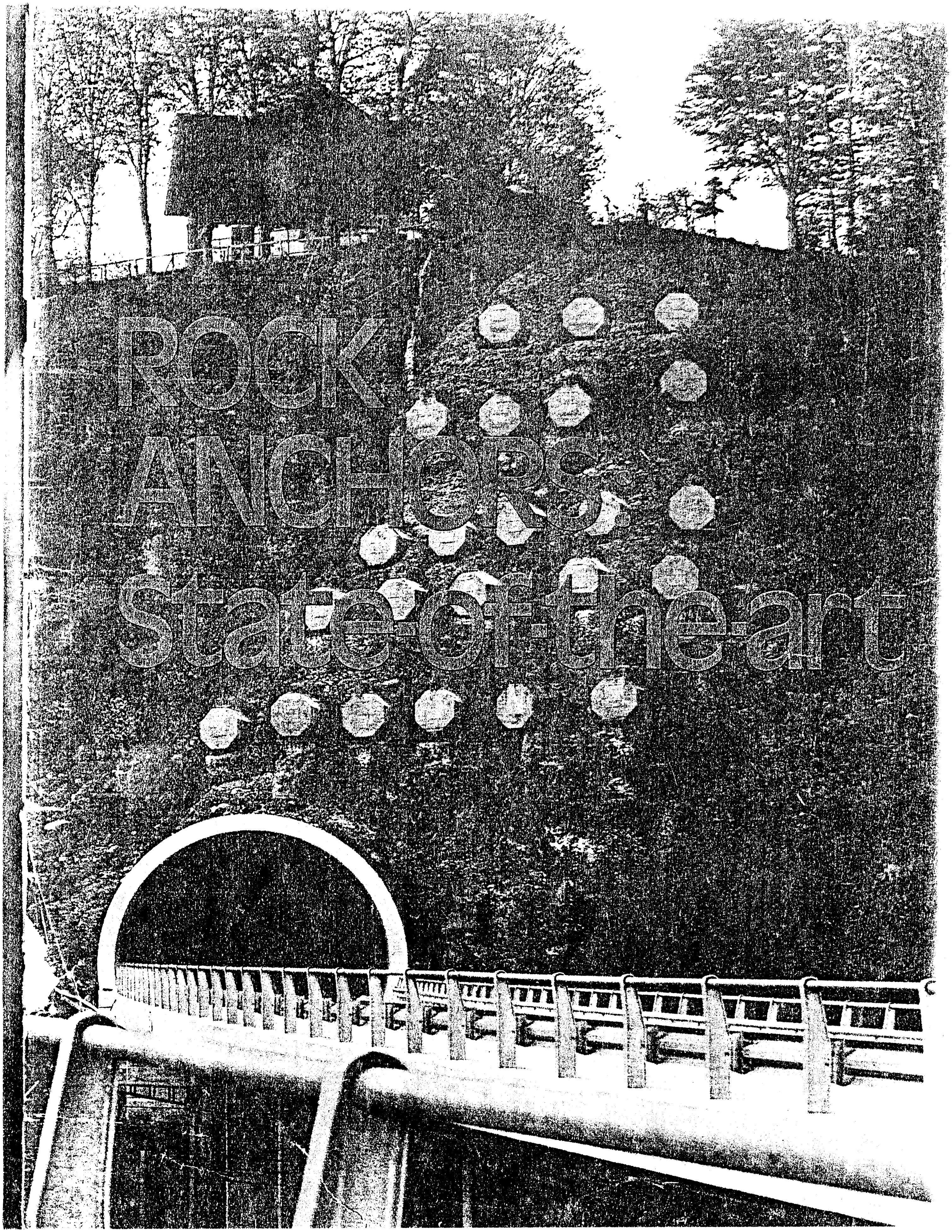
ROCK
ANCHORS
State-of-the-art

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Donald Bruce was educated at Robert Gordon's college in Scotland and received his first degree in Geology at Aberdeen University. After this he continued his graduate study as a member of the Geotechnic's Research Group at the University's Department of Engineering. During a period of three years he studied the design and performance of prestressed rock anchors with particular reference to load transfer mechanisms.

After receiving his doctorate Donald Bruce joined the Northern office of Colcrete Ltd. as a contracts engineer and is currently involved in ground stabilisation including supervision of dam grouting contracts. He is a member of the British Drilling Association and the International Society of Rock Mechanics' Committee reviewing rock anchor testing.

Rock anchors - state of the art

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Part 1: Design

INTRODUCTION

THE HISTORY OF prestressed rock anchors dates from 1934, when the late Andre Coyne pioneered their use during the raising of the Cheurfas Dam, in Algeria. Since then, the employment of rock anchors in dam construction has become world wide, and several million tons of working capacity have been successfully installed. Rock anchors have also been used for many years to ensure the safety of large underground excavations and the stability of natural and artificial rock slopes.

In recent years the range of applications has widened considerably due in part to the success achieved by soil anchors in tying-back retaining walls, holding down dock floors, and pile testing. Now, largely as a consequence of the success of anchors in these new applications, 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 200t/m may be required, necessitating individual anchors of capacity well in excess of 1 000 tonnes. In the field of suspension bridges concentrated groups of anchors with a working capacity of 6 000t are already being seriously considered, and design loads of 15 000t 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 and realistic safety factors together with relevant quality controls and testing procedures.

The first article in this state-of-the-art review, therefore, considers design procedures relating to overall stability, grout/rock bond, tendon/grout bond, and tendon, along with the choice of safety factors. The second article deals with the practicalities of installation, construction and quality control, whilst the third examines testing and stressing procedures§.

The purpose of this general appraisal is to describe current practice in relation to rock anchors by drawing on the experience gained in various countries over the past 30 years. Experimental and theoretical studies in the fields of reinforced and prestressed concrete are also included where relevant. It is hoped that the information provided will be of direct benefit to anchoring specialists but, at the same time, the series of articles are intended as a basis for discussion since points are highlighted concerning the validity of the basic design assumptions, and the lack of knowledge of full-scale anchor performance.

DESIGN—AN INTRODUCTION

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.

Broadly speaking, present design criteria may be classified into two equally unsatisfactory groups. On the one hand there are the procedures based on the classical theory of elasticity. Clearly, the validity of results derived from, for example, photoelastic or finite element techniques dependent on such a theory, is questionable when dealing with a heterogeneous rock mass. On the other hand, anchor parameters are frequently selected by, at best, crude empirical rules or trial and error methods, and at worst, by pure guesswork. The gap between these two extremes is still very real, despite a growing awareness of the problems, as witnessed by the recent appearance of standards or draft codes on ground anchors in several countries.

The main design concepts are now reviewed with respect to the four failure modes listed above, but it should be emphasised that these concepts relate primarily to prestressed cement grout injection anchors.

UPLIFT CAPACITY OF THE ROCK ANCHOR SYSTEM

Design procedures

This section deals with methods currently used in practice to estimate the anchor depth required to ensure that the working load will be resisted safely without failure occurring in the rock mass. The methods described apply to anchors which have been constructed in a vertical or steeply inclined downwards direction.

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). The uplift capacity is normally equated to the weight of the specified rock 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 manner may, of course, be reduced where it can be demonstrated by test anchors that the working force can be otherwise achieved 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. This approach has been employed by many engineers in practice and is described by Parker (1958), Hobst (1965), Littlejohn (1972) and Hilf (1973).

However, although the shape of the fail-

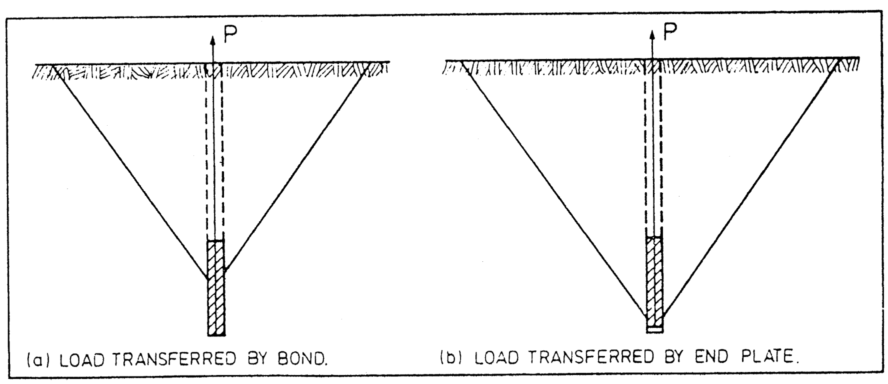


Fig. 1. Geometry of cone, assumed to be mobilised when failure occurs in a homogeneous rock mass

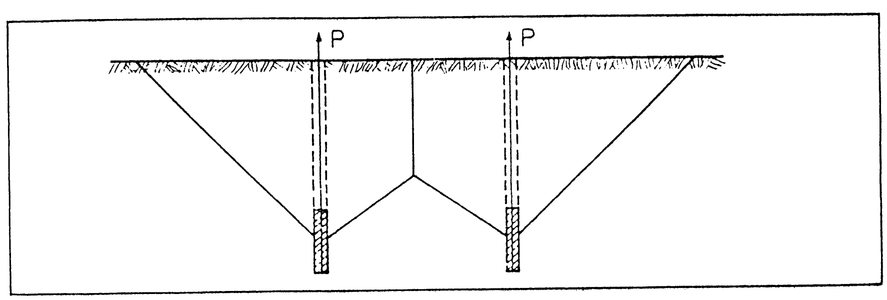


Fig. 2. Interaction of inverted cones in an overall stability analysis

TABLE I—GEOMETRIES OF ROCK CONE RELATED TO FIXED ANCHOR WHICH HAVE BEEN EMPLOYED IN PRACTICE

Geometry of inverted cone		*Source*
Included angle	*Position of apex*	
60 deg	Base of anchor	Canada—Saliman & Schaefer (1968)
60 deg	Base of anchor	USA—Hilf (1973)
90 deg	Base of anchor	Britain—Banks (1955)
90 deg	Base of anchor	Britain—Parker (1958)
90 deg	Base of anchor	Czechoslovakia—Hobst (1965)
90 deg	Base of anchor	USA—Wolf et al (1965)
90 deg	Base of anchor	Canada—Brown (1970)
90 deg	Base of anchor	Australia—Longworth (1971)
90 deg	Base of rock bolt	USA—Lang (1972)
90 deg	Base of anchor	USA—White (1973)
90 deg	Base of anchor where load is transferred by end plate or wedges	Germany—Stocker (1973)
90 deg	Middle of grouted fixed anchor where load is transferred by bond	Germany—Stocker (1973)
90 deg	Middle of anchor	Britain—Morris & Garrett (1956)
90 deg	Middle of anchor	India—Rao (1964)
90 deg	Middle of anchor	USA—Eberhardt & Veltrop (1965)
90 deg	Top of fixed anchor	Australia—Rawlings (1968)
90 deg	Top of fixed anchor	Austria—Rescher (1968)
90 deg	Top of fixed anchor	Canada—Golder Brawner (1973)
*60 deg-90 deg	Middle of fixed anchor where load is transferred by bond	Britain—Littlejohn (1972)
*60 deg-90 deg	Base of anchor where load is transferred by end plate or wedges	
90 deg	Top of fixed anchor	Australia—Standard CA35 (1973)
	or	
60 deg	Base of anchor	

