ROCK ANCHORS - DESIGN AND QUALITY CONTROL

ANCRAGES EN ROCHER - LE CALCUL ET LE CONTRÔLE DE QUALITÉ

FELSANKER - ENTWURF UND QUALITÄTS KONTROLLE

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ABSTRACT

The paper reviews some design rules and quality controls associated with prestressed, cement grouted rock anchors. Design data, relating to uplift capacity, rock/grout bond, grout/tendon bond, and tendon, are appraised with special reference to the choice of safety factors.

For comparison, the results of relevant theoretical and experimental investigations are presented, which tend to contradict the fundamental assumptions of uniform interfacial stress distribution commonly made by designers.

On site quality control measures are strongly recommended, and guidance is provided on permissible drilling tolerances, waterproofing, grouting and final stressing.

ABSTRACTION

Ce papier analyse des règles de calcul et des contrôles de qualité associés aux ancrages précontraints en rocher, injectés à coulis. Des informations de calcul au sujet de la résistance à soulèvement, du scellement au rocher, du scellement à l'acier, et de la câble, sont évaluées particulièrement en ce qui concerne le choix des coefficients de securité.

Pour comparer, les resultats des investigations applicables, théoretiques et experimentales, sont présentés qui semblent contredire les suppositions fondamentales de l'uniformité des scellements, généralement faites par les ingénieurs. Des mesures de la contrôle de qualité à pied d'oeuvre, sont fortement recommanoées et des conseils sont donnés au sujet des tolérances permises de perforation, des systèmes d'hydrofuge, et de la mise en tension finale.

ZUSAMMENFASSUNG

Die Schrift zeigt eine Ubersicht über Entwürfe und Qualitäts Kontrollen in Zusammenhangmit vorgespannten Injektionsanker im Fels. Die Ausfuhrungsdaten beziehen sich auf die Abhebungfähigkeit, auf die Fels-Mörtel Grenzen, die Mörtel-Stahl Grenzen, und die zuggliede, welche abgschätzt werden, unter besondere Berücksichtigung zu der Wahl der Sicherheitsfaktoren.

Zum Vergleich werden die Resultate von zutreiffenden theorischen und experimentalen Untersuchungen angeboten, die dazu neigen, in Gegensatz zu den fundamentalen Annahmen der einheitlichen Spannungsverteilung zu stehen, welche im allgemeinen bei Entwürfen gemacht werden.

Es wird besonders empfohlen die Qualität des Bauplatzwerkes zu Kontrollieren. Für die zulässige Bohrungstoleranz, die Wasserdichtung, das Mörteln und für die engultige Beanspruchnahme wird eine Anleitung angeboten.

INTRODUCTION

Although rock anchors have been used successfully for many years in connection with the prestressing of dams, roof strata control, and slope stabilisation, in recent years the range of applications has widened considerably. This is due in part to the success achieved by soil anchors in tying back retaining walls, holding down dock floors, and pile testing. Now, rock anchors are expected to perform without difficulty, even when installed in relatively poor quality, weathered, or laminated, rock. In addition, there is a trend towards higher load capacities for individual and concentrated groups of anchors. For the higher dams in vogue today, prestressing of the order of 2 MN per metre may be required, necessitating individual anchors of capacity well in excess of 10 MN. In the field of suspension bridges concentrated groups of anchors with a working capacity of 60 MN are already being seriously considered, and design loads of 150 MN are anticipated in the future. Even in strong competent rocks, these high prestress levels are demanding engineering judgements in areas where no relevant precedents exist.

Bearing these points in mind, the authors believe that there is a growing need to establish and employ reliable design formulae with realistic safety factors. During construction, quality controls should be agreed, and stressing procedures standardised since it is the tensioning operation which finally tests the anchor and demonstrates its safety.

The purpose of this review is to describe current practices in relation to rock anchors by drawing on the experience gained in various countries over the past 30 years. It is intended that the paper should form a basis for discussion since the validity of the basic design assumptions is questioned, and the lack of knowledge of full scale anchor performance is highlighted.

DESIGN

General

A grouted rock anchor may fail in one or more of the following modes:

(a) By failure within the rock mass,
(b) By failure of the rock/grout bond,
(c) By failure of the grout/tendon bond, or
(d) By failure of the steel tendon, or top anchorage.

Therefore, in order to establish the overall safety factor for the anchor, each of the above phenomena must be considered in turn. In reviewing the main design concepts, it should be emphasised that these concepts relate primarily to prestressed cement grout injection anchors, which have been constructed in a vertical, or steeply inclined downwards, direction.

Uplift Capacity

The assessment of the overall stability, or uplift capacity, of an anchor is carried out in order to ensure that failure of the rock mass surrounding the anchor does not occur. In the case of single anchors, most engineers assume that at failure an inverted cone of rock is pulled out of the rock mass (Fig. 1).

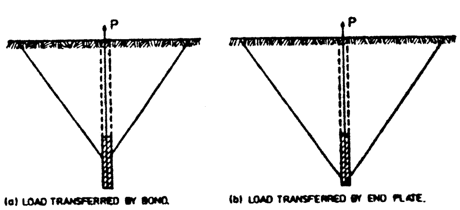

FIGURE 1. GEOMETRY OF CONE, ASSUMED TO BE MOBILISED WHEN FAILURE OCCURS IN A HOMOGENEOUS ROCK MASS.

The uplift capacity is normally equated to the weight of the specified cone and where the ground is situated beneath the water table, the submerged weight of rock is used. The depth of anchor calculated in this way may, of course, be reduced where it can be demonstrated by test anchors that the applied prestress can be otherwise resisted safely. The effect in groups of anchors is the production of a flat vertical plane at the interface of adjoining cones (Fig. 2). As the spacing for a single line of anchors reduces further, a simple continuous wedge failure in the rock is assumed.

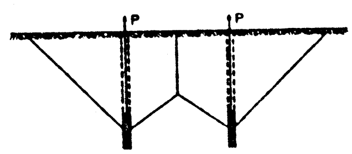

FIGURE 2. INTERACTION OF INVERTED CONES IN AN OVERALL STABILITY ANALYSIS.

However, although the shape of the failure volume is widely agreed, its position with respect to the grouted fixed anchor length (socket) varies considerably in practice. This aspect is illustrated by Table 1, which contains examples drawn from anchor designs in various countries. Another feature which although widely recognised receives little consideration, is that a solid, homogeneous rock mass is seldom encountered. Therefore, in the vast majority of cases, modification to the simple cone approach should be made by experienced rock mechanics engineers. Little data are available on the safety factors employed when analysing the weight of rock in the assumed pull-out zone, but it is known that values of 3.0 (Schmidt, 1956), 2.0 (Rawlings, 1968) and 1.6 (Littlejohn and Truman-Davies, 1974) have been employed in practice.

In current practice the factor of safety is reduced to unity on many occasions on the basis that certain rock parameters, e.g. shear strength, otherwise ignored in the design, will give rise to a sufficiently large factor of safety as a matter of course. This bonus of shear strength is, however, greatly reduced when anchors are installed in highly fissured "loose" rock masses, especially in those with much interstitial material or high pore water pressure. In this connection it is noteworthy that Klopp (1970) found, in typical Rhine Slate, that elevated hydrostatic and seepage pressures could reduce the shear strength of mylonitic zones to about 20% of the "ideal" laboratory dry value, and occasionally to as low as 4% of this figure.

Other engineers confirm that rock shear strength generally contributes a major component of the ultimate pull-out resistance and suggest the use of an allowable shear stress acting over the cone surface e.g. 0.034 N/mm^2 (Saliman & Schaefer, 1968) and 0.24 N/mm^2 (Hilf, 1973).

In general, there is a dearth of data on anchor failures induced in the rock mass. However, Saliman and Schaefer (1968) did obtain some valuable information, on this overall stability aspect, by testing to failure grouted bars in connection with the Trinity Clear Creek transmission line. Four tests were carried out on deformed reinforcement bars grouted into 70 mm diameter holes to a depth of 1.52 m in sediments, largely shale. In all cases, failure occurred when a block of grout and rock pulled-out; the propagation of cracking to the rock surface gave an indication of the cone of influence (Fig. 3).

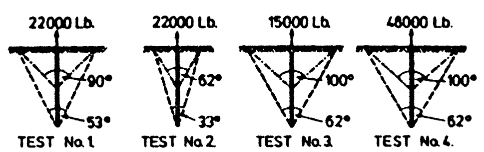

FIGURE 3. POSSIBLE FAILURE MODES BASED ON TEST RESULTS AT TRINITY CLEAR CREEK. (AFTER SALIMAN & SCHAEFER, 1968).

Geometry of Inverted Cone		Source
Included Angle	Position of Apex	
60°	Base of Anchor	Saliman & Schaefer [1968]
60°	" "	Hilf [1973]
90°	Base of Anchor	Banks [1955]
90°	" "	Parker [1958]
90°	" "	Hobst [1965]
90°	" "	Wolf et al [1965]
90°	" "	Brown [1970]
90°	" "	Longworth [1971]
90°	" "	Lang [1972]
90°	" "	White [1973]
90°	Base of Anchor (where load is transferred by end plate)	Stocker [1973]
90°	Middle of fixed anchor (where load is transferred by bond	Stocker [1973]
90°	Middle of anchor	Morris & Garrett [1956]
90°	" "	Rao [1964]
90°	" "	Eberhardt & Veltrop [1965]
90°	Top of Fixed anchor	Rawlings [1968]
90°	"	Rescher [1968]
90°	"	Golder Brawner [1973]
*60-90°	Middle of fixed anchor (where load is transferred by bond)	Littlejohn [1972]
*60-90°	Base of anchor where load is transferred by end plate)	
90°	Top of fixed anchor, or	Australian Standard CA 35 [1973]
60°	Base of anchor.	

*60° employed primarily in soft, heavily fissured or weathered rock mass

Table 1 Geometries of Rock Cone Related to Fixed Anchor Which Have Been Employed in Practice

Assuming a bulk density of 2 Mg/m^3 for the rock, back analyses of the failure loads indicate very conservative results - safety factors on the pull-out load between 7.4 and 23.5 - if the apex of the 90° cone is assumed at the mid point of the anchor length, but lower factors - 0.9 to 2.9 - for a cone with the apex at the base. In contrast, in the laminated dolomite in which Brown [1970] installed shallow test anchors, the shape of the pull-out zone could not be observed, although the extensive area over which the rock surface was uplifted around certain anchors suggested failure along a horizontal bedding plane (laminar failure). Rock failures of this mode Brown thought to be restricted to shallow anchors, but in current practice, fear of laminar failure, or excessive fixed anchor movement during service, has led to the adoption of staggered anchor lengths, even at great depths, for closely spaced anchors. In unfavourable conditions, for example, where a major rock discontinuity occurs normal to the anchor axis, the purpose of staggered lengths is to reduce the intensity of tensile stress across such planes at the level of the fixed anchor.

