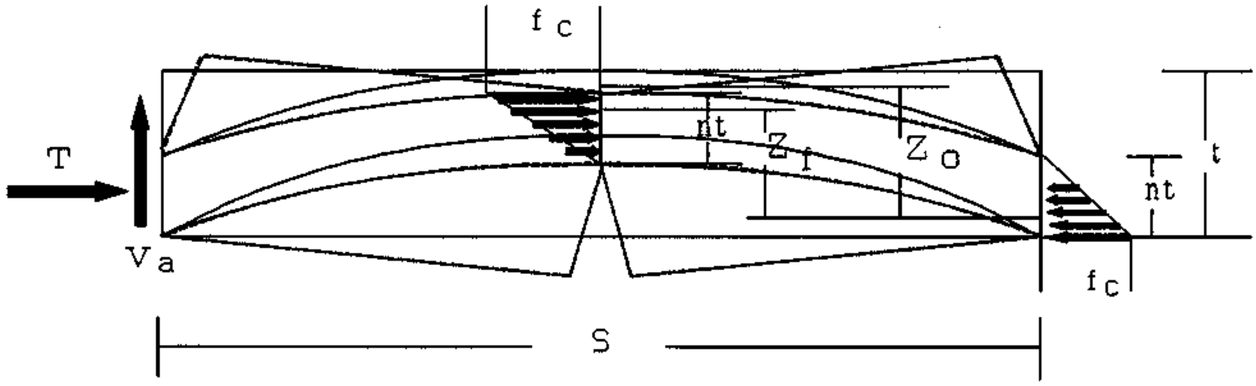

Figure 3. Voussoir beam model.

Three modes of failure may be considered. The first assumes that the developed lateral stress will be greater than the compressive strength of the rock mass. The second assumes that buckling of the beam occurs. The third assumes that shear failure occurs at the abutments. Stability analysis examines the safety against each mode of failure.

Evans (1940-41) and Beer and Meek (1982) developed models to simulate the behaviour of such beams formed in bedded formations. The assumptions made were :

a. Rock is an elastic no-tension material.
b. A thrust zone will be formed with an arch shaped thrust line.
c. The thickness of the formed thrust zone is the same at the midspan and at the abutments.
d. The distribution of normal stress at any cross section is triangular.
e. The arch is loaded by the weight of the beam only.

In order to attain equilibrium of the arch the following equations have to be satisfied :

$$f_c = 0.25 \cdot \gamma \cdot s^2 / (n \cdot z_f) \qquad (1)$$

$$z_o = t \cdot (1 - n \cdot 2/3) \qquad (2)$$

$$0 < n < 1 \qquad (2a)$$

$$1/3 < z_o/t < 1 \qquad (2b)$$

f_c : is the maximum stress at the abutment or the midspan,

The final moment arm z_f may be evaluated by considering the elastic squeezing of the beam due to the vertical movement at midspan and the lateral constraint at the abutments. In order to evaluate this squeezing, a certain distribution of stress along the beam and a parabolic shape of the thrust line have been assumed. Thus the following equation (Brady and Brown, 1993) is developed :

$$z_f/s = \sqrt{\{(z_o/s)^2 - f_c.(1/3+n/4)[3/16+(z_o/s)^2]/E'\}} \qquad (3)$$

$$f_c/E' < 2/\ (2/3 + n/2) \qquad (3a)$$

$$f_c/E' < (z_o/s)^2 \ / \ \{(1/3 + n/4).[3/16 + (z_o/s)^2]\} \qquad (3b)$$

E': is the Young's modulus of the rock mass in compression.

Inequality 3b establishes an upper bound for the strain f_c/E'. There is a simple infinity of possible solutions that satisfy equations 1 through 3. Each solution corresponds to a specific thrust line which may be characterized say by the ratio n. For each thrust line, the values of z_f, z_o and f_c are determinate and may be evaluated by equations 1 through 3. The solution thrust line corresponds to a stationary minimum value of f_c.

In order to satisfy against the first mode of failure, the value of f_c has to be correlated to the uniaxial compressive strength q_u of the rock mass. The chosen values of the uniaxial compressive strength and the modulus of deformation have to consider any anisotropic behaviour of the rock mass, due to the existence of any weakness planes. The factor of safety against compression failure FS_c is given by :

$$FS_c = q_u/f_c \qquad (4)$$

In order to satisfy against the second mode of failure the derivative of the moment of resistance of the arch due to an increase in deflection has to be positive. For z_f less or equal to zero this is violated. Inequality 3b ensures that z_f is positive, but it is only necessary and not sufficient for preventing the second mode of failure.