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

ure volume is widely agreed, its position with respect to the grouted fixed anchor length (socket) varies somewhat in practice. This aspect is illustrated by Table I, which contains examples drawn from anchor designs in various countries. Another feature which is widely appreciated, but receives little attention is that a solid, homogeneous rock mass is seldom encountered, and so, in the vast majority of cases, modifications to the simple cone approach should be made by experienced rock mechanics engineers.

In connection with this "weight of rock" method of calculating the ultimate resistance to withdrawal, little data are available on the safety factors employed. However, it is known that safety factors of 3 and 2 have been used by Schmidt (1956) and Rawlings (1968) respectively, while most recently a factor of 1.6 was employed for anchors at the Devonport Nuclear Complex by Littlejohn and Truman-Davies (1974). 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, of course, greatly reduced when anchors are installed in highly fissured "loose" rock masses, especially those with much interstitial material or high pore water pressure. This point was recognised by Hobst (1965) when he presented the formulae given in Table II for calculating the depth of the cone; in these

τ = shear strength of rock (tonnes/m^2)
F = factor of safety against failure (F = 2-3 customary)
s = spacing of anchors (metres)
ϕ = angle of friction across fractures in rock mass
γ = specific gravity of rock (tonnes/m^3)

Note that the shear strength is considered in dealing with anchors in homogeneous rock, whereas rock weight is the dominant parameter when dealing with fissured rock masses. In Britain, the shear strength parameter is usually ignored in practice (thus erring conservatively) since quantitative data on the fracture geometry and shear strength of the rock mass are seldom available at the design stage. 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 per cent of the "ideal" laboratory dry value, and occasionally to as low as 4 per cent of this figure.

Other engineers confirm that rock shear strength generally contributes a major component of the ultimate pull-out resistance. Brown (1970) states that the ultimate capacity of an anchor, in homogeneous, massive rock, is dependent on the shear strength of the rock and the surface area of the cone, which for a 90 deg cone is proportional to the square of the depth of embedment i.e. $4\pi h^2$. Usually a maximum allowable shear stress is specified, acting over the cone surface e.g. 0.034N/mm^2 (Saliman and Schaefer, 1968)). Hilf (1973) advocates that regardless of rock type a value of 0.024N/mm^2 may be allowed and specifies a safety factor of 2 on a test load displacement of up to 12mm. Values in excess of 0.024N/mm^2 may be used if verified by field tests.

Experimental evidence

In general, there is a dearth of data on anchor failures in the rock mass but a set of tests which provides some results on the overall stability aspect is presented by Saliman and Schaefer (1968) who describe the failure of grouted bars on the Trinity Clear Creek 230kV transmission line. Four tests were carried out on deformed reinforcement bars grouted into 70mm diameter holes to a depth of 1.52m 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). Assuming a bulk density of 2Mg/m^3 for the rock, back analysis of the failure loads indicates very conservative results—safety factors on the pull-out load between 7.4 and 23.5—if the apex of the 90 deg 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.

However, in 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).

Spacing

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 rock bedding planes occur normal to the anchor axis, the purpose of staggered lengths is to reduce the intensity of stress across such planes at the level of the fixed anchors.

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 length in closely spaced anchor groups.

The South African Recommendations (1972) suggest that in the case of a "concentrated" group, 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 recommended for the Devonport Nuclear Com-

TABLE II—DEPTH OF ANCHOR FOR OVERALL STABILITY (after Hobst, 1965)

Rock type	*Formula for depth of cone*	
	One anchor	*Group of anchors*
"Sound" homogeneous rock	$\sqrt{\frac{F.P}{4.44\,\tau}}$	$\frac{F.P}{2.83\,\tau\,.\,s}$
Irregular fissured rock	$\sqrt[3]{\frac{3F.P}{\gamma\pi\,\tan\phi}}$	$\sqrt{\frac{F.P}{\gamma\,.s.\,\tan\phi}}$
Irregular submerged fissured rock	$\sqrt[3]{\frac{3F.P}{(\gamma-1)\,\pi\,\tan\phi}}$	$\sqrt{\frac{F.P}{(\gamma-1).s\,.\,\tan\phi}}$

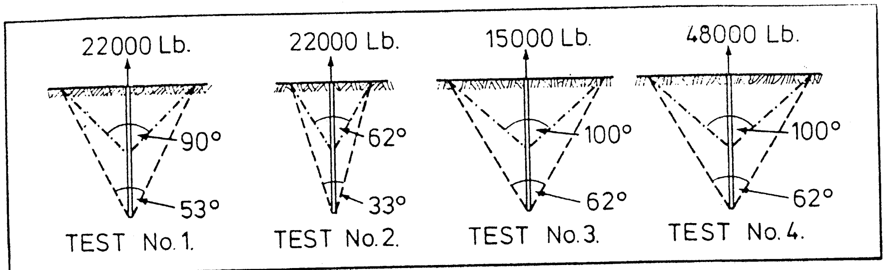


Fig. 3. Possible failure modes based on test results at Trinity Clear Creek
(after Saliman and Schaefer, 1968)

plex by Littlejohn and Truman-Davies (1974) where 2 000kN anchors were spaced at 1m 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, Algeria. In some other countries a minimum distance between anchors is stipulated. Broms (1968), reviewing Swedish practice, confirmed a minimum spacing of 2.5m, whilst the Czech Standard (1974) recommends 1.5m, one consideration being to reduce "interhole grouting", although this phenomenon is not necessarily a disadvantage in practice.

It is noteworthy that these guide rules or approaches are based on experience and engineering judgement, and not on an intimate knowledge of stress distribution around the anchor.

Remarks

With regard to uplift capacity no experimental or practical evidence and only very little theoretical data substantiate the methods currently used (Table I) to calculate the ultimate resistance to pull-out of individual, or groups of anchors. Indeed, there would appear to be results (Saliman and Schaefer (1968) and Brown (1970)) which indicate that failure in a rock mass does not generally occur in the form of an inverted 90 deg cone or wedge. However, it is reassuring to know that most designs are likely to be conservative in adopting a cone method with no allowance for the shear strength of the rock mass.

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.

In general, 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.

BOND BETWEEN CEMENT GROUT AND ROCK

Introduction

Most designs to date concerning straight shaft fixed anchors have been successfully based on the assumption of uniform bond distribution over the fixed anchor surface area. In other words it has been generally accepted that the bond developed is merely a function of fixed anchor dimensions and applied load.

However, recent experimental and theoretical analyses have indicated that the character of the bond to the rock is more complex, and reflects additional parameters which often give rise to a markedly non-uniform stress distribution. Thus, in many cases the assumed mechanism of load transfer in the fixed anchor zone may be grossly inaccurate. For example, the situation could well arise where, for a high capacity anchor, the level of bond stress at the loaded (or proximal) end may be extremely high, possibly approaching failure, whereas the more distal parts of the fixed anchor may in effect be redundant. Clearly, such a situation will have a bearing on overall stability analyses, the interpretation of anchor extensions, and long-term creep behaviour.

Design criteria are reviewed relating to the magnitude and distribution of bond, fixed anchor dimensions, and factors of safety. For comparison, the results of relevant theoretical and experimental investigations are presented.

Fixed anchor design

The straight shaft anchor relies mainly on the development of bond or shear in the region of the rock/grout interface, and as described by Littlejohn (1972) it is usual in Britain to assume an equivalent uniform distribution of bond stress along the fixed anchor. Thus the anchor load, P, is related to the fixed anchor design by the equation:

$$P = \pi d L \tau \qquad (1)$$

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

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

The rule is based on the following simple assumptions:

(*i*) Transfer of the load from the fixed anchor to the rock occurs by a uniformly distributed stress acting over the whole of the curved surface of the fixed anchor.
(*ii*) The diameter of the borehole and the fixed anchor are identical.
(*iii*) Failure takes place by sliding at the rock/grout interface (smooth borehole) or by shearing adjacent to the rock/grout interface in weaker medium (rough borehole).
(*iv*) There are no discontinuities or inherent weakness planes along which failure can be induced, and
(*v*) There is no local debonding at the grout/rock interface.