It is thus evident that whilst a major factor in the choice of anchor depth is the size of rock cone or wedge to be engaged, the possibility of laminar failure may also influence the designer's choice of lengths in closely spaced anchor groups. The South African Recommendations (1972) suggest that in the case of "concentrated" groups, where the fixed anchors are spaced at less than 0.5 x the fixed anchor length apart, the stagger between alternate anchors should be 0.5 x the fixed anchor length. This compares with a stagger of 0.25 x the fixed anchor length used at the Devonport Nuclear Complex by Littlejohn and Truman-Davies (1974), where 2 MN anchors in slate where spaced at 1 m centres. Another method to dissipate load within the rock mass is simply to install anchors at different inclinations, as in the design by Soletanche (1968) for the Zardesas Dam in Algeria.

Remarks

With regard to uplift capacity no experimental or practical evidence substantiates the methods currently used (Table 1) to calculate the ultimate resistance to pull-out of individual, or groups of anchors. However, it is reassuring to note that most designs are likely to be conservative in adopting a cone method in which no allowance for the shear strength of the rock mass has been made.

Nevertheless some standardisation on safety factors for temporary and permanent anchors is desirable, together with agreement on what allowances should be made for surcharge due to unconsolidated overburden, and the effect of upper layers of weathered rock.

Bond Between Cement Grout and Rock

The straight shaft anchor relies mainly on the development of bond, or shear stress, along the rock/grout interface, and it is usual to assume an equivalent uniform distribution of bond stress over the fixed anchor surface. Thus the anchor force, F, is related to the fixed anchor design by the equation:

$$F = \pi d \ell \tau \qquad \text{---(1)}$$

where ℓ = fixed anchor length
d = effective anchor diameter
τ = working bond stress

This approach is used in many countries e.g. Canada (Coates, 1970), Britain (Littlejohn, 1972), France (Fargeot, 1972), Italy (Mascardi, 1973), and U.S.A. (White, 1973).

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average working bond stress at the rock/grout interface should not exceed the minimum shear strength divided by the relevant safety factor (normally not less than 2). This approach applies primarily to soft rocks where the uniaxial compressive strength (U.C.S.) is less than 7/mm^2, and in which the holes have been drilled using a rotary percussive technique. In the absence of shear strength data, or field pull-out tests, Littlejohn (1972) states that the ultimate bond stress is often taken as 10% of the uniaxial compressive strength of massive rocks (100% core recovery) upto a maximum value of $\tau_{ultimate}$ of 4.2 N/mm^2, assuming that the crushing strength of the cement grout is equal to or greater than 42 N/mm^2. Applying an apparent safety factor of 3 or more - which is conservative bearing in mind the lack of relevant data - the working bond stress is therefore limited to 1.4 N/mm^2. In some rocks, and particularly granular, weathered varieties with a relatively low ϕ value, the assumption that $\tau_{ultimate}$ equals 10% rock U.C.S. may lead to an artificially low estimate of shear strength (Figs. 4 & 5).

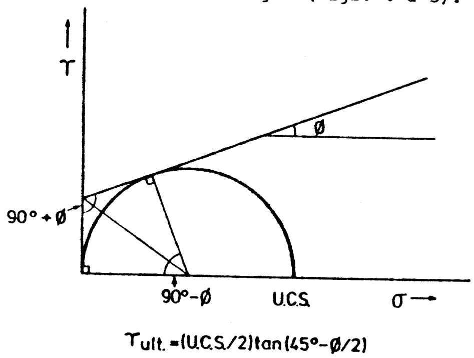

FIGURE 4. RELATIONSHIP BETWEEN SHEAR STRESS AND UNIAXIAL COMPRESSIVE STRENGTH.

In such cases the assumption that $\tau_{ultimate}$ = 20-35% U.C.S. may be justified. As a guide to specialists, bond values which have been used throughout the world for a wide range of igneous, metamorphic and sedimentary rocks, are presented in Table 2. Where included, the factor of safety relates to the ultimate and working bond values, calculated assuming uniform bond distribution. Usually the recommended safety factor ranges from 2 to 3, but is frequently lower in very competent rocks and higher in weaker, fissured, or weathered types.

The degree of weathering of the rock is a major factor which affects not only the magnitude of the ultimate bond but also the load/deflection characteristics.

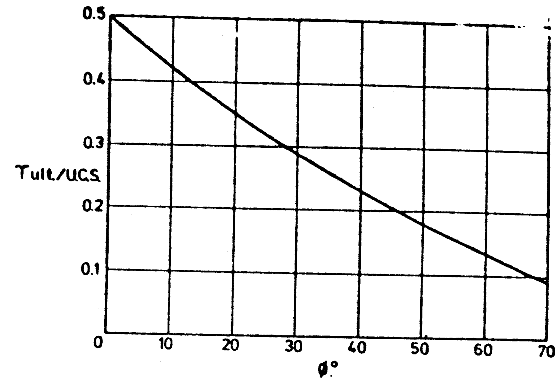

FIGURE 5. EFFECT OF ø ON Tult/U.C.S. RATIO

Figure 6 shows the results obtained from test anchors in rhyolite tuff, of both sound and weathered varieties. It is significant that the equivalent uniform bond stress - at maximum jack capacity - is scarcely 0.1 N/mm^2.

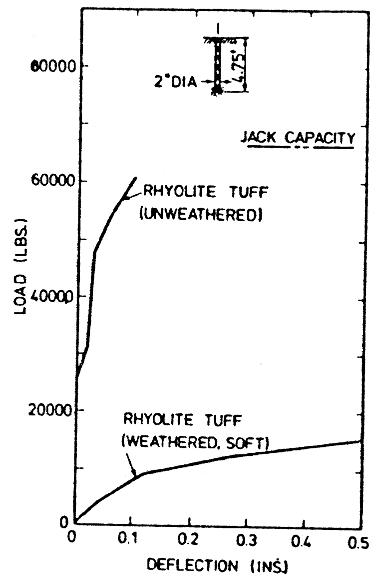

FIGURE 6. EFFECT OF WEATHERING AT CURRECANTI MIDWAY TRANSMISSION LINE (AFTER SALIMAN & SCHAEFFER, 1968).

For design in soft or weathered rocks there are signs that the standard penetration test is being further exploited. For example, Suzuki et al (1972) state that for weathered granite, the magnitude of the bond can be determined from the equation.

$$\tau_{ultimate} = 0.007\ N + 0.12\ (N/mm^2) \qquad --- (2)$$

where N = number of blows per 0.3 m

Similarly, Littlejohn (1970) shows for stiff/hard chalk that

$$\tau_{ultimate} = 0.01\ N\ (N/mm^2) \qquad --- (3)$$

Although it would appear from evidence presented in subsequent sections that the assumptions made in relation to uniform bond distribution are not wholly

Rock Type	Working Bond (N/mm^2)	Ultimate Bond (N/mm^2)	Factor of Safety	Source
IGNEOUS				
Medium hard basalt		5.73	3 - 4	India - Rao [1964]
Weathered granite		1.50 - 2.50		Japan - Suzuki et al [1972]
Basalt	1.21 - 1.38	3.86	2.8 - 3.2	Britain - Wycliffe Jones [1974]
Granite	1.38 - 1.55	4.83	3.1 - 3.5	" - " " "
Serpentine	0.45 - 0.59	1.55	2.6 - 3.5	" - " " "
Granite & basalt		1.72 - 3.10	1.5 - 2.5	U.S.A. - P.C.I. [1974]
METAMORPHIC				
Manhattan schist	0.70	2.80	4.0	U.S.A. - White [1973]
Slate & hard shale		0.83 - 1.38	1.5 - 2.5	U.S.A. - P.C.I. [1974]
CALCAREOUS SEDIMENTS				
Limestone	1.00	2.83	2.8	Switzerland - Losinger [1966]
Chalk - Grades I-III		0.22 - 1.07	1.5 - 3.0	Britain - Littlejohn [1970]
Tertiary limestone	0.83 - 0.97	2.76	2.9 - 3.3	Britain - Wycliffe-Jones [1974]
Chalk limestone	0.86 - 1.00	2.76	2.8 - 3.2	" - " " "
Soft limestone		1.03 - 1.52	1.5 - 2.5	U.S.A. - P.C.I. [1974]
Dolomitic limestone		1.38 - 2.07	1.5 - 2.5	" - P.C.I. "
ARENACEOUS SEDIMENTS				
Hard, coarse-grained sandstone	2.45		1.75	Canada - Coates [1970]
Weathered sandstone		0.69 - 0.85	3.0	New Zealand - Irwin [1971]
Well cemented mudstone		0.69	2.0 - 2.5	" " - " "
Bunter sandstone	0.40		3.0	Britain - Littlejohn [1973]
Bunter sandstone (U.C.S. > 2.0 N/mm^2)	0.60		3.0	" - " "
Hard find sandstone	0.69 - 0.83	2.24	2.7 - 3.3	Britain - Wycliffe-Jones [1974]
Sandstone		0.83 - 1.73	1.5 - 2.5	U.S.A. - P.C.I. [1974]
ARGILLACEOUS SEDIMENTS				
Keuper marl		0.17 - 0.25	3.0	Britain - Littlejohn [1970]
Weak shale		0.35		Canada - Golder Brawner [1973]
Soft sandstone & shale	0.10 - 0.14	0.37	2.7 - 3.7	Britain - Wycliffe Jones [1974]
Soft shale		0.21 - 0.83	1.5 - 2.5	U.S.A. - P.C.I. [1974]
GENERAL				
Competent rock (where U.C.S. > 20 N/mm^2)	U.C.S. ÷ 30 (up to a maximum value of 1.4 N/mm^2)	U.C.S. ÷ 10 (up to a maximum value of 4.2 N/mm^2)	3.0	Britain - Littlejohn [1972]
Weak rock	0.35 - 0.70			Australia - Koch [1972]
Medium rock	0.70 - 1.05			
Strong rock	1.05 - 1.40			
Wide wariety of igneous and metamorphic rocks	1.05		2.0	Australia - Standard CA35 [1973]

Table 2 Rock/Grout Bond Values Which Have Been Recommended in Practice

accurate, it is noteworthy that few failures are encountered at the rock/grout interface, and new designs are often based on the successful completion of former projects; that is, former "working" bond values are re-employed or slightly modified depending on the judgement of the designer.