The possibility against shear failure at the abutments may be evaluated directly. The frictional resistance force V_R that may be mobilized is :

$$V_R = (1/2)f_c.n.t.\tan\phi \qquad (5)$$

ϕ : the angle of internal friction of the rock mass.

The shear force V_a acting at the abutment is given by:

$$V_a = (1/2).\gamma.s.t \qquad (6)$$

Then the factor of safety FS_s against shear failure at the abutments may be defined by :

$$FS_s = V_R/V_a = [f_c.n/(\gamma.s)].\tan\phi \qquad (7)$$

3 IN SITU OBSERVATIONS

Bauxite deposits are abundant in the Greek mainland and intense mining operations expand within the geosyncline of Parnassos - Giona - Helicon -Oiti mountains in Central Greece, the main axis of which has a northwest - southeast orientation. The prevailing rocks in these regions generally consist of white, usually non crystalline limestones which stratigraphically belong to the Maestrichtian period. Apart from the upper strata which are rather blocky, the underlying limestone formations are usually clearly thin bedded. The majority of the bauxite deposits, which are intensively mined nowadays, are underlying dark coloured roudists bearing bituminous limestones of the Touronian-Senonian period.

Mechanized room and pillar mining is the usual mining method. Mining excavations are supported by abandoned ore pillars. Room layout and dimensions are designed under the general requirement that working stresses at any point within the surrounding rock should not cause under any circumstances roof or pillar failure, at least until the end of the depillaring stage, when gradual reduction of pillar initial dimensions takes place. The stability of the limestone roof is of dominant importance for the safe exploitation of the deposits, due to the quite frequent entry of personnel and equipment into the mined void. The main task of roof control depends on the unmined mineral which forms the pillars and on the self supporting ability of the rock mass of the roof, which is intensively reinforced by systematic and mechanized rock bolting.

The initial development (advancing) of the deposit involves the extraction of the ore by excavating 5 meter wide rectangular rooms and leaving 8 meter wide square pillars of ore. During the retreating stage, pillar sidewalls are reduced to 5 meters in each direction.

The Touronian limestones forming the immediate roof of the mining rooms, are generally considered to be relatively competent rocks. Hence, the mine layout and the reinforcement system selection is highly dependent on the persistence and the frequency of the main joint sets. However, despite the ordinary crossing of the rock mass of the roof by clear bedding planes and cross fractures, the roof appears to behave satisfactory.

Laboratory tests on intact rock specimens evaluated a uniaxial compressive strength of the limestone rock mass in the range of 70 to 150MPa (with a mean value of 100MPa), a Young's Modulus between 6-14GPa, a Poisson's ratio between 0.25-0.35, a unit weight of 26.5KN/m^3 and a Point Load Index of about 4. The angle of internal friction is found to be between 33°-40°. The limestone has an RQD value within 50 and 75 and an average strata layer thickness of 0.2-1.0m. The usual condition of the main discontinuity sets may be characterized by the relatively rough surfaces, the separation of no more than 1mm, filled sometimes with soft clayey or calcite material. Most of the joints dissecting the rock mass are characterized as fair to favourable for the majority of the main access and development drifts, provided that they have been aligned properly. In general, groundwater appears as interstitial, with the exception of some winter months.

Due to the impermeability and deferrification of the bauxite orebodies, groundwater percolating through the limestone of the hangingwall, lays down clayey marl sediments, particularly in the areas of high joint density. These sediments have formed at the contact of the bauxite deposits and the limestone roof a relatively thin yellow clayey marl layer, 20-50cm thick, completely distinguished from the deposit and the hangingwall which is almost always scaled by mechanical means.

Besides the bedding planes of the roof, one or two (depending on the region) natural subvertical (75°-85°) main joint sets and sometimes a subordinate flat dipping subset may be distinguished. Limestone roofs are also suffering by nearby production blasting vibrations, while they are usually dissected by a karstic network containing numerous irregularly branching and interconnected channels. These features necessitate the modelling of the cracked roof beams as voussoir ones.

The overburden of the mining excavations varies with respect to the surface relief, but the majority of the mine workings takes place at a depth of about 300m under a gravitational stress field of more than 8MPa.

The Rock Mass Rating, according to CSIR, is evaluated to be in the range of 55 to 70, which characterizes the limestone as fair to good rock. This suggests a mean standup time of about 6 months for a 4m span. Rock support interaction analyses, as suggested by Hoek & Brown (1980), have been implemented successfully to the main access drifts of the bauxite mines. The rock mass strength parameters m and s were estimated for these openings in the range of 2.0-0.5 and 0.05-0.0005 respectively (Economopoulos et al,1992).