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 (UCS) is less than 7N/mm², 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 one-tenth of the uniaxial compressive strength of massive rocks (100 per cent core recovery) up to a maximum value τ_{ult} of 4.2N/mm², assuming that the crushing strength of the cement grout is equal to or greater than 42N/mm². Applying an apparent safety factor of 3 or more, which is conservative bearing in mind the lack of relevant data, the work-

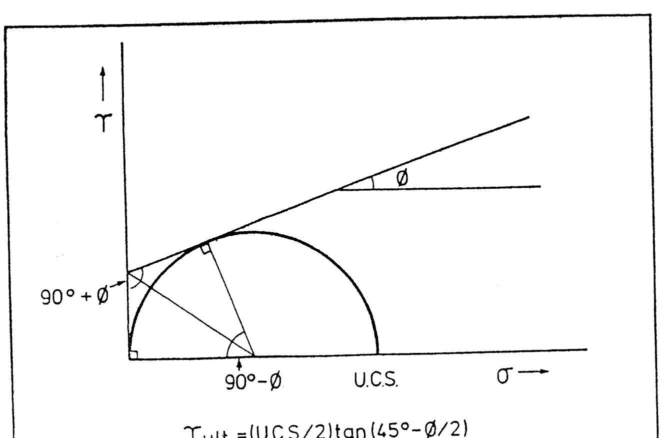


Fig. 4. Relationship between shear stress and uniaxial compressive strength

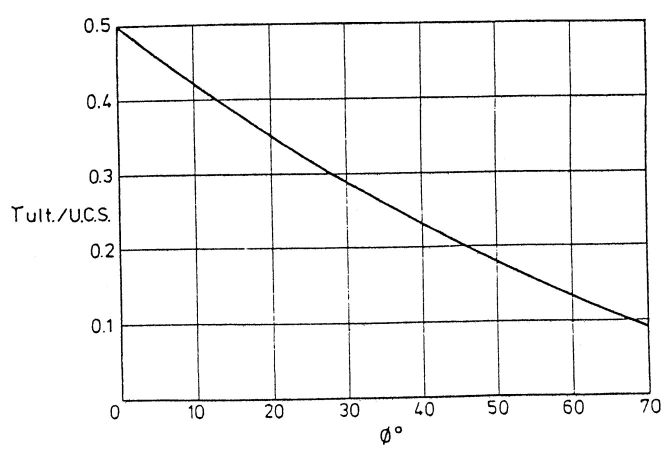


Fig. 5. Effect of ϕ on $\tau_{ult.}/UCS$ ratio

ing bond stress is therefore limited to 1.4N/mm².

In this connection it is noteworthy that Coates (1970) allows a maximum working bond of 2.45N/mm² but with a safety factor of 1.75, which indicates a value of τ_{ult} of 4.3N/mm². In some rocks, particularly granular, weathered varieties with a relatively low ϕ value, the assumption that τ_{ult} equals 10 per cent rock UCS may lead to an artificially low estimate of shear strength (Figs. 4 and 5). In such cases, the assumption that τ_{ult} equals 20-35 per cent UCS may be justified.

As a guide to specialists, bond values, as recommended throughout the world for wide range of igneous, metamorphic and sedimentary rocks, are presented in Table III. Where included, the factor of safety relates to the ultimate and working bond values, calculated assuming uniform bond distribution. It is common to find that the magnitude of bond is simply assessed by experienced engineers; the value adopted for working bond stress often lies in the range 0.35 to 1.4N/mm². Koch (1972) suggests bond stresses in this range for weak, medium and strong rock (Table III), and the Australian Code CA 35—1973 states that a value of 1.05N/mm² has been used in a wide range of igneous and sedimentary rocks, but confirms that site testing has permitted bond values of up to 2.1 N/mm² to be employed.

In this connection the draft Czech Standard (1974) concludes that since the estimation of bond magnitude and distribution is a complex problem, field anchor tests should always be conducted to confirm bond values in design, as there is no efficient or reliable alternative. Certainly, a common procedure amongst anchor designers is to arrive at estimates of permissible working bond values by factoring the value of the average ultimate bond calculated from test anchors, when available. 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 varieties.

The degree of weathering of the rock is

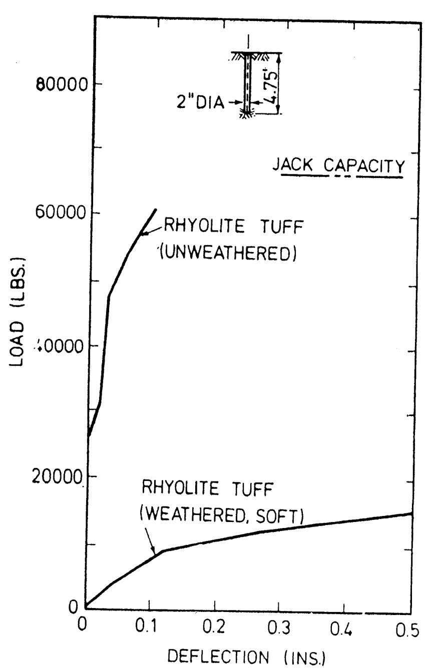


Fig. 6. Effect of weathering at Currecanti Midway Transmission Line

(after Saliman and Schaefer, 1968)

TABLE III—ROCK/GROUT BOND VALUES WHICH HAVE BEEN RECOMMENDED FOR DESIGN

Rock type	*Working bond (N/mm²)*	*Ultimate bond (N/mm²)*	*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	Britain—Wycliffe-Jones (1974)
Serpentine	0.45-0.59	1.55	2.6-3.5	Britain—Wycliffe-Jones (1974)
Granite & basalt		1.72-3.10	1.5-2.5	USA—PCI (1974
Metamorphic				
Manhattan schist	0.70	2.80	4.0	USA—White (1973)
Slate & hard shale		0.83-1.38	1.5-2.5	USA—PCI (1974)
Calcareous sediments				
Limestone	1.00	2.83	2.8	Switzerland—Losinger (1966)
Chalk—Grades I-III	0.01N (N=SPT in blows/0.3m)	0.22-1.07	1.5-2.0 (Temporary) 3.0-4.0 (Permanent)	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	Britain—Wycliffe-Jones (1974)
Soft limestone		1.03-1.52	1.5-2.5	USA—PCI (1974)
Dolomitic limestone		1.38-2.07	1.5-2.5	USA—PCI (1974)
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 mudstones		0.69	2.0-2.5	New Zealand—Irwin (1971)
Bunter sandstone	0.40		3.0	Britain—Littlejohn (1973)
Bunter sandstone (UCS > 2.0N/mm²)	0.60		3.0	Britain—Littlejohn (1973)
Hard fine sandstone	0.69-0.83	2.24	2.7-3.3	Britain—Wycliffe-Jones (1974)
Sandstone		0.83-1.73	1.5-2.5	USA—PCI (1974)
Argillaceous sediments				
Keuper marl		0.17-0.25 (0.45 C_u)	3.0	Britain—Littlejohn (1970) C_u=undrained cohesion
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	USA—PCI (1974)
General				
Competent rock (where UCS > 20N/mm²)	Uniaxial compressive strength—30 (up to a maximum value of 1.4N/mm²)	Uniaxial compressive strength—10 up to a maximum value of (4.2N/mm²)	3	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 variety of igneous and metamorphic rocks	1.05		2	Australia—Standard CA35 (1973)
Wide variety of rocks	0.98			France—Fargeot (1972)
	0.50			Switzerland—Walther (1959)
	0.70			Switzerland—Comte (1965)
		1.20-2.50		Switzerland—Comte (1971)
	0.70		2-2.5 (Temporary) 3 (Permanent)	Italy—Mascardi (1973)
	0.69	2.76	4	Canada—Golder Brawner (1973)
	1.4	4.2	3	USA—White (1973)
		15-20 per cent of grout crushing strength	3	Australia—Longworth (1971)
Concrete		1.38-2.76	1.5-2.5	USA—PCI (1974)

TABLE IV—ROCK/GROUT BOND VALUES WHICH HAVE BEEN EMPLOYED IN PRACTICE

Rock type	*Working bond (N/mm²)*	*Test bond (N/mm²)*	*Ultimate bond (N/mm²)*	s_m *(test)*	s_f *(ultimate)*	*Source*
Igneous						
Basalt	1.93		6.37		3.3	Britain—Parker (1958)
Basalt	1.10	3.60				USA—Eberhardt & Veltrop (1965)
Tuff	0.80					France—Cambefort (1966)
Basalt	0.63	0.72				Britain—Cementation (1962)
Granite	1.56	1.72				Britain—Cementation (1962)
Dolerite	1.56	1.72				Britain—Cementation (1962)
Very fissured felsite	1.56	1.72				Britain—Cementation (1962)
Very hard dolerite	1.56	1.72				Britain—Cementation (1962)
Hard granite	1.56	1.72				Britain—Cementation (1962)
Basalt & tuff	1.56	1.72				Britain—Cementation (1962)
Granodiorite	1.09					Britain—Cementation (1962)
Shattered basalt		1.01				USA—Saliman & Schaefer (1968)
Decomposed granite		1.24				USA—Saliman & Schaefer (1968)
Flow breccia		0.93				USA—Saliman & Schaefer (1968)
Mylontised porphyrite	0.32-0.57					Switzerland—Descoeudres (1969)
Fractured diorite	0.95					Switzerland—Descoeudres (1969)
Granite	0.63	0.81				Canada—Barron et al (1971)
Metamorphic						
Schist	0.31					Switzerland—Birkenmaier (1953)
All types	1.20					Finland—Maijala (1966)
Weathered fractured quartzite	1.56	1.72		1.1		Britain—Cementation (1962)
Blue schist	1.52	1.67		1.1		Britain—Cementation (1962)
Weak meta sediments	1.10	1.23		1.1		Britain—Cementation (1962)
Slate	0.43					Britain—Cementation (1962)
Slate/meta greywacke	1.57	1.73		1.1		Britain—Cementation (1962)
Granite gneiss	0.36-0.69					Sweden—Broms (1968)
Folded quartzite	0.51					Australia—Rawlings (1968)
Weathered meta tuff		0.29				USA—Saliman & Schaefer (1968)
Greywacke	0.34					Germany—Heitfeld & Schaurte (1969)
Quartzite	0.93-1.20	1.02-1.32		1.1		Britain—Gosschalk & Taylor (1970)
Microgneiss	0.95					Italy—Mantovani (1970)