Fixed Anchor Dimensions

Most fixed anchor lengths which have been employed in practice are in the range 3 to 10 m. A minimum length of 3 m is generally recommended, although 5 m has been suggested by the Bureau Securitas (1972) and White (1973), whilst the South African Code stipulates 4 and 6 m, for very hard and soft rock, respectively. Under certain conditions, it is recognised that much shorter lengths would suffice, even after the application of a generous factor of safety. However, a sudden drop in rock quality along the anchorage zone, and/or constructional inefficiencies, would seriously impair the efficiency of short fixed anchors.

With regard to the choice of anchor diameter several considerations must be taken into account:

1. Type and size of tendon.
2. The relation of diameter to the perimeter area of fixed anchor and hence to the anchor capacity.
3. Ratio of steel area to cross-sectional area of borehole for efficient bond distribution and corrosion protection.
4. Drilling method and rig to be used.
5. Nature of rock in the anchorage zone and presence of unconsolidated overburden, if any.

The authors find from a survey of several hundred commercial anchor reports that no direct relationship may be observed bearing in mind the range of anchor types, but that most anchors conform to the following trend (see Table 3)

Capacity (kN)	Diameter (mm)
200 - 1200	50 - 100
1000 - 3000	90 - 150
3000 - 4500	150 - 200
4500 - 14000	200 - 400

Table 3 Approximate Relationship Between Fixed Anchor Diameter and Working Capacity

Where corrosion protection is important, the South African Code (1972) stipulates that the fixed anchor diameter should be equal to the outside diameter of the tendon plus at least 12 mm. This approach has also been discussed by F.I.P. (1972) who recommend a grout cover to the tendon of 5 mm, and 5-10 mm for temporary and permanent rock anchors, respectively. With regard to the amount of steel which may be placed in an anchor borehole, there is a scarcity of information, although Littlejohn and Truman-Davies [1974] suggest that the steel should not exceed 15% of the borehole cross-sectional area.

Theoretical Evidence

Studies of the stress distribution around a cylindrical anchorage in a triaxial stress field have been carried out by Coates and Yu (1970) using a finite element method. The authors show that the shear stress (i.e. bond) distribution, is dependent on the ratio of the elastic modulii of the anchor material (E_a) and the rock (E_r). Fig. 7 shows the variation of the shear stress along the interface of an anchor of length equal to 6 times its radius for E_a/E_r ratios of 0.1, 1 and 10. The smaller this ratio the larger is the stress calculated at the proximal (or loaded) end of the anchor; higher values of the ratio are associated with more even stress distributions. It is also apparent for E_a/E_r greater than 10, i.e. for very soft rocks, it is reasonable to assume that the bond is evenly distributed along the anchor, and that the anchor design may be based accurately and directly on the shear strength of the weaker medium.

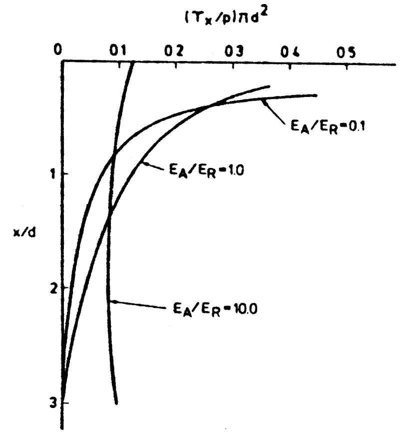

FIGURE 7. VARIATION OF SHEAR STRESS WITH DEPTH ALONG THE ROCK/GROUT INTERFACE OF AN ANCHOR. (AFTER COATES & YU, 1970).

It is likely that the majority of rock anchors to date have been installed in rocks giving values for the ratio E_a/E_r of between 0.1 and 1, and for which according to Fig. 7, the bond distribution is markedly non-uniform. Indeed, for anchors in rocks of compressive strength in excess of 7 N/mm^2, say, stress concentrations at the proximal end are most likely, having a magnitude possibly 5 - 10 times the average stress level.

Although less satisfactory from a theoretical point of view, anchors in strong rocks at present represent less of a problem in practice, since a larger safety factor can be accommodated without significantly increasing the cost. However, for the accurate design of high capacity anchors, insufficient attention has been paid to the high stresses at the proximal end, and in particular to the effect of debonding on stress distribution.

Experimental Evidence

In Italy much valuable experimental research has been conducted, principally by Berardi, into the distribution of stresses along the fixed anchor and into the rock. In 1967 he concluded from tests on the distribution of fixed anchor stresses, that the active portion of the anchor is independent of the total fixed anchor length but dependent on its diameter and the mechanical properties of the surrounding rock, especially its modulus of elasticity. Figs. 8 a and b are typical diagrams which illustrate the uneven bond distribution, as calculated from strain gauge readings. Those anchors were installed in 120 mm diameter boreholes in marly limestone ($E = 3 \times 10^4 kN/m^2$; U.C.S. = 100 N/mm^2 approximately). Other results show that the bond distribution is more uniform for high values of E_{grout}/E_{rock}, and non-uniform for low values of this ratio i.e. for rock of high elastic modulus. These results thus confirm the conclusions drawn by Coates and Yu.

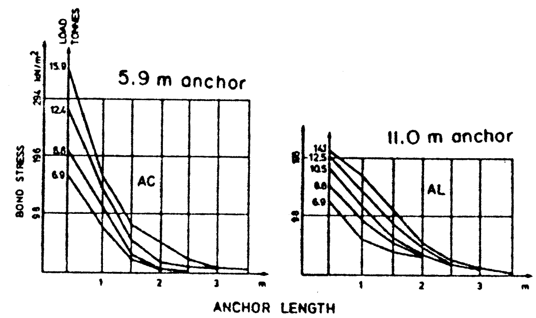

FIGURE 8. DISTRIBUTION OF BOND ALONG FIXED ANCHOR LENGTH (AFTER BERARDI, 1967).

Muller (1966) produced interesting results in Switzerland on the distribution of shear stress along the 8 m fixed anchor of a 2200 kN anchor (Fig 9). At a load of 550 kN the force was transmitted uniformly over the proximal 5.55 metres. At 1850 kN however, the load was recorded over the lower 4.1 m of the tendon with apparent debonding of the tendon of the upper 3.9 m. At 2800 kN a comparison of the theoretical and measured anchor elongations suggested that total debonding of the tendon had occurred, and that all the load was resisted by the foot of the fixed anchor.

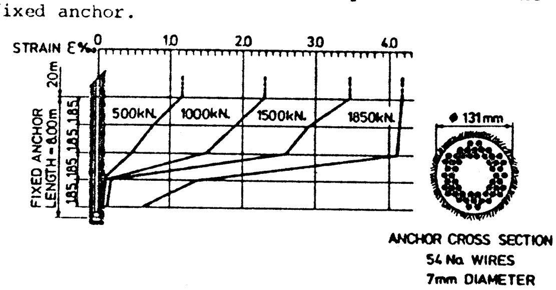

FIGURE 9. STRAIN DISTRIBUTION ALONG TENDON IN FIXED ANCHOR ZONE OF A 2200kN CAPACITY ANCHOR. (AFTER MULLER, 1966).

Remarks

Mathematical, laboratory and field evidence indicate that the distribution of the bond, mobilised at the rock/grout interface, is unlikely to be uniform unless the rock is 'soft. In the case of high capacity anchors, evidence exists that partial debonding in the fixed anchor occurs, and the debonding progresses towards the end of the anchor as the load is increased. Information is scarce however concerning the conditions where debonding is serious.

In general, there is a scarcity of empirical design rules for the various categories of rock and too often bond values are quoted without provision of strength data, or a proper classification of the rock or cement grout.

Bond Between Cement Grout and Steel Tendon

Little attention has been paid to this aspect of rock anchor design, principally because engineers usually consider that the fixed anchor length chosen with respect to the rock/grout bond, ensures more than adequate tendon embedment length.

In fixed anchor design it is common in practice to find embedment lengths for bars, wires and strands quoted as equivalent to a certain number of diameters as this method ensures a maximum value of apparent average bond stress for each type of tendon. The transmission length is the length required to transmit the initial prestressing force in a tendon to the surrounding grout. It should be borne in mind, however, that the transmission length varies with grout strength as well as size and type of tendon and it is still advisable on occasions to measure experimentally the transmission length for the known site conditions.

In Britain a minimum anchor length of 100 diameters for plain wire (grout U.C.S. > 35 N/mm^2) is specified, whilst for small diameter strand (9.3 - 18.0 mm diameter) the transmission length varies from 19 - 31 diameters, based on a grout strength range of 34 - 48 N/mm^2. For compact strand e.g. Dyform, it is accepted that transmission lengths are generally 25% greater than those for normal 7 wire strand. Sudden release of load also increases the transmission length and an additional 25% is recommended in Rumania.

The Australian Code (1973) stipulates a maximum value of 1.05 N/mm^2 for the bond stress for a clean wire tendon, and 2.10 N/mm^2 for a clean strand tendon. With regard to permissible bond stresses for plain and deformed bars Table 4 illustrates the values stipulated by the British Code CP 110 for different grades of concrete. These values are applied to neat cement grouts on occasions.

Type of bar.	Characteristic Strength of Concrete (f_{cu} - N/mm^2)			
	20	25	30	40+
Plain	1.2	1.4	1.5	1.9
Deformed	1.7	1.9	2.2	2.6

Table 4 Ultimate Anchorage Bond stresses

It is important to note that no information is provided on the minimum spacing where reduction factors should be employed to take account of group effects, and no guidance is provided on the use of spacers and centralisers which could lead to decoupling.

With reference to minimum embedment lengths used in practice, Morris and Garrett (1956) have calculated from stressing tests on 5 mm diameter wires that the minimum necessary embedment is just over 1 m. Golder Brawner (1973) found that although the grout/strand bond is higher than expected from tests on single wires due to "spiral interlock", the value drops rapidly if the embedment length is less than 0.6 metres.

Distribution of Bond

The assumption of uniform bond distribution at the tendon interface is seldom true in practice. Invariably, as the load increases, progressive slip at the proximal end occurs, and the location of the maximum intensity of bond stresses moves towards the distal end. Hawkes and Evans (1951) were able to conclude from pull-out tests that the distribution of bond obeys an exponential law of the form:

$$\tau_x = \tau_o e^{\frac{-Ax}{d}} \qquad ---(4)$$

where τ_x = bond stress at a distance x from the proximal end.
τ_o = bond stress at the proximal end of the bar.
d = diameter of the bar.
A = a constant relating axial stress in the bar to bond stress in the anchor material.