Gradual sagging of the immediate roof layers (not always clearly visible, as these beds usually bend very little before they rupture), opening or propagation of critical fractures (usually from the center of the beam to the abutments) and shearing at the abutments renders the roof appropriate to voussoir beam analysis. Whenever the beam is relatively thin, it breaks through. In areas where the lateral stress is inadequate and the vertical load on the roof beam significant, the lower stratum is subjected to high tensile stress and usually fractures. This happens also when the roof beam consists of incompetent rock and the abutments may not provide sufficient lateral confinement. The arch action of the beam and any gradually growing non self supporting portion (weight) of the lower separated strata may overstress the rockbolts at the center of the room, which behave as drawbolts, thus encouraging further separation (Economopoulos et al, 1993).

Usually, there are very few cases where the visible transverse cracks of the roof are open initially, i.e. immediately after the complete exposure of the first roof layer. In most cases, although the tensile strength of the limestone rock mass has been exceeded and cracks are densely spaced, horizontal thrust holds the blocks tightly together.

The opening's span to bed thickness ratio varies within the range of 5 to 30, with a most characteristic mean value of 15. The maximum critical width, during the retreating stage, for almost all mining roofs with bed thickness of about 0.5m, is about 12m. Some critical transverse cracks may be observed in the external surface of the roof. Although small debris of rock are sometimes falling, the majority of such roofs remains stable for a long time, allowing the safe completion of the pertinent mining activities.

4 APPLICATIONS

A computer code is compiled in order to analyze the stability of a voussoir beam by using the equations described above. In this analysis a stationary minimum value of the maximum normal stress f_c is directly pursued for values of n greater than 0.5. For each thrust line, which corresponds to a value of n, stability against buckling is explicitly checked. Input parameters are the thickness of the strata layer t, the clear distance s between the abutments, the modulus of deformation E' and the unconfined compressive strength q_u of the rock mass, the angle of internal friction ϕ of the fractures and the unit weight γ of the rock mass. It is noted that the values of E' and q_u should be obtained either directly by in situ testings parallel to bedding or by laboratory tests after considering the appropriate scale effect and the directions of the main joint sets. Initially, the maximum normal stress f_c and the corresponding deflection Δ is calculated. At the same time, the evidence of buckling failure is investigated. Finally, the factors of safety against compression and shear failure are evaluated. The solution procedure involves sequential calculation of n, z_0, f_c, z_f and Δ. The unit weight of the limestone rock mass is taken as $26kN/m^3$.

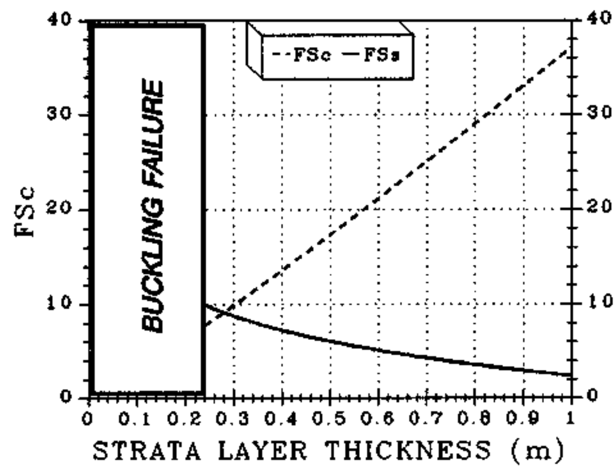

Figure 4. Factors of safety for compression and shear against strata layer thickness (E'=4GPa, q_u=40MPa, ϕ=35°).

In Fig.4, the thickness t of the strata layer is correlated to the factors of safety against compression and shear and to the evidence of buckling. Input parameters are E'=4000MPa, q_u=40MPa, ϕ=35° and s=8m. Buckling occurs for values of t less than 0.2m. Increasing values of t cause a rapid increase in FS_c and a decrease in FS_s.

In Fig.5, the span s of the opening is correlated to the factors of safety against compression and shear and to the evidence of buckling. Input parameters are E'=4000MPa, q_u=40MPa, ϕ=35° and t=0.5m. Buckling occurs for values of s greater than 15m. Increasing values of s cause an increase in FS_s and a

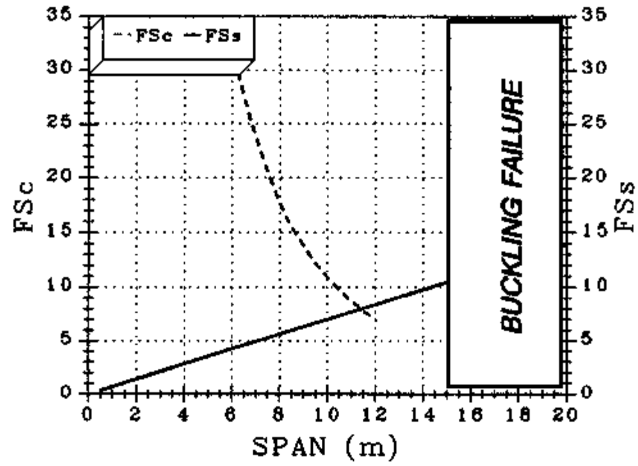

Figure 5. Factors of safety for compression and shear against the span of the opening (E'=4GPa, q_u=40MPa, t=0.5m, ϕ=35°).