Rock	Working bond (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	s_m (test)	s_f (ultimate)	Source
Metamorphic—contd.						
Sericite schist	0.05					Italy—Berardi (1972)
Quartzite/schist	0.10					Italy—Berardi (1972)
Argillaceous & calcareous schist	0.63					Italy—Berardi (1972)
Slate	0.95	1.24		1.3		Switzerland—Moschler & Matt (1972)
Highly metasediments	0.83	1.08		1.3		USA—Buro (1972)
Slate & greywacke	1.08	1.40		1.3		Germany—Anon (1972)
Various metasediments			1.57			Germany—Abraham & Porzig (1973)
Micaschist/biotite gneiss	0.53	0.80		1.5		USA—Nicholson Anchorage Co. Ltd. (1973)
Slate	0.60	0.90	1.80	1.5	3.0	Britain—Littlejohn & Truman-Davies (1974)
Sound Micaschist	1.74	2.16				USA—Feld & White (1974)
Micaschist	0.52-0.74			1.24		USA—Feld & White (1974)
Very-poor gneiss & mud-band	0.07					USA—Feld & White (1974)
Carbonate sediments						
Loamy limestone		0.63				Italy—Berardi (1960)
Fissured limestone & intercalations	1.08	1.19		1.1		Britain—Cementation Co. Ltd. (1962)
Limestone	0.65					Switzerland—Muller (1966)
Poor limestone	0.32					France—Hennequin & Cambefort (1966)
Massive limestone	0.39-0.78					France—Hennequin & Cambefort (1966)
Karstic limestone	0.54					France—Hennequin & Cambefort (1966)
Tertiary limestone	1.00		2.83		2.8	Switzerland—Losinger & Co. Ltd (1966)
Limestone			4.55-4.80			Switzerland—Ruttner (1966)
Marly limestone	0.03-0.07 (average) 0.21-0.36 (measured)					Italy—Berardi (1967)
Limestone			0.27			USA—Saliman & Schaefer (1968)
Limestone	0.28					Italy—Berardi (1969)
Dolomitic limestone			1.80			Canada—Brown (1970)
Marly limestone	0.39-0.94					Italy—Berardi (1972)
Limestone	0.26					Italy—Berardi (1972)
Limestone/ puddingstone	0.44					France—Soletanche Co Ltd (1968)
Limestone	1.18	1.42		1.2		USA—Buro (1972)
Chalk			0.70			Britain—Associated Tunnelling Co Ltd. (1973)
Dolomite		1.66				Canada—Golder Brawner (1973)
Dolomitic siltstone	0.43					USA—White (1973)
Limestone & marly bands	0.37	0.55		1.5		Italy—Mongilardi (1972)
Arenaceous sediments						
Sandstone	1.44	1.58		1.1		Britain—Morris & Garrett (1956)
Hard sandstone	1.42	1.56		1.1		Britain—Cementation Co. Ltd. (1962)
Bunter sandstone	0.95	0.98		1.03		Britain—Cementation Co. Ltd. (1962)
Sandstone	0.76	0.84		1.1		Britain—Cementation Co. Ltd. (1962)
Sandstone	0.74					Czechoslovakia—Hobst (1968)
Sandstone	0.31	0.40	1.73	1.29	5.6	USA—Drossel (1970)
Sandstone	0.80					USA—Thompson (1970)
Poor sandstone	0.40					Germany—Brunner (1970)
Good sandstone	1.14					Germany—Brunner (1970)
Sandstone & Breccia	0.38					France—Soletanche (1968)
Sandstone		0.95				Australia—Williams et al (1972)
Bunter sandstone	0.60	1.20		2.0		Britain—Littlejohn (1973)
Sandstone	1.17					Australia—McLeod & Hoadley (1974)
Argillaceous sediments						
Shale	0.62					Canada—Juergens (1965)
Marl	0.10	0.28		2.8		Italy—Berardi (1967)
Shale	0.30		0.63		2.1	Canada—Hanna & Seaton (1967)
Very weathered shale			0.39			USA—Saliman & Schaefer (1968)
Shale	0.13-0.24					USA—Koziakin (1970)
Grey siltstone	0.62					Britain—Universal Anchorage Co. Ltd. (1972)
Clay marl	0.14-0.24	0.21-0.36		1.5		Germany—Schwarz (1972)
Shale	0.62					Canada—McRosite et al (1972)
Argillite	0.82					Canada—Golder Brawner (1973)
Mudstone	0.63	0.88		1.4		Australia—McLeod & Hoadley (1974)
Miscellaneous						
Bedded sandstone & shale	0.25-0.50					Italy—Beomonte (1961)
Porous, sound goassamer	1.57	1.72		1.1		Britain—Cementation Co. Ltd. (1962)
Shale & sandstone	0.07	0.10		1.5		USA—Reti (1964)
Soft rocks	0.75					Sweden—Nordin (1966)
Sandstone & shale	1.82					Poland—Bujak et al (1967)
Siltstone & mudstone	1.65					Australia—Maddox et al (1967)
Fractured rock (75 per cent shale)						
Poor			0.24			USA—Saliman & Schaefer (1968)
Average			0.35			USA—Saliman & Schaefer (1968)
Good			0.75			USA—Saliman & Schaefer (1968)
Limestone & caly breccia	0.20-0.23					Italy—Berardi (1972)

a major factor which affects not only the ultimate bond but also the load-deflection characteristics. Fig. 6 shows the results produced by test anchors in rhyolite tuff, of both sound and weathered varieties. No data are provided on grout or rock strengths but it is significant that the equivalent uniform bond stress at maximum jack capacity is scarcely 0.1N/mm². 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:

$$\tau_{ult} = 0.007N + 0.12 \text{ (N/mm}^2\text{)}$$
where N = number of blows per 0.3m ....(2)

Similarly, Littlejohn (1970) illustrates a correlation between N and ultimate bond for stiff/hard chalk, as follows:

$$\tau_{ult} = 0.01N \text{ (N/mm}^2\text{)} \quad \text{.........}(3)$$

In grades III, II and I of chalk, he observed a range of τ_{ult} of 0.21–1.07N/mm² based on test anchors pulled to failure.

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

Table IV contains data abstracted from reports of rock anchor contracts throughout the world. In addition to the working, test, and ultimate bond values, the measured and designed safety factors are provided where available. In certain cases, the fixed anchor diameter has been inferred, to facilitate analysis of the data, as published.

It will be noted that, even for one rock type, the magnitude of bond used in practice is extremely variable. There are many reasons for this, the most important of which are:

(*i*) Different designers use different bond values and safety factors, which may be related to type of anchor and extent of the anchor testing programme.

(*ii*) "Standard" values for a certain rock type have often been modified to reflect local peculiarities or irregularities of the geology.

(*iii*) Factors related to the construction techniques e.g. drilling method, flushing procedure, and grout pressure will influence the results obtained. (The effect of these aspects will be discussed in Part 2—Construction.)

On the whole however, it would appear that the bond values employed are to a degree consistent with rock type and competency.

Fixed anchor dimensions

The recommendations made by various engineers with respect to length of fixed anchor are presented in Table V. Under certain conditions it is recognised that much shorter lengths would suffice, even after the application of a generous factor of safety. However, for a very short anchor the effect of any sudden drop in rock quality along the anchorage zone, and/or constructional errors or inefficiencies, could induce a serious decrease in that anchor's capacity.

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

(*i*) Type and size of tendon,

(*ii*) The relation of diameter to perimeter area of fixed anchor and hence the anchor capacity, assuming uniform bond,

(*iii*) Ratio of steel area to cross-sectional area of borehole for efficient bond distribution and corrosion protection,

(*iv*) Drilling method and rig to be used, and

(*v*) Nature of rock in the anchor 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 trend indicated in Table V1.

TABLE V—FIXED ANCHOR LENGTHS FOR CEMENT GROUTED ROCK ANCHORS WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Fixed anchor length (metres)		*Source*
Minimum	*Range*	
3.0		Sweden—Nordin (1966)
3.0		Italy—Berardi (1967)
	4.0- 6.5	Canada—Hanna & Seaton (1967)
3.0	3.0-10.0	Britain—Littlejohn (1972)
	3.0-10.0	France—Fenoux et al (1972)
	3.0- 8.0	Italy—Conti (1972)
4.0 (very hard rock)		South Africa—Code of Practice (1972)
6.0 (soft rock)		South Africa—Code of Practice (1972)
5.0		France—Bureau Securitas (1972)
5.0		USA—White (1973)
3.0	3.0- 6.0	Germany—Stocker (1973)
3.0		Italy—Mascardi (1973)
3.0		Britain—Universal Anchorage Co. Ltd. (1972)
3.0		Britain—Ground Anchors Ltd. (1974)
3.5 (chalk)		Britain—Associated Tunnelling Co. Ltd. (1973)

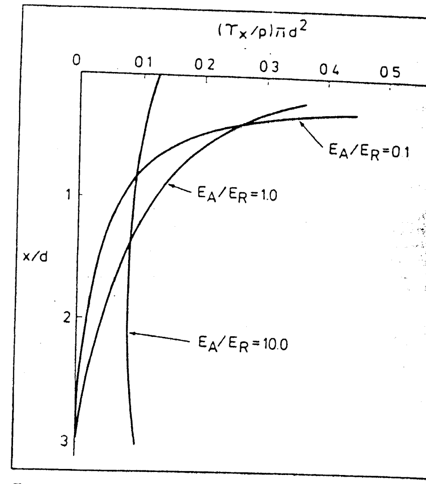


Fig. 8. Variation of shear stress with depth along the rock/grout interface of an anchor

(after Coates and Yu, 1970)

TABLE VI—APPROXIMATE RELATIONSHIP BETWEEN FIXED ANCHOR DIAMETER AND WORKING CAPACITY

Capacity, kN	*Diameter, mm*
200— 1 200	50—100
1 000— 3 000	90—150
3 000— 4 500	150—200
4 500—14 000	200—400

The third and fourth considerations will be dealt with in Part 2—Construction, but it is noteworthy that 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 12mm. This approach has also been discussed by FIP (1972) who recommend a grout cover to the tendon of 5mm, and 5-10mm for temporary and permanent rock anchors, respectively.