The greater the value of A, the larger the stress concentration at the free or proximal end of the anchor. The smaller the value of A, the more evenly the stresses are distributed along the length of the anchor. Insufficient information exists at present on the behaviour of cement grout anchors in rock to provide meaningful values for A but it is reassuring to find that the theoretical trends are very similar to those in Fig. 7 (Coates & Yu), with E_a/E_r proportional to 1/A. This indicates that at least the basic approach of Hawkes and Evans is applicable to rock anchors.

Remarks

Some designers consider the question of grout/tendon bond in anchor systems to present no problems as the design at the rock/grout interface is more critical. A factor of safety of at least 2 against tendon pull-out is stipulated by other designers.

Little work has been done on multi-unit tendons with respect to bond distribution. The use of spacers and centralisers, and the problem of decoupling also warrant investigation. In general, recommendations pertaining to grout/tendon bond values used in current rock anchor practice, commonly take no account of the length and type of tendon, or the tendon geometry. For these reasons it is still advisable to measure experimentally the embedment length for known field conditions.

Tendon

Accurate information on the strength and elastic properties of tendon components is readily available, but the choice of the type of tendon and the safety factors to be employed against rupture, still demands assessment and judgement by the designer, especially in countries not covered by a Code relating to anchors.

Tendons may be formed of bars, wire or strand. The latter two have distinct advantages with respect to tensile strength, ease of storage, transportation and fabrication. Bars, however, are more readily protected against corrosion and in the case of shallow low capacity anchors, are often easier and cheaper to install. In strong competent rocks where the amount of fixed anchor creep is negligible, an allowance should be made for tendon relaxation under sustained loading. Under these circumstances, a low relaxation tendon should be used; the loss at 1000 hours being less than 2.5%.

Fig. 10 compares relaxation losses for bars, wires and strands under similar conditions. It should, of course, be remembered that the amount of loss depends on the initial stress in the steel, its production history, and the ambient temperature.

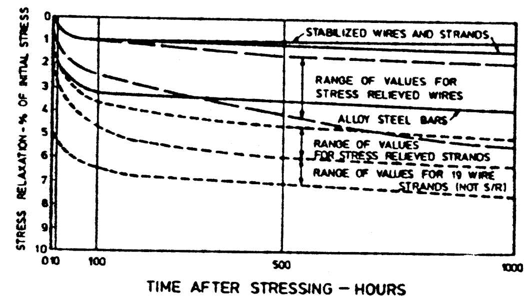

FIGURE 10. RELAXATION OF BRITISH TENDONS AT 20°C. FROM INITIAL STRESSES OF 0.7 U.T.S.

Allowable Stresses and Safety Factors

In Britain, permissible stresses are quoted in terms of the specified characteristics strength which is the guaranteed limit below which not more than 5% test results fall, and none of these are less than 95% characteristic strength. For permanent and temporary anchorages the authors' recommendations are summarised in Table 4.

Item	Anchor Category	
	Temporary (life < 2 years)	Permanent
Design force	62.5% fpu	50% fpu
Test force	78% fpu	75% fpu
Ultimate safety factor	1.6	2.0
Measured safety factor	1.25	1.5

Table 4: Recommended Safety Factors for Tendon Design

Testing to 1.5 times the working stress seems at present to be the exception rather than the rule, and commonly contract anchors are over stressed by an amount thought equivalent to long term load losses - usually 10%. It is noteworthy however that the current trend in European countries is towards higher safety factors and more rigorous tests.

Remarks

Whilst tendons are produced to a high standard with reliable minimum breaking loads, few load/extension tests have been carried out on long tendons (10 - 30 metres) which are comparable in size to the free anchor lengths used in practice. Since interpretation of anchor load/displacement characteristics can be problemmatical in practice, particularly for strand anchors, it would be of value to know if long strand tests give E values which are significantly different from those obtained using short gauge lengths of 0.6 m.

QUALITY CONTROL

Drilling

The technology of drilling is both highly complex and extensively documented e.g. McGregor (1967). The method of drilling is chosen primarily for optimum production and in most rocks percussive equipment is common for depths down to 60 m and diameters up to 100 mm. In weathered rocks care should be taken to adopt a flushing medium which will not adversely affect rock strength properties.

Little guidance is available on maximum permitted drillhole deviations, but tolerances of 0° 28' (Parker, 1958), 1° 10' (Eberhard & Veltrop, 1965) and 0° 43' (Littlejohn and Truman-Davies, 1974) appear fairly rigorous compared with the 2° 30' permitted by the South African Code (1972). Borehole inclinometers may be employed to ensure that deviations are within acceptable limits.

In general, all changes in ground strata should be recorded by the driller, in addition to notes on drilling rates and loss of flushing medium. In this connection the recent addition of torque and thrust gauges on drilling rigs is a welcome innovation.

Waterproofing

On completion of drilling, the holes must be tested for "watertightness", by measuring the water outflow, or leakage rate into the surrounding rock. A falling head or packer test determines whether pregrouting of the hole is necessary. Bearing in mind that loss of grout from around the tendon in the fixed anchor zone is of prime importance in relation to efficient distribution of load and corrosion protection, the minimum width of fissure which will permit flow of cement at low pressure must be assessed. Littlejohn (1975) reviewed current practice world wide and concluded that:

(1) Waterproofing is required if leakage exceeds 3 litres/min/atmosphere, measured over a period of 10 minutes.

(2) For a measured outflow (or gain under artesian conditions), a "backpressure" is required during the grouting stage. If the flow cannot be counteracted in this way, waterproofing is necessary regardless of the magnitude of the water gain.

Homing

Immediately prior to homing, the tendon should be carefully inspected. As a principle it must be possible to examine at least one stage of protection against corrosion, for permanent anchors. For temporary anchors, normal rusting is acceptable since it improves the grout/tendon bond, but strands with flaky, loose rust must be thoroughly wiped.

In certain situations the efficiency of the centraliser spacer units may be judged by carefully withdrawing the tendon to observe damage, distortion, or presence of smear. In this connection tendons of total weight in excess of 200 Kg should be lowered in a controlled manner with the aid of a mechanically operated drum.

Grouting

The delay between drilling and grouting should always be kept to a minimum,and, as a policy, one should always drill and grout the fixed anchor on the same day. The use of grout cubes for strength control, and flow meters or viscometers to monitor pumpability in relation to tremie grouting should be standard practice. In situations where pressure is required, this is often limited to 50-70% overburden pressure, although on occasions 150% overburden pressure has been employed. Use of higher pressures leading possibly to hydrofracture and surface heave should be avoided. Neat cement grouts are usually designed to give a 28-day crushing strength of 42 N/mm^2 and anchor stressing is not permitted in many countries until a strength of 28 N/mm^2 has been attained, normally 7-10 days after grouting.

Stressing

A major advantage of prestressed over "passive" anchor systems is that prestressing to the design working load automatically checks the security and efficiency of the anchor. Thus, if errors have been made in either the design or construction stages, these will be immediately pinpointed and potentially dangerous and expensive consequences avoided.

As a first priority, the stressing procedure must yield a measured safety factor, which should be obtained by overloading every contract anchor for a short period (see Table 4).

In addition it is essential that the load-extension curve be plotted for each anchor tested. In Europe, about 10% of the designed working load is usually applied to "seat" the anchor; extensions are thereafter measured at a minimum of four equal load increments up to the working load.

The load-extension diagram must bear a reasonable similarity to that calculated by theory. In Germany it is stipulated that the plotted results should lie between the lines corresponding to:

(1) The extension of a tendon of length equivalent to 80% free length, and

(2) The extension of a tendon of length equivalent to the free length plus 50% fixed (socket) length.

The recorded curve will probably approximate to curve (1) but tend to curve (2), as load increases and debonding progresses.

Wherever possible, efforts should be made, on either the preliminary test anchors, or on early production anchors, to obtain an indication of the fixed anchor movement. This is obtained by subtracting from the total permanent displacement (measured by a simple load-unload cycle) the top anchorage plate movement (monitored by independent survey).

For anchors in competent rock, both these displacements are small with respect to total measured tendon extension. However, if they are found to contribute in excess of 5% of the total extension, at lock off, then allowance must be made in the comparison of the theoretical and actual extensions.

Variations of up to 5% of the stated value may be expected in the steel modulus of elasticity, thus providing another cause of theoretical and measured extension discrepancy. The twisting and rubbing known to occur in long flexible tendons may also contribute towards apparently anomolous extensions.

Sources of Load Loss in Prestressed Anchors

It is well known that numerous sources of load loss, both immediate and long term, afflict prestressed rock anchors, and commonly an allowance is made for expected losses in the form of an initial overstress.

Friction always acts in the jack and in the anchor. Unless a load cell is incorporated in the anchor, a correction factor must be applied to the jack load. This factor will be minimised if jacks are frequently calibrated (every 3000 strands) and regularly serviced on site. Frictional losses occurring within the anchor - especially in the free length of long multi-strand anchors and particularly just under the head- can only be compensated for by increasing the applied load by a certain amount. The exact allowance can be obtained from a load cell, or by a cyclic loading analysis. Frequently up to 10% of the applied load is lost in friction, and occasionally as much as 30% (Hennequin & Cambefort 1966).

Lock off losses occur in strand anchors due to wedge "pull-in" at the head, and are proportionally higher for shorter tendons. To allow for this an overstress by a nominated amount - usually 10% is frequently recommended. However, a more accurate method is to observe on test anchors, the actual amount of wedge "pull-in", and thereafter to stipulate an overload of magnitude sufficient to produce an additional tendon extension of this size.

Long term losses are due to a combination of steel relaxation and anchor creep. The relaxation characteristics of prestressing steel are well known, and readily available from manufacturers. Depending on the initial prestress level, restressing after 1000 hours may reduce ultimate prestress loss due to relaxation by up to four times.

Less is known about creep in rock anchor systems largely because information regarding the magnitude and distribution of stresses around the fixed anchor is very scarce. In heavily fissured weathered rock, or fractured rock with clay infill, creep losses may be significant and an estimation of the amount to be expected can be gauged from test anchors installed well in advance of the contract. Unfortunately there is no simple alternative short term test to predict the long term behaviour of production anchors. A load loss of up to 5%, or a creep displacement of 1 mm, measured after 24 hours, has been specified on occasions in soil, but no reliance should be placed on these arbitrary figures. Only when creep losses are monitored over long periods for a variety of anchor loads and systems, and for a wide range of rock types, will an accurate predictive capacity be available. Until then, it is recommended that periodic checks of anchor stress should be carried out on production anchors, as follows:

1. The load in all anchors should be checked 24 hours after stressing, to provide an early warning of load loss, if any. This check applies to temporary and permanent anchors.