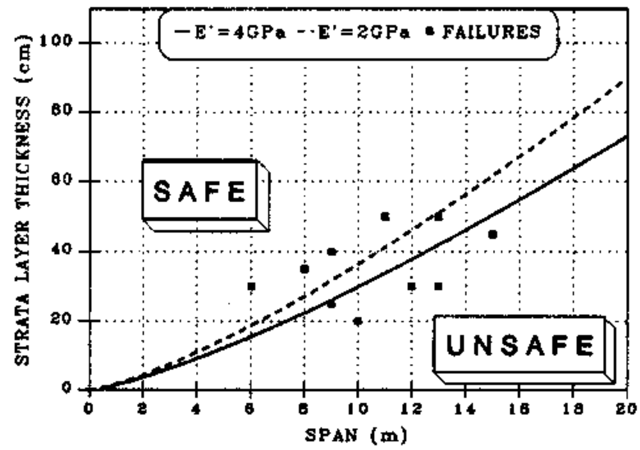

Figure 7. Stable and unstable conditions of Voussoir beam roofs against buckling.

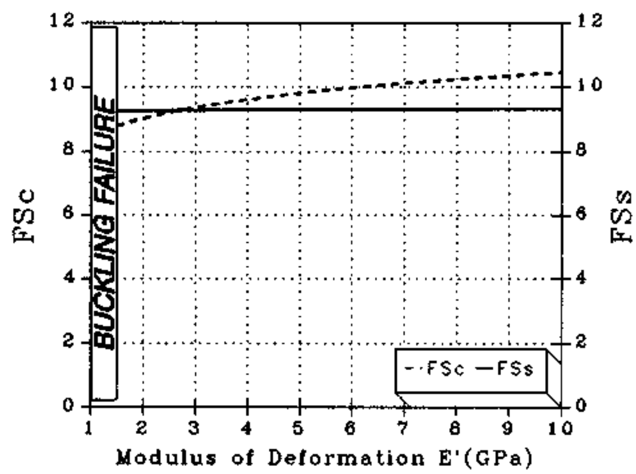

Figure 6. Factors of safety for compression and shear against the Modulus of Deformation (t=0.3m, s=8m, q_u=40MPa, ϕ=35°).

decrease in FS_c.

In Fig.6, the modulus of deformation E' is correlated to the factors of safety against compression and shear and to the evidence of buckling. Input parameters are q_u=40MPa, ϕ=35°, t=0.3m and s=8m. Buckling occurs for values of E' less than 1500MPa. Increasing values of E' cause a slight increase in FS_c, while FS_s remains constant.

In Fig.7, stable and unstable regions in buckling for values of s less than 20m and t less than 1m, are determined. This diagram assumes two values of E', i.e. 2000 and 4000MPa which are estimated to correspond to the long term value of E' for the particular limestone roofs and hangingwalls. The calculated value of f_c nowhere in the chart exceeds 17MPa. The thickness of the roof layers may be easily estimated at the site either during drilling operations or by obseving fallen layers of rock. Numerous observed failures in the mines, have been projected in the same figure. A comparison between the model results and the real failure conditions shows a satisfactory correlation.

5 CONCLUSIONS

The prevailing ground conditions in an opening, and especially in an actual bauxite mine network, are complicated due to the difficult to determine mechanical parameters and the involved unpredicted factors. The stability of the fractured roof beam layers is evaluated through an ad hoc compiled computer code which simulates them as voussoir beams. This modelling established correlations between the main design parameters, as encountered in Greek underground bauxite mines, and the three modes of failure involved.

A typical chart for the prediction of stable and unstable situations of the limestone beam roof in buckling, adapted to the real mining conditions, is provided. This chart is checked on numerous reported failures. These failures lie nearby the prediction lines; nevertheless there are reported failures within the safe region. This may be attributed to the dynamic forces acting on the beam, mainly due to production blasting, to the compliance in rotation at the abutments, mainly due to the clayey marl layer between the roof and the bauxite pillars, and to the rotation due to insufficient bearing, at the square (not rib) shaped pillars. Further research and monitoring is necessary for the evolution of the theoretical curves into reliable design charts.

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