With regard to the amount of steel which should 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 per cent 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. Figs. 7a and 7b show the typical anchor geometry and the model employed to calculate approximately the stress induced by an anchor loaded either in tension or compression. The authors show that the shear stress (i.e. bond) distribution, is dependent on the ratio of the elastic moduli of the anchor material (E_a) and the rock (E_r). Fig. 8 shows the variation of the shear stress along the interface of an anchor of length equal to six 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 end of the anchor; higher values of the ratio are associated with more even stress distributions. It is also apparent that for $E_a/E_r > 10$, 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.

For anchors subjected to tensile loading the shear stresses in turn induce tensile stresses in the rock, which reach a maximum value at the proximal end of the anchorage. Fig. 9 illustrates the rapid dissipation of the tensile stresses radially at the distal end of the fixed anchor. For a 1 500kN capacity anchor in a 75mm diameter hole, the maximum tensile stress is estimated to be about 145N/mm² at the proximal end of the fixed anchor in rock, whilst at the opposite end, this stress is 48N/mm², provided, of course, that the rock can sustain these stresses. It seems probable that cracking will occur, and the magnitude of the maximum tensile stress decrease, as it transmits radially outwards, reaching an equilibrium position if the rock remains in position. The propagation of such cracks due to large tensile stresses acting parallel to the anchor axis possibly accounts in part for the anchor creep frequently observed to occur for a period of time after stressing. Deformation measurements adjacent to such anchors would provide useful information in this respect.

With regard to the magnitude of E_a, Phillips (1970) quotes a value of 2.1×10^4 N/mm² for a neat grout of water/cement ratio 0.4 and Boyne (1972), using a 0.35 water/cement ratio expansion grout, obtained a value of 1.0×10^4N/mm². Therefore, before the uniform bond distribution can be assumed, the rock must have an elastic modulus in the range $0.1\text{-}0.2 \times 10^4$ N/mm². Using a statistical relationship derived by Judd and Huber (1961), which relates rock compressive strength to elastic modulus:

$$UCS = \frac{E}{350} \qquad (4)$$

Phillips estimates therefore that the compressive strength of the rock in this case should be significantly less than 6N/mm².

However, 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. 8, the bond distribution is markedly non-uniform. Indeed, for anchors in these rocks of compressive strength in excess of 6N/mm², 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 large 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

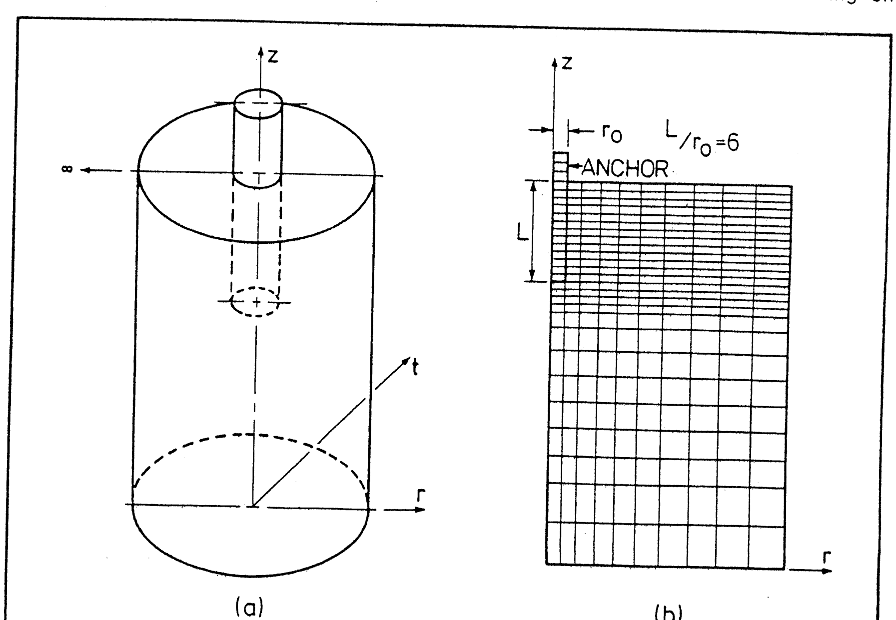


Fig. 7. The geometry of the rock anchor studies: (a) definition of axes; (b) finite element model

(after Coates and Yu, 1970)

stress distribution. In this context Phillips (1970) suggests three possible approaches:

1. Following debonding, the restraint imposed by the rock on the uneven rock-grout interface causes dilation. Additional anchorage movement is only possible through further shear failure of the grout, giving a possible stress distribution as shown in Figs. 10a and 10b.
2. The residual bond stress, when considered alone, and ignoring dilation, will depend on the magnitude of "ground pressure" acting normal to the interface. This will probably vary over the debonded length and it may be less than the grout shear strength (Fig. 10c). If it is greater than the grout shear strength, the stress distribution will revert to that of Figs. 10a and 10b.
3. It is probable that the stress distribution will vary with applied load possibly as shown in Figs. 10d, 10e and 10f. This presumes an initial stress distribution similar to the theoretical stress distribution (Fig. 10b). At large loads, virtually the whole of the anchor is debonded and the stress is distributed according to the amount of relative movement and the degree of dilation or frictional shear strength mobilised (see Fig. 10f).

It should be emphasised however that these approaches are hypothetical and experimental work is required to confirm their validity in relation to rock anchor design.

Experimental evidence

In Italy much valuable experimental research has been conducted, principally by Berardi, into the distribution of stresses both along the fixed anchor and into the rock. In 1967 he reported on tests to determine the distribution of fixed anchor stresses and concluded 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. 11a and 11b are typical diagrams which illustrate the uneven bond distribution as calculated from strain gauge data. Both anchors were installed in 120mm diameter boreholes in marly limestone ($E = 3 \times 10^4 kN/m^2$; UCS = $100N/mm^2$ approximately). Other results show that the bond distributions are more uniform for high values of E_{grout}/E_{rock}, non-uniform for low values of this ratio i.e. for rock of high elastic modulus, thus confirming the predictions of Coates and Yu.

Muller (1966) produced interesting results in Switzerland on the distribution of shear stress along the 8m fixed anchor of a 220 tonne BBRV anchor (Fig. 12). From readings obtained during stressing, he concluded that the load was not uniformly distributed to the rock over the length of the fixed anchor. For example, at a load of 55 tonnes the force was transmitted uniformly over the proximal 5.55m, implying an average bond of $0.22N/mm^2$. At 185 tonnes however, load was recorded over the lower 4.1m of the tendon with apparent debonding of the tendon over the upper 3.9m. About 30 tonnes was resisted by the bottom of the anchor, but between points *A* and *C*, (Fig. 12) the average bond stress was about $0.98N/mm^2$. At 280 tonnes, a comparison of 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. The values

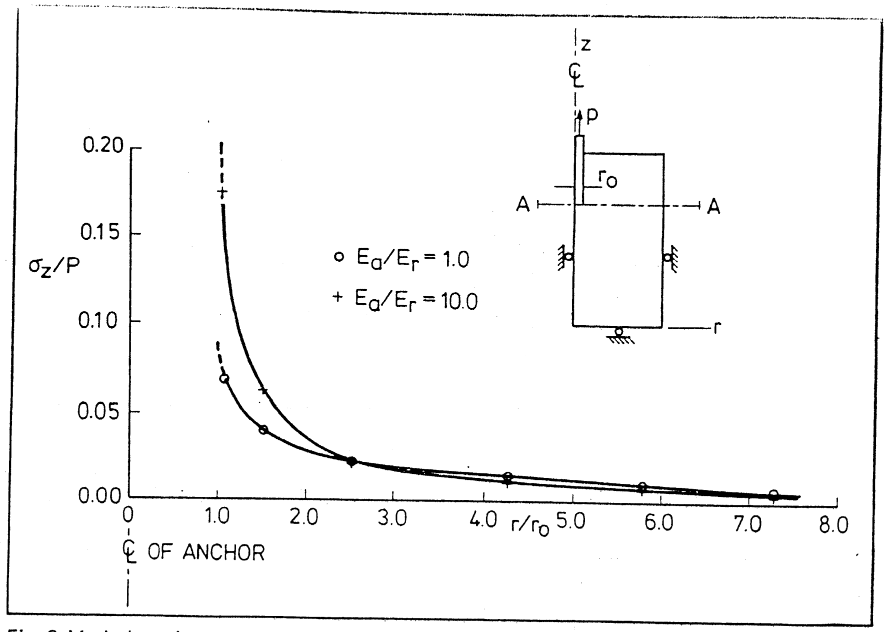


Fig. 9. Variation of tensile stress in the rock adjacent to the end of a tension anchor
(after Coates and Yu, 1970)