2. On a large contract, the first 10 anchors should be checked weekly for 1 month, then monthly for 3 months.

3. Subject to satisfactory results from the 4 months testing program, 5% of all anchors should be checked at 6 months, and at 12 months.

In this way a correlation may be attempted between loss at 24 hours or 1 week, and long term behaviour, which may eventually result in a more reliable allowance for short term loss being specified.

In practice, if the anchor fails or creeps significantly during stressing, then the anchor should be unloaded to the level at which no creep occurs. The revised working load will then be that level divided by the design safety factor for the tendon.

Remarks

Good site supervision and the provision of adequate quality controls are the exception rather than the rule at present. It should be appreciated that precautionary measures save more time and money in the long run compared with remedial measures. In addition, records covering the drilling, grouting and stressing stages, can be invaluable to the engineer asked to provide an explanation for possible anchor malfunctions.

There is a growing need to standardise the stressing and monitoring procedures which guarantee the safety and satisfactory performance of anchors during service.

Conclusions

Effort should now be expended in the form of field testing in a wide range of rock materials and masses which have been carefully classified in order to study the shape and position of the rock zones mobilised at failure. Such programmes should accommodate single anchors and groups tested over a range of inclinations. Only in this way can anchor design in relation to overall stability be optimised both technically and economically.

Since the validity of the uniform distribution of bond which is commonly assumed by designers is clearly in question, it is recommended that instrumented anchors should be pulled to failure in wide range of rock masses whose engineering and geological properties can be fully classified, in order to ascertain which parameters significantly affect anchor performance. In this way it should be possible in due course to provide more reliable and economic design criteria.

Whilst there is an appreciable amount of information available concerning the mechanism of bond transfer in the field of reinforced and prestressed concrete, it is considered that much more study is required in the field of rock anchors. The mode of failure of a tendon in a rock anchor situation may be dissimilar to that of the tendon pull-out test used in concrete technology and from which most design data are obtained. In the former case, the grout is usually in tension whereas during a standard bond test, part, at least, of the surrounding concrete is in compression.

A high standard of workmanship coupled with careful inspection and record keeping are the keys to success on site. In this connection closer liaison is required between drilling and grouting personnel, and the supervising engineer.

An agreed approach to the testing and analysis of anchor behaviour should be established which will quarantee satisfactory performance both in the short and long term.

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SESSION 2 SLOPES AND FOUNDATIONS

REVIEW AND COMMENTARY

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INTRODUCTION

I have been asked to comment on some of the practical aspects of rock slope foundation engineering with particular reference to papers presented in this session. I would like to preface my remarks by commenting upon the relationship between the type of problem under consideration and the level of geotechnical effort which can be devoted to the solution of the problem.

COMPARISON BETWEEN HIGH AND LOW INTENSITY PROGRAMS

Consider two projects, each involving a total capital cost of ten million dollars. Project A is the foundation for a concrete arch dam while project B is a 10 kilometre long mountain highway. In each case the geotechnical budget is assumed to be 1% of the total capital cost, in other words $100,000, which may be regarded as a reasonable average percentage in a major civil engineering project.

In the case of project A, the volume of rock involved in the dam foundation would be relatively small, say 100,000 cubic metres, and hence the geotechnical engineer has $1.00 per cubic metre of rock to spend on his investigations. On the other hand, the volume of rock which has to be considered in project B, the 10 km highway, is very large, say ten million cubic metres. Within the constraints of his budget of $100,000, the geotechnical engineer only has one cent per cubic metre available for his investigation.

Clearly, the approach which must be adopted in planning these two geotechnical studies must be quite different. In the case of project A, the serious consequences associated with the failure of a concrete arch dam would justify the use of the most sophisticated site investigation and analytical techniques. It is probable that such studies could be accommodated within the overall budget since the limited amount of rock involved in the study means that the geological data collection phase can be kept within reasonable limits.

The cost of comprehensive geological data collection on a 10 km highway route would far exceed the allocated budget. Consequently, the approach which would probably be adopted in this case would be to carry out a low cost, airphoto study supported by a limited amount of ground observation, designed to identify potential problem areas. The major proportion of the budget would then be devoted to the study of methods of avoiding these problems. It is more than likely that some problem areas would be missed in the initial superficial study and it would be important to provide a stand-by budget to deal with these problems during the construction and maintenance phases of the highway project.

Comparison between the requirements, in these two projects, for diamond drilling, structural geology logging, material properties testing, analytical design methods and practical remedial measures would reveal significant differences in all areas. In my experience, such differences are not always recognised and this results in inadequately planned geotechnical studies which are unlikely to meet the client's requirements. Note that inadequate planning does not mean that too little work is done, in fact, in many cases the reverse is true. When an engineer or geologist has not clearly thought through his proposed programme, and checked the relevance of each step in the investigation against the solution which is required, wasteful and irrelevant studies can be carried out and can lead to inadequate designs, budget over-runs and a general deterioration in relationships between the various parties involved in the project.

REVIEW OF PAPERS PRESENTED IN SESSION 2

The four papers presented in this session represent an interesting and varied set of contributions to the field of practical rock slope design.

In the light of comments made earlier about the level of investigation justified on a project, the paper by Londe and Tardieu is an example of a high intensity program. The design of the foundation for a dam has to be safe and the use of the sophisticated finite element model described in this paper is more than justified. The concepts included in this model are extremely interesting and the rock mechanics community can look forward to seeing further developments in this model by a team which has already made significant contributions to practical rock mechanics.

Bukovansky and Piercy's paper also makes use of the finite element technique but, perhaps, with slightly less justification than its use by Londe and Tardieu. This is not to say that the results presented are not interesting and that they have not contributed to a practical engineering solution - indeed, in both cases the paper makes a positive contribution. It could, however, be speculated that the same practical conclusion may have been reached without the aid of the finite element analysis in this case and, had the project budget been severely limited, this is the one component in the study which could have been dispensed with. In spite of these comments, it is good to see confirmatory studies of this sort carried out when they can be justified within a project budget.

The paper by Littlejohn and Bruce is a welcome summary of practical rock anchor data, particularly since it comes from Great Britain which is not usually thought of for work in rock anchors. A point which is brought home by this paper is that a significant amount of research into rock anchors is still justified. For example, in discussing the usual method of assessing the capacity of an anchor by assuming a cone fracture with the anchor at the cone apex, the authors comment"...in the vast majority of cases, modifications to this simple cone approach should be made by experienced rock mechanics engineers." The reviewer's comment on this statement is that, in the vast majority of cases, even experienced rock mechanics engineers would not have the faintest idea of how to make such modifications. The question of how stress is transferred from the rock through a grout bond to the steel bar or cable is also imperfectly understood at this time. Fortunately, in most cases, the ability of the rock mass to accommodate the ignorance of the rock mechanics engineer comes to the rescue and rock reinforcement systems can be remarkably effective, in spite of the lack of precision in their design.

Comments on the paper by Lawrence Von Thun have been left to last because this is the most philosophical paper of the four presented in this session. Many interesting and important points are discussed but the author may have been a little optimistic in choosing the title of his paper. This is because many of the techniques which he discusses have yet to be fully worked out as practical every-day rock slope design tools. In particular, methods of data collection to provide reliable input data for these analyses are not in common use.

The importance of a curvilinear relationship between normal stress and shear strength of discontinuities in rock has been recognised for some time* but the difficulties involved in incorporating this failure criterion into stability calculations for the wide variety of failure modes encountered in the field has inhibited its wide acceptance by practical rock mechanics engineers. This difficulty is further compounded by the problem of obtaining a reliable shear strength envelope which can be applied with confidence to the in-situ rock mass.

These comments should be taken as cautionary rather than negative. In fact the reviewer's opinion is that a curvilinear relationship between shear strength and normal stress is the only correct relationship for practically all rock masses and for most rock discontinuities. However, until we learn how to utilise these relationships, it is still permissible to obtain meaningful and practical rock slope designs on the basis of linear relationships within specified normal stress ranges.

Papers such as that by Von Thun should always be welcomed at a conference since they challenge many of the concepts and techniques with which we have become familiar and therefore, perhaps, complacent. The gradual evolution of new concepts and the development of new and better design methods depends upon challenges of this sort.

CONCLUSIONS

The aim of the Sixteenth Symposium on Rock Mechanics was to review practical design methods in rock mechanics. It would probably be impossible to completely satisfy this aim, even at a major international conference, because authors find it easier to write about their current interests rather than to attempt the much more difficult task of summarising and comparing established design methods. In any case, this latter task is more appropriately dealt with in a text book than in a short symposium paper.

Given the limitations inherent in a three day symposium, the papers in this session, and in the other sessions, represent a reasonable cross-section of current thinking on rock mechanics design.

*JAEGER, J. C. Friction of rocks and the stability of rock slopes. Geotechnique, Volume 21, No. 2, 1971, pages 97-134.

SESSION 2 - AUTHORS' REPLIES

Bukovansky

Dr. Hoek expressed some doubt that the finite element analysis was really necessary for the design of rock cuts.

From the practical point of view, we admit that the cuts could be designed without the finite element analysis. There is one reason for this: most of the rock cuts effectively decrease the slope angle of the existing cliffs and increase the overall stability.

We felt, however, that the finite element analysis should be carried out for three reasons:

1. To evaluate the state of the stresses in natural cliffs prior to the excavation. There were few data available on the stability of the cliffs. Many cliffs in the canyon carry traces of instability such as large, open vertical fractures behind them. Some of the cliffs are tilted and some of them have failed. Finite element analysis provides good data on the stress distribution and potential unstable zones both before and after the excavation.

2. Finite element analysis provides quantitative data on tensile stresses behind individual benches and it can be used for the evaluation of total bolting forces, if necessary.

3. The costs for the analysis were very low compared to the costs of the project, and of the geotechnical investigation.

Littlejohn

The paper presented is written from a civil engineers point of view, with the consequence that wide variations in design methods and quality controls associated with rock anchors have been deliberately highlighted; for example, calculations on uplift capacity based on crude cone and wedge mechanisms, and the related importance of the structure of the rock as discussed by Prof. Hoek. The possibility of laminar failure, even in horizontally bedded rock, is a matter of concern in civil engineering, where movements are equally as important as load safety factors. We feel that even in rock engineering of slopes it is very important to know precisely where to put the anchor, and would appreciate comments on where the anchor should go relative to the failure plane. It may be adequate in purely rock-stitching operations to locate the socket a nominal distance beyond the fracture being stitched, but in basic slope stability the distance must be sufficient that the wedges visualized in the factor of safety calculation are indeed mobilized.

In the paper, the validity of the assumptions that load at the rock/grout/tendon interfaces is uniform is seriously questioned, and the lack of data on decoupling noted. These are important in civil engineering because of effects on the load/displacement relationship and in corrosion protection. The influence of multi-unit tendons and spacers on the efficiency of load transfer appears to be practically unknown.

On the basis of 40-50 years reasonably successful experience in rock anchoring these factors may seem unimportant, but they are of relevance in civil engineering right now because codes of practices have recently been written in France, Germany, Australia and South Africa, and are under current attention in Switzerland, Austria, Sweden, Britain, Czechoslovakia and the United States. The civil engineering community, lacking wide experience in rock bolting, would benefit immensely from discussion on this subject, especially if there is the future possibility of civil engineering codes having jurisdiction over the activities of rock mechanics practioners now operating outside them.

On the question of quality control in civil engineering the stressing operation pretests the anchor, thus insuring its safety, but, after 40 years, there is still no standard procedure agreed upon today. In particular there is a dearth of data on long term behaviour, and, consequently, arbitrary acceptance figures based on short term behaivour, for example a 5% loss of prestress in 24 hours, or a 1 mm creep displacement measured over the same period, are being laid down in an effort to guarantee satisfactory performance in the long term. Long term data confirming that these short term recommendations are valid is not yet available.

The mode of stressing is also a subject for discussion. For example, in prestressing a dam into its rock foundation, to lock off every anchor to 1000 tons may not truly reflect the uplift capacity available in the rock. Similarly for example, a slab might be prestressed to a soil anchor with 100 tons of prestress, and yet the whole slab/soil/anchor system lifted out of the ground by a crane with 2-3 tons. Obviously the load testing and overall stability of the prestressing technique is an area of concern.

This morning Prof. John raised the point that, in rock engineering in particular regard to the reinforcement of rock slopes, the cost of tendons could be excessive. It seems important to recognise the existence of two distinct markets; mining engineering and civil engineering. In civil engineering the use of rock anchors is much more deliberately planned during the initial design of the overall structural system. Applications occur in retaining walls for deep excavations, dams, grading docks, and piling. The most spectacular recent example has been the large tension roofs at the Munich Olympic Complex. Also, in civil engineering, displacements that could occur in an overload situation are just as important as load safety factors.

Londe

In our paper, Tardieu and I present basically two methods for designing rock foundations. The first is the limit equilibrium of a solid, rigid rock volume. A 3-dimensional approach is used, as is required in most rock foundation problems. With limit equilibrium analysis we investigate failure only; i.e. the basic safety condition of the structure.

For many years we did not use finite element analysis because it was restricted to 2-dimensions. Now that it is available for use in 3-dimensions it is a very useful tool, primarily to analyse the behaviour of structures in normal operation. Examples given in our paper also show analyses including the effect of seepage water forces in one case, and the effect of cracks in an abutment in another case, representing examples of the use of finite element techniques to ascertain stability of the structure. A more recent analysis, just completed, has dealt with the foundation of a dam with three large geological discontinuities under or on the river banks.

One major problem with the 3-dimensional analysis is visualizing the results. It is necessary to plot several cross sections. Finally, something that isn't obtained from limit analysis, finite element analysis also provides stresses in the dam itself. Such initial information as possible excessive stress intensity in the toe of the dam adjacent to one bank is obtained in the example shown.

Von Thun

The aim of my paper was to highlight a number of practical problems encountered in slope stability analyses over recent years.

To continue from where Dr. Londe ended, one important area of interest is control of deflections associated with arch dams; with various possible stress/strain relationships. For example, it is important that the deflection on one side of the dam is not greater than on the other side, otherwise high stresses arise as Dr. Londe pointed out.

The use of curvilinear stress/strain relationships, commented on by Dr. Hoek, does not seem difficult if the results of limit equilibrium analysis are referred back to the graph of the stress/strain relationship.

Regarding Dr. Barton's question of Dr. Cundall, in Session 1, the location of the plane that does fail will depend upon the shear strength relationships because each plane will experience different normal stresses. Similarly this relationship will influence the position and mode of application of rock anchors.

SESSION 2 - GENERAL DISCUSSION

Question by John (for Bukovansky and Piercy)

i) In comparing two finite element methods, one using a simple straight-forward approach and the other a more sophisticated Goodman-type model, is it worthwhile to use the more sophisticated model?

ii) Why not use a routine zero tension program?

Reply by Bukovansky

Prof. John asked whether the finite element analysis, which included the joint elements, is necessary, in addition to the analysis without the joint elements.

As can be seen from both analyses, the model with joints seems to provide much more realistic results. The authors believe that only this model should be used for the final engineering design.

The described finite element models were used for additional analyses of cuts in deep soils. No-tension analyses, mentioned by Prof. John, could certainly be applied for this problem.

Discussion by Robertson

In an essentially similar problem area in South Africa a rather different design philosopy was adopted. The authors may care to comment on such an approach.

The gorge of Buffelspoort is formed through a simple anticlinal fold in Table Mountain Series Sandstone. Total gorge length is approximately 13 km. Through the gorge flows a major river which in a 50 year flood would flood the gorge to a height of 8 m. The restricted flow conditions require the road be located some 10 m up the gorge walls with little fill being permitted to further restrict flow.

Resulting cuts would be high and most unattractive. To minimise cut heights vertical or overhanging cuts were considered where ever possible. Natural overhangs of equivalent dimensions suggested that this might be possible.

The jointing patterns as measured in the gorge conform to the classical patterns anticipated for the simple tectonic stress situation which gave rise to the anticlinal fold.

Typical slope cuts were first carefully mapped for structural detail. Fracture data almost invariably were as predicted form the major lineations observed in air photos.

Kinematic modes of failure were determined from great circle analyses on stereoplots.

Stability analyses were generally made numerically. From such analyses failure situations and design measures were determined. Failure modes included plane failure on bedding, wedge failure and toppling.

Friction angle estimates were made from field observations of regions where shear failure had occurred along similar features. A value of 36^{o} was obtained for bedding joints.

High overhang cuts would be controlled largely by cohesion on the vertically intersecting wedges formed by joints. Estimates of such cohesion were made from back analysis of existing overhangs.

It is our intention to form overhang cuts by blasting out sections of the cut, leaving temporary support pillars. The overhang will then be instrumented. The support pillars are then to be blasted out and the performance of the overhang evaluated. Should this be satisfactory the overhang cuts will be retained.

Question by John (for Littlejohn and Bruce)

In considering the quality control of rock anchors should not corrosion control be stressed very strongly?

Reply by Littlejohn

This is a very valid point. Certainly the civil engineering consulting industry in Britain is very interested in this at present. The current philosophy is that for a permanent or temporary anchor application where the consequences of failure would be severe, the anchor must be doubly protected. Every component of the anchor must have two stages of protection, and the first stage must be able to be inspected and tested if necessary, prior to the placement of the tendon into the rock bore hole. The most common technique at present is to have rods, wires or strands pregreased and coated in plastic under factory controlled conditions.

Question by Robertson (for Littlejohn and Bruce)

The following tests on anchor bond and pull out for anchors installed into jointed and bedded quartzitic sandstones may interest the authors, and they may care to comment on the results obtained.

Anchors comprised 12 to 15 mm diameter 3 strand twisted cable yielding a maximum load capacity of 2750 kN. These were grouted into 100 mm diameter percussion drilled holes.

Bond tests consisted of 5 fixed length anchor tests ranging in length from 1 m to 3 m in increments of 0.5 m. Ultimate bond failure was attributed to rock/grout bond failure. Mean stress at failure varied from 5 N/mm^2 for the 1m long anchorage to 2.5N/mm^2 for the 3m long anchorage. Corresponding tendon$_2$ grout mean bond stresses ranged from 2.7 N/mm^2 to 1.3 N/mm^2. Extension at failure was typically about 30 mm.

Five pull out tests were performed, two with total anchor lengths of 3 m and 3 with total anchor lengths of 2m. Anchors were inclined upwards at angles ranging from 24° to 44°.

Classical wedge theory predicts that pull out cones should fall out under their own weight. We were successful in failing only one of the 2 m long anchors at a load of 2200kN. This failure occurred on joint surfaces of classically poor orientation. Preliminary analyses indicate that shear strengths on the cone surfaces of 0.1 N/mm^2 would be conservative. Values of 0.5 N/mm^2 may be applicable.

Reply by Littlejohn

This is exactly the kind of test that should be performed more often. Differing values of apparent uniform bond observed for different anchor lengths may indicate debonding or decoupling. Extension information would have confirmed that. However, non-uniform distribution of bond appears to be clearly shown, confirming what has been known for a long time, and paralleling the results of Chamberlain in the 1940's in reinforced concrete.

It was not quite clear whether stressing was remote from the face or against a load bearing plate on the rock itself. Recently, Bruce has installed about 40 anchors graphically downwards, to investigate load transfer mechanisms at the rock/grout interface and the grout/tendon interface. Anchor lengths range from 0.75 m to 5.0 m. Hopefully the shallow anchors will cause failure in the rock mass.

The effect of cyclic loading is also being investigated; this is important in itself in civil engineering applications.

Question by Barton (for Littlejohn and Bruce)

The authors are to be congratulated on a valuable review article. The following comments relate to recent experiences of the Norwegian Geotechnical Institute concerning anchor pull out tests, which I hope will be of interest here.

We were recently hired by a major Norwegian chemicals firm to estimate the required depth and spacing of peripheral rock anchors to stabilize a 60 metres diameter ammonia storage tank. Under certain gaseous storage phases, an uplift of 3000 tons can be generated.

The rock consisted of nodular limestone and shale, with nearly horizontal bedding planes, and two perpendicular sets of vertical cross-joints. Some of these were calcite coated, and undulating to planar - with occasional steps. The groundwater was at the surface.

In view of the possibility of prismatic block pull out, we neglected the usual conical failure assumption, and concentrated on finding a typical block dimension based on bore core analysis and surface mapping of the joints. If I remember correctly, we settled for a typical block dimension of 60 cm x 150 cm, with depth depending on the depth of the grouted anchor. Bedding features were much more closely spaced. Using different anchor depths we estimated the effective shear resistance generated on the four vertical sides of horizontal stresses. The most pessimistic assumptions indicated that an anchor depth of at least 10 metres was required for a maximum load of some 45 tons. About 70 anchors were needed in all.