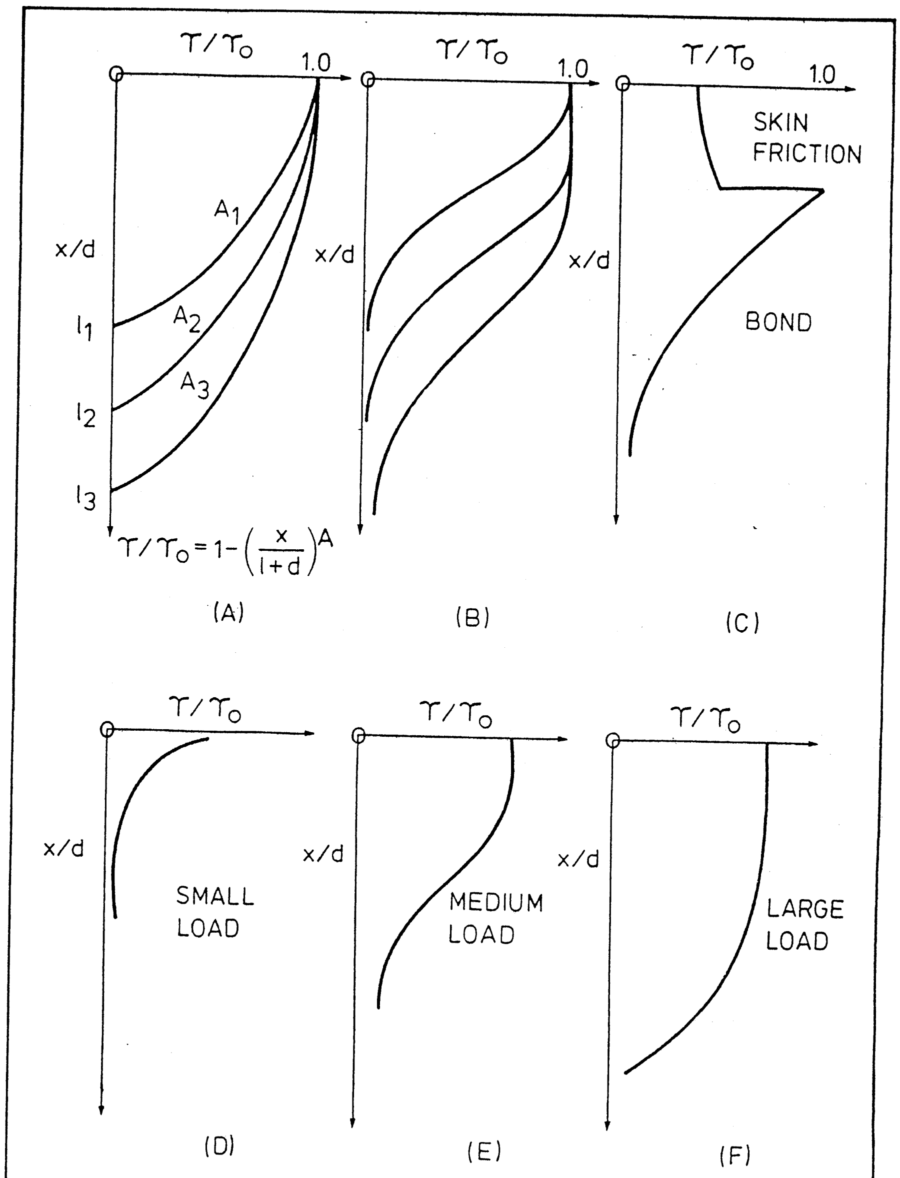


Fig. 10. Hypothetical stress distributions around a partly debonded anchorage
(after Phillips, 1970)

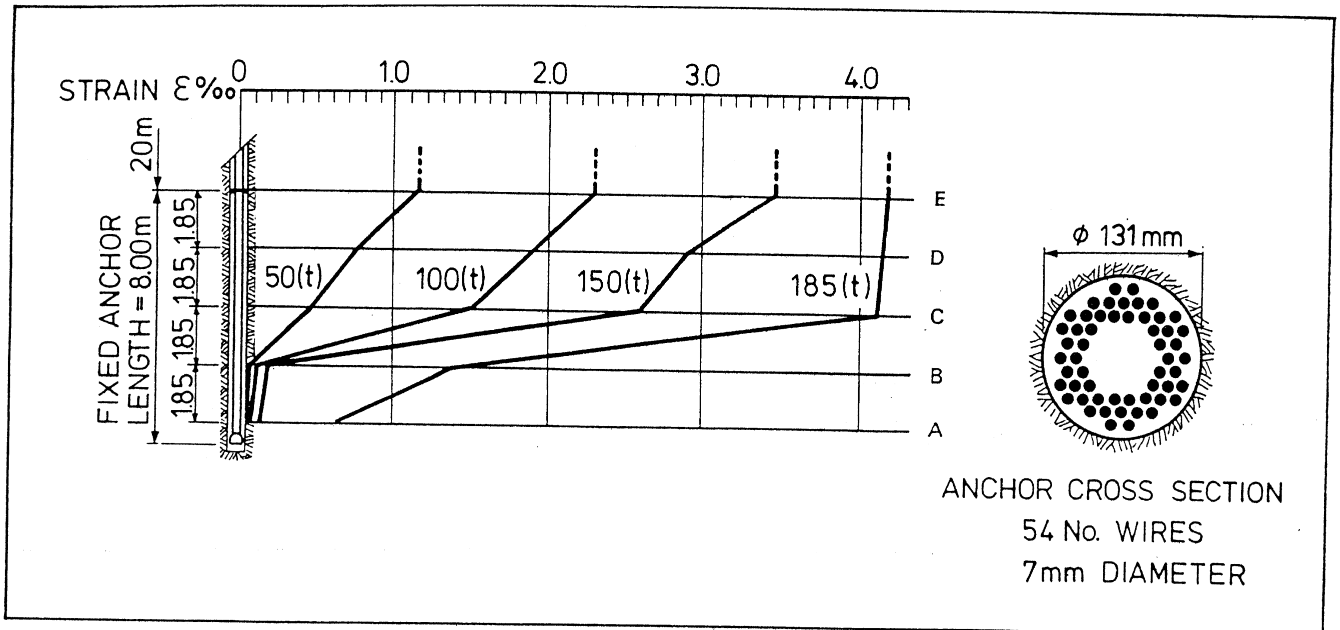


Fig. 12. Strain distribution along tendon in fixed anchor zone of a 220t capacity anchor

(after Muller, 1966)

for bond strength quoted above compare with an average value, based on uniform distribution, of about 0.65N/mm², which is well below both the actual value at 185 tonnes and the grout shear strength.

Decoupling, equivalent to an addition in free length of 2.2m, has also been reported by Eberhardt and Veltrop (1965), during the stressing of a 1 300t capacity test anchor installed in basalt (fixed anchor length = 11.5m, diameter = 406mm).

Remarks

From mathematical, laboratory and field evidence, the distribution of the bond mobilised at the rock/grout interface is unlikely to be uniform unless the rock is "soft". It appears that non-uniformity applies to most rocks where E_{grout}/E_{rock} is less than 10.

In the case of high capacity anchors evidence exists that partial debonding in the fixed anchor occurs, and that debonding progresses towards the end of the anchor as the load is increased. Information is scarce concerning the conditions where debonding is serious.

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 a wide range of rock masses whose engineering and geological properties can be fully classified, in order to ascertain which parameters dictate anchor performance. In this way it should be possible in due course to provide more reliable design criteria.

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

The prior knowledge of certain geological and geotechnical data pertaining to the rock is essential for the safe, economic design of the anchor and correct choice of construction method. The authors believe that the following geotechnical properties should be evaluated during the site investigation stage, in addition to the conventional descriptions of lithology and petrography: quantitative data on the nature, orientation, frequency and roughness of the major rock mass discontinuities; shear strength of these discontinuities; and compressive and shear strength of the rock material.

Also, particularly in the softer rocks, weatherability and durability should be assessed, especially on samples drawn from the level of prospective fixed anchor zones. It is realised that the determination of the modulus of elasticity is rather involved and expensive, particularly for rock masses. However, as the influence of this parameter on anchor performance has already been demonstrated, efforts should be made whenever possible, to obtain a realistic value.

The ground water regime is also of prime importance, especially the position of the water-table, and the groundwater rate of flow, pressures and aggressivity. It should be noted that the ratio of anchor length to discontinuity spacing determines the relative importance of intact material and rock mass properties in any one case. For example, where the fracture spacing is relatively large, the rock material properties will be the dominant controls of, for example, drillability and rock/grout bond. However, this is rarely the case, and the properties of the rock mass are usually crucial, particularly in the assessment of the overall stability of the anchor system.

The extent of the site investigation should be determined by the importance of the contract, and the potential difficulties and risks inherent in its execution. In situ anchor tests should be carried out wherever possible to clarify design proposals.

Bearing in mind that anchors are often installed at very close centres it would appear in site investigation that a "construction" stage is required where drill logs, penetration rates, grout consumptions and check pull-out tests are monitored in order to highlight "difficult" or changed rock conditions. These terms need to be defined in order to avoid legal problems and the question is important whenever doubt about anchor competence exists.

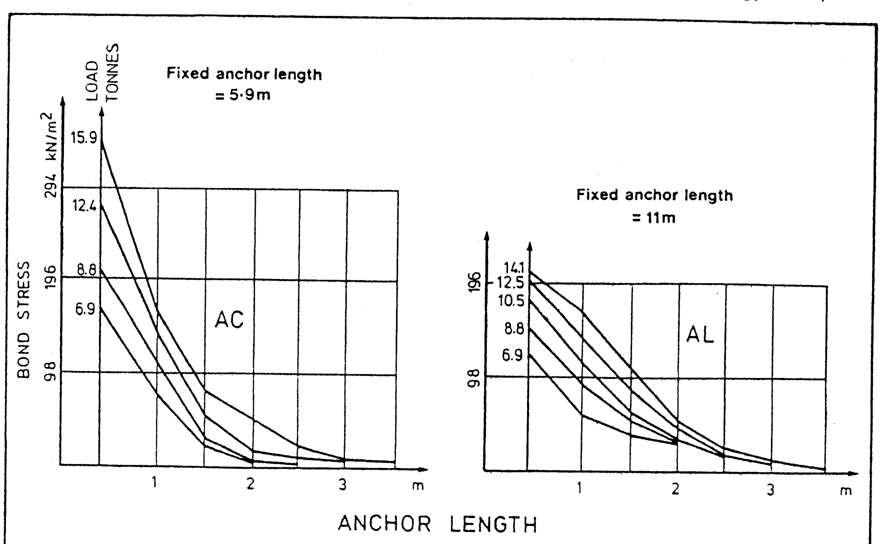


Fig. 11. Distribution of bond along fixed anchor length

(after Berardi, 1967)

BOND BETWEEN CEMENT GROUT AND STEEL TENDON

Introduction

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.

However, as has been demonstrated in the section dealing with rock/grout bond§, little standardisation or uniformity of approach is apparent related to the grout/tendon bond, and the rather simple design assumptions commonly made are in contradiction to certain experimental observations.

In this section, the mechanisms of bond are discussed and anchor design procedures employed in practice are reviewed. Bearing in mind the scarcity of information pertaining to anchors, data abstracted largely from the fields of reinforced and prestressed concrete are also presented, which relate to the magnitude and distribution of bond.