This preliminary design was checked by a series of pullout tests at the site. We used two widely spaced abutments as reaction to the 100 ton jack. Six different anchor depths were tested ranging from 1 to 8 metres. We hoped to pin-point the failure depth by using these short lengths. Load-displacement measurements were recorded for a series of load cycles up to 60 tons.

Unfortunately no block pullout occurred, nor conical failures; not even with the 1 metre depth. However one grout bond failed at 60 tons for one of the 1 metre long anchors. We used deformed, 32 mm. diameter, high strength steel bars.

The significant feature of the tests was that several millimeters of rock uplift were occurring for the shorter anchors. This was almost irrecoverable. We interpreted this as a wedging process. The non-planar vertical joints sheared slightly until they had dilated sufficiently to increase the horizontal (or normal) effective stress such that the vertical load was balanced by a greatly increased shear strength.

My question is: how should we estimate the contribution of dilation for design purposes?

Reply by Littlejohn

To begin with, the quality of the grout must affect the occurrence of dilation, say at the interface, depending on how particulate in nature it is. In civil engineering this phenomenon is not relied upon to be effective because it is felt that soft zones may occur with enough lateral yield that the dilation effect is lost. Undoubtedly dilation exists in hard rock though it is not taken into account primarily because of a serious lack of knowledge of stress friction; hopefully this stance is conservative. The kind of information needed, the lateral pressures normal to the axis of the cable as it is being stressed up and down, would be very interesting if it could be obtained; until it can be, reliance on this effect seems unwise.

Further reply by Littlejohn

The displacement we measured for the loads was measured for the rock surface, and not related to the displacement of the anchor or bolts as such; it was the whole block moving up. It was measured some 20 cm away from the anchor on the rock surface.

Question by Ladanyi (for Littlejohn and Bruce)

Do you have any information about the problem of how the cone develops actually, and its shape?

Reply by Littlejohn

(First part of reply lost as not spoken into microphone.)...Invariably the site information received in civil engineering is not extensive; for instance, it is not usual to get unconfined compressive strengths from cores. In our experience geotechnical mapping does not usually give a very good classification of the rock structure. The validity of shear strength parameters is often viewed with suspicion. For these reasons civil engineers tend to rely on a much simpler approach which can't be argued against. Typically, conservative mechanisms of failure are considered; say 60° or 90° cones, taking no account of shear strength, using submerged weight if underneath water, with a factor of safety of 2, and then insisting that every permanent anchor tests to 1.5x that working load.

It does not seem that this practice is going to change for a long time while the present techniques, the simple wedges, give safe and economically attractive solutions.

Question by Bello (for Littlejohn and Bruce)

What do you mean by testing anchors installed at the rock site? Since loading influences only a few meters of anchor length are you really describing pullout tests? Have you found any examples of anchor failure by shearing between the grout and the rock?

Reply by Littlejohn

With reference to the standard routine testing of anchors, all we're really doing is testing the installed anchor system to give a measure of factor of safety. Main interest lies in the safety factor and in the extension or displacement likely to occur in the top anchor in an overload situation. This is important because 70% of the civil engineering market for anchors is in holding back retaining walls. In an urban area, in a deep excavation surrounded by multi-stoney blocks, the client, and the owners of surrounding buildings, are extremely concerned about movements; more so than about safety factors, which is why every single anchor is tested. Of course, also, in rock and soils variations in ground conditions can occur from meter to meter. Anchors may be installed at close centers, but site investigation reports do not have fine accuracy. It is always possible to miss, say, a soft pocket; which is another justification for testing every anchor.

Failure at the ground/grout interface is the most commonly observed type of failure with soil anchors. This is because it is usual to specify crushing grout strengths of 42 N/m^2 (6000 psi), and anchors are not tested until a crushing strength of 28 N/m^2 (4000 psi) has been reached. Interface failures have also been observed in the ground/grout interface in soft sandstones in Britain, with crushing strengths less than about 700 psi. These are weakly cemented sandstones that could almost be dug by hand, and could be regarded by a soil mechanics engineers as a very compact, weakly cemented sand. I have never encountered an interface failure at the rock in materials stronger than that.

Comment by Londe

Like Mr. Littlejohn I am a civil engineer and can concur with his experience. I have been in charge of strengthening or raising the height of about 15 gravity dams by prestressing, involving the use of several thousand cables, some up to 1200 tons capacity. They were all designed using a conventional 90° cone, and a grout assumption for the anchor of just bond. There was not a single failure, which probably indicates that the 90° assumption is too safe, but at this stage is the only thing of which we are sure.

Comment by Hoek

To comment briefly on the question of dilation raised by Barton, attempts have been made at Imperial College by Moy and Boyd to use stereographic methods to try to define the situation better. This is in relation to underground excavation where there is clearly a possibility of gravity pullout. If the jointing is such that no direct gravity wedges are possible then there is the possibility of dilation developing. It is not yet clear how to analyse the contribution of dilation to strength, but very simple stereographic checks have been developed, primarily by Moy, to decide the situation from the structural input; whether strength pullout can occur, sliding on one or two planes, or an interlocking type of situation. This work has been reported in theses at Imperial College.

Question by Gerdeen (for Littlejohn and Bruce)

The values of strength in Table 2 seem to be too low by a factor of 2-4. Please explain.

The results, quoted from the theory of Coates and Yu and from the experiments by Berardi, are for what kind of anchors? (Solid rebar or strands as reinforcement?)

Equation (4) was obtained from experiments. I have used a similar equation from Theory of Elasticity by Timoshenko and Goodier for a concentrated load on a half-plane. Assuming homogeneous material conditions (an approximation) for a fully grouted bolt, it is found that

$$\tau_x = 0.672\ \tau_0 z^2\ (.25 + z^2)^{-5/2}$$

where $z = x/d$, $\tau_x = \tau_0$ (max) at $z = 1/\sqrt{6}$

which also shows that τ_x decreases rapidly with x/d, e.g. $\tau_x = .07\ \tau_0$ when $x = 2d$. This agrees well with Figure 7 for $E_A/E_R = 1.0$.

At Michigan Tech we have also instrumented roof bolts with strain gages. In addition to measuring decay of axial load, however, we have measured the bending of bolts due to interlamellar slip in rock strata.

Reply by Littejohn

With regard to Table 2 you mention that some of the skin friction values probably under the working bond column are low when compared with this general rule in the bottom section. Working bonds quoted in this short table have been recommended by other experimenters or other designers whose reasons for choosing them are not available to me at present. Presumably most of these were tested in the field, and may be the observed minimum value used for design on the basis of individual experience. Often, unfortunately, an anchor solution is asked for purely on the basis of a rock type and perhaps unconfined compressive strength. The general rule given has been formulated in order to achieve some design results on this sort of limited information. It is based on experience in reinforced concrete, where crushing strengths are used. As is frequently done in practice, an indication of the ultimate bond between the reinforcing bar and concrete is obtained by dividing the crushing strength by 10. That is the basis of the present formula, except that an apparent safety factor of 3 is written in, giving rise to the dividing factor 30. This rule is admittedly imprecise, but is useful in the absence of better information.

Discussion by Hargraves

In discussion, Barton gave the instance of a single anchorage failure where a rockbolt of 32 mm deformed bar failed at 60 tonnes. Later in the same discussion Littlejohn quoted rock strengths as low as one sixth grout strength. Apart from the 60 tonne load quoted by Barton appearing to exceed the yield load of the bolt, it seems pertinent to examine the mode of failure of anchorages as the actual mechanism seems open to question. Where bolts are tensioned after grouting with loads approaching yield, the radial strain induced in the bolt might be sufficient to place the grout in radial tension, or with a strong grout and a weak surrounding rock, even to place the rock in radial tension, and tensile failures could give rise to the "debonding" described by Muller (1966). Under normal conditions of testing for both cone shearing and anchorage shearing with a jack around the bolt bearing on the rock face outside a diameter equal to the bolt length, or with remote pulling, from the commencement of the grout for some distance towards the anchorage end of the bolt there must be a reduction in tension in the bolt. This was described in the paper. Failure ("debonding") could be progressive towards the bottom, rather than shearing out of the whole anchorage. Perhaps "debonding" is time-dependent and is accelerated by live loads. It is hard to visualize permanent stability of an anchor bond, even with deformed bar, where grouting is completed prior to tensioning of the tendon and where distribution of tension in the grout is stated to depend on "progressive slip" at the tendon interface. Did the load-extension curves of Barton's anchorage failure throw light on the mechanism of failure?

In mining there is a movement away from mechanical anchorages, with increasing use of resin anchorages. The use of cement grouts is almost unknown in mine roof bolting. Bearing in mind the usual long life of civil works, steps should be taken to waterproof grouts. Investigations should also be made into the deterioration of grout-steel bond due to progressive oxidation of steel by percolating surface waters permeating grout.

Question by John (for Londe and Tardieu)

Fig. 8 is a very sophisticated figure and difficult to follow. In the final publication it would be desirable to have it explained in more detail.

Figs. 13, 14 and 16 represent replacement patterns of a buttressed dam. What were the actual displacements in mm at the crest?

Reply by Londe

It may be necessary to refer to a previous paper in order to explain Fig. 8 more fully.

Regarding Figs. 13, 14 and 16, from memory, the scale of displacement is full scale. So, for instance, if the crest has moved by 20 mm on the drawing, this is the actual movement. The dam is about 50 m high.

Question by Gerdeen (for Londe and Tardieu)

Your finite element modeling of bolted joints, Fig. 5 is interesting for we have conducted similar anlayses. Is the model represented in Fig. 5, two-dimensional or three-dimensional? Were joint stiffness values K_n and K_s measured or assumed? What are realistic values for joints?

Have you achieved any results on the three-dimensional finite element model with discontinuities? Have you analyzed the effect of bolt reinforcement in the 3-D model?

Reply by Londe

Fig. 5 shows a 2-dimensional representation of anchor bars in rock only, as it is part of a preliminary study at present underway; since the situation is 2-dimensional it is known to be an approximation.

The values of stiffness coefficients K_n and K_s are assumed, not measured; the actual measurement of these two stiffnesses is currently under study. Hence, while the values given are values quoted in the literature, and are not ridiculuous, it is not possible to say that they are realistic for the case in point.

We hope to be able to report on 3-dimensional analyses next year.

Comment by Kanji

In relation to the factor of safety, Prof. John has already mentioned probablistic failure analysis and M. Londe has in a previous paper questioned the value of factor of safety, but today has allowed that it is a very good sensitivity index. If anyone present has had experience of probabilistic analysis of geotechnical work, along the lines used by structural engineers in their concrete structures, would they care to comment about it? Since several different sets of statistical data may be required for one rock mass, there are many more problems in the analysis of rock than of concrete, which is more or less homogeneous. While it would be interesting to hear of any probabilistic methods of analysis currently in use, it must be pointed out that so much information has accumulated about the factor of safety concept and so little, as yet, on the probabilistic approach, that it seems doubtful that the latter is yet in a position to supersede the former.