The mechanisms of bond

It is widely accepted that there are three mechanisms:

1. *Adhesion.* This provides the initial "bond" before slip, and arises mainly from the physical interlocking (i.e. gluing) of the microscopically rough steel and the surrounding grout (Fig. 13). Molecular attrac-

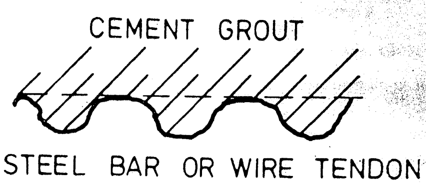


Fig. 13. Magnified view of interface between grout and steel

tion is also thought to act. Adhesion is considered to disappear when slip comparable with the size of the micro indentations on the steel occurs.

2. *Friction.* This component depends on the confining pressure, the surface characteristics of the steel, and the amount of slip, but is largely independent of the magnitude of the tendon stress. The phenomena of dilatancy and wedge action also contribute to this frictional resistance as radial strains are mobilised where the longitudinal strain changes.

3. *Mechanical interlock.* This is similar to micro mechanical locking, but on a much larger scale, as the shear strength of the grout is mobilised against major tendon irregularities, e.g. ribs, twists.

An idealised representation of these three major bond components is shown in Fig. 14. For short embedment lengths the adhesive component is most important, but

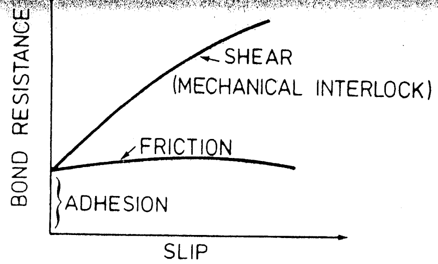


Fig. 14. Idealised representation of major components of bond

for longer lengths, all three may operate—adhesion failure occurring initially at the proximal end and then moving progressively distally to be replaced by friction and/or mechanical interlock. Frictional and interlocking resistances increase with lateral compression and decrease with lateral tension. Clearly, the grout shear strength and the nature of the tendon surface, both micro-and macroscopically, are major factors in determining bond characteristics.

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 or concrete.

In Britain, the following general recommendations may be followed, based on CP 110, 1972 and information supplied by Bridon Wire Ltd (1968).

Wire

(*i*) For a bright or rusted, plain or indented wire with a small off-set crimp e.g. 0.3mm off-set, 40mm pitch, a transmission length of 100 diameters may be assumed when the cube strength of the concrete or grout at transfer is not less than 35N/mm².

(*ii*) For a wire of a considerable crimp e.g. 1.0mm off-set, 40mm pitch, a bond length of 65 diameters may be assumed for the above conditions.

(*iii*) Galvanised wire provides a poor bond, less than half that of comparable plain wire.

(*iv*) It may be assumed that 80 per cent of the maximum stress is developed in a length of 70 diameters for the conditions mentioned in (*i*) and in a length of 54 diameters for the conditions mentioned in (*ii*).

Strand

(*i*) From the available experimental data, the transmission length for small diameter ordinary strand is not proportional to the diameter of the tendon. Table VII gives values of transmission length for strand working at an initial stress of 70 per cent ultimate in concrete of strength 34.5-48.3 N/mm².

TABLE VII—TRANSMISSION LENGTHS FOR SMALL DIAMETER STRAND

Diameter of strand (mm)	*Transmission length (mm)*	*(diameters)*
9.3	200 ($\pm$25)	19-24
12.5	330 ($\pm$25)	25-28
18.0	500 ($\pm$50)	25-31

N.B.—Range of results given in brackets.

(*ii*) Tests in concrete of strength 41.4-48.3N/mm² with Dyform compact strand at 70 per cent ultimate show an average transmission length of 30-36 diameters.

According to the results of an FIP questionnaire (1974) national specifications vary considerably for transmission lengths, the most optimistic being those of the United Kingdom. It is accepted that compact strand e.g. Dyform, has transmission lengths 25 per cent greater than those for normal 7-wire strand, and that sudden release of load also increases the transmission length. (An additional 25 per cent is recommended in Rumania).

Bar

(*i*) With regard to permissible bond stresses for single plain and deformed bars in concrete, Table VIII illustrates the values stipulated by the British Code for different grades of concrete. These values are applied to neat cement grouts on occasions.

(*ii*) For a group of bars, the effective perimeter of the individual bars is multiplied by the reduction factors below

No. of bars in group	*Reduction factor*
2	0.8
3	0.6
4	0.4

It is important to note that no information is provided in the Code on group geometry e.g. minimum spacing, where the reduction factors should be employed. In addition no guidance of any kind is given for groups of strands or wires.

With reference to minimum embedment lengths, Morris and Garret (1956) have calculated from stressing tests on 5mm diameter wires that the minimum necessary embedment is just over 1m. Golder Brawner Assocs. (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 0.6m. Results from Freyssinet anchors with spacers have shown that each strand can withstand about 156-178kN with 0.6m embedment. Since the capacity of such strand is usually in the range 178-270kN, Golder Brawner Assocs. conclude that no strand of a rock anchor logically needs an embedment length in excess of 1.5m. However, for other reasons, a length of 3m is usually considered the minimum acceptable.

Data abstracted from papers describing rock anchor contracts is presented in

TABLE VIII—ULTIMATE ANCHORAGE BOND STRESSES

Type of bar	Characteristic strength of concrete (f_{cu}, N/mm²) 20	25	30	40+
	Maximum bond stress, N/mm²			
Plain	1.2	1.4	1.5	1.9
Deformed	1.7	1.9	2.2	2.6

Tables IX, X and XI for bar, wire and strand, respectively. In all the calculations, except where otherwise noted, the bond is assumed uniform over the whole tendon embedment zone, which is taken as equal to the length of the fixed anchor.

Bearing in mind the relatively small number of values, comments are limited to the following:

(*i*) There would appear to be a greater degree of uniformity on values chosen for the working bond between strand and grout, than for the bond developed by bars and wires with grout. The value of the bond (up to 0.88N/mm²) for 15.2mm strand is slightly higher overall than that for 12.7mm strand (up to 0.72N/mm²), and in both cases there is a trend towards a reduction of the bond with an increase in number of strands.

(*ii*) The actual safety factor against failure of the grout/tendon bond is usually well in excess of 2.

(*iii*) The average bond developed by bars, especially deformed types, is on average significantly higher than that developed by strands or wires. Also the presence of deformities increases the bond magnitude by up to 2 times with respect to plain bars.

Distribution of bond

Much of the work to investigate the distribution of bond along grout/steel interfaces has been carried out in the United States in connection with prestressed and reinforced concrete. Gilkey, Chamberlin

TABLE IX—GROUT/BAR BOND VALUES WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Bar tendon	*Embedment (m)*	*Load (kN)*	*Working load (N/mm²)*	*Test bond (N/mm²)*	*Ultimate bond (N/mm²)*	*Remarks*	*Source*
Plain			1.2-1.9			Design criteria: bond dependent on concrete	Britain—CP110(1972)
Deformed			1.7-2.6				
Square twist			5.25			Design criteria	Britain—Roberts (1970)
Ribbed			7.0				
Plain					1.38	Short embedment test	Canada—Brown (1970)
Plain and threaded end					2.62	Bond dependent on embedment and grout tensile stress	
Deformed bar	30d					Design criteria: "solid" rock	Canada—Ontario Hydro (1972)
Deformed bar	40d+					Design criteria: "seamy" rock	
20 No. 20mm dia plain	2.5	1750		0.56		Test anchor	Italy—Berardi (1960)
20mm dia ribbed and threaded with end nut	2.2			1.1		Test anchor	Italy—Beomonte (1961)
	3.9			0.6		Test anchor	
	2.2			1.2		Test anchor	
	2.2			0.9		Test anchor	
25mm dia deformed bar	0.2				2.7	Test anchor	Canada—Brown (1970)
	0.4				5.0	Test anchor, Bond for deformed bar=5 x bond for plain bar	
	0.6						
25.4mm dia square	1.83	289		1.98		Test anchor	USA—Salisman & Schaefer (1968)
25.4mm dia plain	0.06	52			10.1	Tests: for each pair the first test conducted at 28 days. and the second at 90 days	Australia—Pender et al (1963)
	0.06	53.5			11.2		
	0.12	52			5.5		
	0.12	67			7.0		
	0.18	63			4.4		
	0.18	117			8.1		
	0.36	139			4.9		
	0.36	148			5.1		
28mm dia plain	5.9	160		0.31		Test anchor: bond known to be much higher locally	Italy—Berardi (1967)
	11.0	160		0.16			
28 mm dia plain		400	0.76			Design criteria	Switzerland—Comte (1971)
28.6mm dia plain	0.91	220		2.72		Test anchor	USA—Drossel (1970)
30mm dia plain		320	3.0	7.2		Test at bar UTS	Switzerland—Muller (1966)
31.8mm dia high tensile	1.83	605	3.3			Commercial anchor	USA—Wosser et al (1970)
31.8mm dia and thread	1.2	700		5.74		Test anchor	USA—Drossel (1970)
31.8mm dia Dywidag & locknut	8.5	545	0.64			Commercial anchor	USA—Oosterbaan et al (1972)
35mm dia mild steel	6.1	360	0.54			Anchor pile	Canada—Jaspar et al (1969)
35mm dia plain	6.1	505		0.75		Test anchor	Canada—Barron et al (1971)
35mm dia plain	6	700	1.06			Commercial anchor	USA—Feld et al (1974)
43mm dia mild steel	12.2	610	0.37			Anchor pile	Canada—Jaspar et al (1969)
44mm dia plain	0.35				4.7	Test anchor	Canada—Brown (1970)
	0.6						