Comment by Einstein (resubmitted at later date)

A few comments and questions in Sessions 1 and 2 have dealt with probabilistic approaches. In addition, the selection and computation of safety factors was discussed extensively. The fact that the safety factors were discussed with no or only passing reference to the probabilistic approaches leads me to make the following comments:

Safety factors and probabilistic approaches represent a recognition of the fact that natural materials and phenomena and their descriptions involve uncertainty. Thus, uncertainty has to be taken into account in the analysis and design.

The presently most common approach to uncertainty is the use of the safety factors, which can be employed with various degrees of sophistication:

- Frequently, an upper and a lower factor are chosen, accounting for a variation of the expected performance, but within fixed boundaries. The selected numerical values do, however, not have a rational correlation with the likelihood of failure.

- The use of the safety margin, i.e., the difference between the actual safety factor and a safety factor of 1, is similar to the aforementioned approach and is subject to the same limitations. The use of a safety factor of 1 as a lower boundary ensures at least a more conscious consideration of the possibility of failure.

- Partial safety factors represent a more advanced application of the safety factor approach. The assignment of partial safety factors takes into account that the uncertainty of different design parameters may be different and it can express the fact that not all parameters are of equal importance.

This short review of the safety factor approach shows that the uncertain or probabilistic nature of design is implicitly assumed. However, safety factors are not a rigorous means of expressing uncertainty since they do not correlate the state of a structure to the likelihood of failure: even for the highest factor of safety, there is a finite likelihood of failure (or vice versa, a safety factor of 1 or below does not mean that the structure actually fails).

Probabilistic approaches to design can remedy this situation, particularly if they are applied in the context of risk analysis, which can be described in a simplified manner, as follows:

The probability of failure of a certain structure (e.g., a slope) is multiplied with the cost consequences of this failure. An improved design will reduce the probability of failure and thus result in a reduced potential cost consequence. The difference in potential failure cost consequences between the two designs is compared to the cost of design improvements. In this manner, risk of failure can be rationally expressed and risk modification can be compared to the cost of remedial measures.

Probabilistic approaches to design are naturally not a panacea. They force the designer, however, to take uncertainty explicitly into consideration and to be aware of the ever present risk of failure. As has been shown above, probabilistic design approaches are the only ones that permit a complete evaluation of design alternatives.

The present state of application of probabilistic approaches leaves still much to be desired as Dr. John correctly pointed out in his general report.

The present limitations are:

- Methodology: The designer does not know how to incorporate probabilistic approaches in the design process.

- Analysis: Although the probabilistic techniques are analytically formulated, not many engineering analyses do exist as yet that are formulated in a probabilistic manner.

- Input Parameters: Probabilistic approaches require parameter input in the form of distributions. Present exploration and testing techniques frequently do not yield such distributions, and the experience with subjective techniques is limited.

To conclude, it can be stated that probabilistic approaches will play a very important role in the design process; that, however, more work is needed to make these approaches practically applicable.

Comment by Londe

Dr. Kanji said that we have to make reference to the conventional factor of safety as we know it. However, we don't know it in rock mechanics. Safety factors of slopes already existing are not known; neither are safety factors for foundations. It is impossible to know the safety factor. It is known that some slopes are stable and some are not, so that we have an idea of limiting cases it is true, but we have a very poor idea of actual safety factor in existing structures.

In my opinion it is possible to use probabilistic reasoning in rock mechanics, but not as a tool for giving a figure, a numerical appreciation of the probability of failure, because there is the problem of measuring the parameter distribution curves. This is a methodology problem, as Dr. Einstein remarked. There are too many parameters that we are not in a position to measure; but we can obtain a model giving us a guideline for studying the problem. In an expression for the total safety of a structure, bank, slope or foundation, those terms with high weight in the expression, and hence an important effect on stability, must be identified. They may have high weight for one of two reasons: either because they strongly affect equilibrium or because they are not well known.

Comment by Kanji

I agree with what you say. My point, however, is that to altogether put aside a way of reasoning with which we are familiar in favour of a new unfamiliar way may be dangerous.

Comment by Sowers (resubmitted at later date)

The technical papers concerned with the anlaysis of rock slope stability and the remarks of Dr. K. John in his review of these papers give rise to two serious concerns in the applications of analytical techniques to the design of permanent (long life) slopes in civil engineering works.

First, the realism of the results of any of the analytical techniques proposed depends on the reliability of the physical properties of the rock and rock joints (as well as their geometric boundaries). Although these properties are treated as constants by the authors, they are really variables subject to the vagaries of environmental change.

The effects of stress relief due to excavation and pore fluid pressure along joints are the only two environmental changes considered by most of the analyses. However, in long-term exposure, other factors may be equally or even more significant.

The change of physical properties of both the intact rock between joints and any joint filling with continued weathering are not mentioned. While the changes that can occur in intact competent igneous rock during the lifetime of a civil engineering project may be insignificant, they are not in shales, mudstones, poorly-indurated sandstones, tuffs, partially weathered crystalline rocks and limestones. For example, failures of highway cuts in shale in Tennessee often occur 10 years after the cut was made. During the ensuing period, there may be little signs of instability. Similar delayed failures have been experienced in the other formations listed. By way of contrast, some tuffs and sandstones can gain strength and stability upon exposure, particularly in dry climates. Such surface hardening has led to an over-estimate of the rock strength based on the observed behavior of old slopes. Such over-estimates have been accompanied by under-design and early failure of new cuts.

Temperature changes, erosion of joint filling materials and frost action are other dimensions of environmental change that can cause both over- and under-design.

The second concern is the comparison of mine slopes with civil engineering slopes. The implication is that the civil engineers might be able to reduce their conservatism by proper programs of slope monitoring and maintenance. Unfortunately, the comparison is not necessarily valid: the dimensions of time and ultimate responsibility are different.

The mining engineer has a well-defined responsibility toward his employers and fellow employees for maintaining an economic balance between initial construction (production) costs and maintenance. Moreover, the designer is more or less involved in both initial excavation and any corrective measures necessary, including the essential monitoring of the performance of the initial design. The combination of responsibility for safety and economy to a narrow segment of society, coupled with the authority to act when monitoring dictates maintenance, provides an optimum combination for overall economy.

Unfortunately, this is not always the case in civil engineering work, particularly that involving public projects. The completed project must function for years. Despite the designer's plans for surveillance and continued maintenance, future decisions will be made by others, sometimes decades later. New administrators may not appreciate the need for such activities that create nothing new. During periods of financial stringency, maintenance and surveillance programs are among the earliest expenditures that are curtailed. Therefore, ideally, the civil engineering work should be designed with margins of safety that minimize future maintenance.

On the other hand, the difference in availability between construction money, a capital expenditure, and maintenance money, an annual expense, sometimes reverses the obvious. Maintenance expenditures provide instant, local jobs and the costs are often lumped together so that they cannot be identified with any specific project. Once a project is constructed, the designer may have no further responsibility for it and frequently has no reports on performance from the maintenance forces. For example, on one highway across a steep mountainside in a shale sandstone complex the design slopes, based on experience, did not reflect either the joint patterns or the continued rock weathering on exposure. Sliding developed in both cuts and fills. Traffic was maintained by filling and repaving as needed, sometimes daily. Borings made 12 years later found 40 ft. thickness of asphalt paving in one area. No records were available regarding the cost of repairs except for the asphalt that was purchased. Based on estimated costs of labor for repair, the slide maintenance cost far more than the original construction. Because the cost could not be pinpointed, the administrators were not concerned; further, the asphalt supplier was quite satisfied with the original design.

Question by Bello (for Von Thun)

Would you comment on the possibility of excavation of the upper part of a slope to increase the factor of safety?

Reply by Von Thun

This is not a really critical aspect when strength is governed by the tangent of the friction angle, depending somewhat on the particular strength curve for the material. In intact material, or where cohesion is significant, excavation in the upper part of the slope could be important, if the friction was quite small.

An analogy exists with the case of a block sitting on a plane. With a linear friction relationship, increasing the size of the block makes no difference to the stability analysis. With a curvilinear relationship the weight has an effect, though not a large one.

Hence the answer to your question is that some effect is achieved if there is a curvilinear relationship, but the effect is not large.

To comment on a remark by Dr. Londe I would like to mention an example of a 3-dimensional analysis of an arch dam, that was inexpensive because it was reduced to the simplest elements. Vertical stringers of talc were present. The design object was to make the foundation able to carry the shear stresses across these zones. By putting the load on in an appropriate manner it was possible to determine the stress to be passed through the talc. Then replacing the talc with concrete in the finite element analysis showed how high it would be stressed, and helped determine how much concrete should actually be placed with a certain margin of safety. In this way the foundation was knitted together prior to the placing of the arch dam.

Question by Lindner (for John)

In Session 1 Dr. John commented on the horizontal stress field, questioning the values .33 and .5 presented by Kalkani and Manfredini et. al. respectively. Would he give his opinion on reported horizontal stress fields of orders 2 or 3 times greater than vertical stresses? (Reported by Sbar and Sykes (1973), Hooker and Johnson (1964).)

Reply by John

That is a very good question that can only be answered in individual situations, not necessarily from statistical surveys, which was the point of my mild objection. I know of projects, particularly tunneling projects, that have been dominated by high longitudinal stresses. The problem is to ascertain them; this may possibly be achieved by testing, by back analysis, from data used by other people, or may even require guesswork, and/or upper and lower bound analyses. There is no clear cut path to the information. The assumptions I objected to are realistic. This does not answer the question, but is as much as I can say.

Comment by Franklin

Concerning the last question, I have found that on arrival in Canada from Europe I have had to change my mind very rapidly concerning the state of stress in the ground and its effects on rock cuts and rock slopes. I have been used to thinking of possibly some elastic gravity loading model where the horizontal stress was less than the vertical, but since arriving in Canada have seen numerous cases where high horizontal stress is evident in rock cuts. For example, in the excavation for the C.N. tower in Toronto, the tallest free standing structure in the world, the rock bed was moving into the excavation on bedding planes in a way that can only be explained by high horizontal stresses. Monitoring of excavations in the vicinity of the Niagara gorge over the last 50 years indicate that movement is still going on. These are deep excavations, vertical slopes in the ground. The point is that stresses must not be taken for granted in slope stability; it is necessary to consider the environment, the geographical location and the experience of people working in that area.

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