TABLE X—GROUT/WIRE BOND VALUES WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Wire tendon	*Embedment (m)*	*Load (kN)*	*Working bond (N/mm²)*	*Test bond (N/mm²)*	*Ultimate bond (N/mm²)*	*Remarks*	*Source*
Plain	100d					Design criteria	Britain—CP110 (1972) also CP115 (1969)
Crimped	65d						
Groups of 5, 6, 7mm			0.5-1.0			Design criteria	Switzerland—Comte (1971)
e.g. from 14 No. 5mm	1.7-3.4	350					
to 54 No. 7mm	6-12	2470					
Plain/Crimped			1.035			Design criteria	Australia—Standard CA 35 (1973)
12 No. 5mm	1.8	280	0.53			Commercial anchor	Switzerland—Pliskin (1965)
24 No. 5mm	4.0	500	0.33			Commercial anchor	Poland—Bujak et al (1967)
37 No. 5mm	1.0	1540			2.62	Test anchor	Britain—Morris et al (1956)
	2.44	700	0.49			Commercial anchor	
	2.44	950	0.67			Commercial anchor	
40 No. 5mm	5.0	850	0.27			Commercial anchor	Switzerland—Birkenmaier (1953)
102 No. 5mm	4.0	2000	0.32			Commercial anchor	India—Rao (1964)
24 No. 6.4mm	18.3	670	0.076			Commercial anchor	USA—Reti 1964)
90 No. 6.4mm HT	9.14	3750	0.23			Commercial anchor	USA—Thompson (1970)
90 No. 6.4mm	9.14	11570	0.70			Commercial anchor	Canada—Golder Brawner (1973)
12 No. 7mm	2.5	515	0.78			Commercial anchor	Switzerland—Pliskin (1965)
12 No. 7mm	7.5	450	0.23			Commercial anchor	Brazil—da Costa Nunes (1969)
12 No. 7mm	2.5	600	0.93			Commercial anchor	Africa—Anon (1970)
12 No. 7mm	4.0	500	0.47			Commercial anchor	Italy—Berardi (1972)
34 No. 7mm	7.6	1300	0.23			Commercial anchor	Australia—Rawlings (1968)
35 No. 7mm	3.5	1380	0.51			Commercial anchor	Switzerland—Ruttner (1966)
	0.6				2.26	Test anchor	
	1.5				2.0	Test anchor	
72 No. 7mm galvanised HT	5.2	2740	0.33			Commercial anchor	Australia—Maddox et al (1967)
33 No. 7.62mm	3.5	1700	0.62			Commercial anchor	Germany—Anon (1972)
12 No. 8mm	2.9	675	0.77			Commercial anchor	Switzerland—Pliskin (1965)
12 No. 8mm					0.47	Test anchor	Britain—Bundred (1973)
18 No. 8mm		1450		0.7		Test anchor:	Switzerland—Walther (1959)
27 No. 8mm		2250		0.7		at wire UTS	
24 No. 8mm	4.5	1725		0.46		Test anchor	Switzerland—Möschler et al (1972)
33 No. 8mm	3	1255		0.69		Test anchor	
4 No. 15.2mm	6.5	500	0.47			Commercial anchor	Italy—Berardi (1972)
6 No. 15.2mm	8.4	870	0.36			Commercial anchor	
9 No. 13-16mm	3.5	700	0.54			Commercial anchor	Germany—Anon (1972)

and Beal (1940) discuss in general terms the bond characteristics of bars during pull-out. 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. The total resistance continues to increase primarily because the length of the tendon which has passed its maximum resistance does not release entirely but exerts a residual resistance or drag acting concurrently with the adhesive bond in the region of maximum bond stress. Fig. 15 is an ideal-

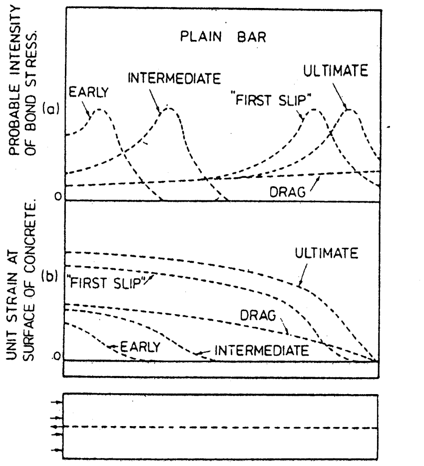


Fig. 15. Qualitative variation of (a) bond stress, (b) total tensile stress, during a pull-out test
(after Gilkey, Chamberlain & Beal, 1940)

ised diagram showing the progressive nature of bond distribution at successive stages of a test. Curves *(a)* represent intensities of bond stress between the bar and concrete. Curves *(b)* may be considered as stresses in bar, at successive points along the specimen. It should be recognised that for curves *(b)*, the intensity of bond stress at any point (rate of change of stress in the bar) is represented by the slope of the curve, with respect to the axis of the specimen, at that point. Bond is what makes stress transfer possible and can be present only in a region of changing stress in the steel or the concrete.

Considering Fig. 15, it is apparent that for a plain bar pull-out test:

(i) Bond resistance is first developed near the proximal end of the bar, and only as slight slip occurs are tensions and bond stresses transmitted progressively distally.

(ii) The region of maximum intensity of bond stress moves away from the proximal end as the pull increases. Between the proximal end and the region of maximum bond stress there is a fairly uniform frictional or drag resistance of greatly reduced intensity.

(iii) "First slip" occurs only after the maximum intensity of bond resistance has travelled nearly the full length of the specimen and has approached the distal end of the bar.

(iv) After appreciable slip, the primary adhesive resistance disappears and the bar offers a frictional or drag resistance throughout its entire length, amounting to perhaps half the ultimate total resistance attained.

In Britain, Hawkes and Evans (1951) were able to conclude from pull-out tests that the distribution of bond obeys an exponential law of the form:

TABLE XI—GROUT/STRAND BOND VALUES WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Strand tendon	*Embedment (m)*	*Load (kN)*	*Working bond (N/mm^2)*	*Test bond (N/mm^2)*	*Ultimate bond (N/mm^2)*	*Remarks*	*Source*
Any type			<2.1			Design criteria	Australia—Standard CA35 (1973)
Any type					3.105	Design criteria	USA—PCI (1974)
4 No. 12.7mm	4.3	495		0.72		Test anchor	Switzerland—Sommer et al (1974)
	1.8	495		1.71		Test anchor	
8 No. 12.7mm	5.0	1020	0.64			Commercial anchor	USA—White (1973)
8 No. 12.7mm	6.0	1120	0.59			Commercial anchor	USA—Feld et al (1974)
8 No. 12.7mm	3.6	1350	0.71			Commercial anchor	
9 No. 12.7mm	5.2	860	0.46			Commercial anchor	Canada—Hanna et al (1967)
12 No. 12.7mm	4.5	1410	0.65			Commercial anchor	Switzerland—Pliskin (1965)
12 No. 12.7mm	6.1	1200	0.41			Commercial anchor	Canada—Juergens (1965)
12 No. 12.7mm	5.2	1565	0.63			Commercial anchor	Canada—Hanna et al (1967)
12 No. 12.7mm	6.1	1335		0.46		Test anchor	Canada—Barron et al (1971)
12 No. 12.7mm	6.5	1360	0.44			Commercial anchor	Canada—McRostie et al (1972)
16 No. 12.7mm	6.1	1760	0.45			Commercial anchor	Canada—Golder Brawner (1973)
54 No. 12.7mm	11.3	7010	0.29			Commercial anchor	USA—PCI (1974)
N.B.—In the following, no distinction is made between "Normal" (15.4mm) and "Dyform" (15.2mm) strand. All results are calculated using the smaller diameter.							
4 No. 15.2mm	3.0	500	0.88			Commercial anchor	Britain—Universal Anchorage Co. Ltd. (1972)
6 No. 15.2mm	7.3	520	0.25			Commercial anchor	USA—Chen et al (1974)
8 No. 15.2mm	6.1	1500		0.64		Test anchor	USA—Nicholson Anchorage Co. Ltd. (1973)
10 No. 15.2mm	6.7	2150	0.67			Commercial anchor	Australia—Williams (1972)
10 No. 15.2mm	6.1	1900	0.65			Commercial anchor	Australia—McLeod et al (1974)
12 No. 15.2mm	8	2000	0.44			Commercial anchor	Britain—Littlejohn et al (1974)
13 No. 15.2mm	3	3000		1.61		Test anchor	
12 No. 15.2mm	6.5	1950	0.52			Commercial anchor	Switzerland—Pliskin (1965)
18 No. 15.2mm	7.6	4330	0.66			Commercial anchor	Canada—Golder Brawner (1973)
18 No. 15.2mm	7.6	2825	0.43			Commercial anchor	USA—Schousboe (1974)
18 No. 15.2mm	7.62	3770		0.58		Test anchor	
19 No. 15.2mm	15	3740	0.28			Commercial anchor	USA—Feld et al (1974)

$$\tau_x = \tau_0 e^{-\frac{Ax}{d}} \qquad \text{..........(5)}$$

where τ_x = bond stress at a distance x from the proximal end
τ_0 = 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 anchorage material

Assuming the applied tensile load, P, is equal to the sum of the total bond stress multiplied by the surface area of the tendon, Phillips (1970) extended the above theory as follows:

$$P = \int_0^L \pi d\, \tau_x . dx = \frac{\pi d^2\, \tau_0}{A}\left(1 - e^{-\frac{AL}{d}}\right) \qquad \text{..............(6)}$$

between the limits $x = o$ and $x = L$, where L is the length of the fixed anchor. The length of the anchor will depend upon the axial distance required to transfer the load across the interface (transmission length L_0).

At $x = L_0$, τ_x approaches 0 and from (5) it can be seen that Ax/d approaches infinity, giving

$$P = \frac{\pi d^2\, \tau_0}{A} \qquad \text{..............(7)}$$

Substituting equation (5) into equation (7) gives

$$\frac{\tau_x}{P}(\pi d^2) = A e^{-\frac{Ax}{d}} \qquad \text{..............(8)}$$

Equations (5) and (8) are represented graphically in Figs. 16 and 17 which show the variation of shear stress along the anchorage and its dependence upon the constant A. 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 anchor.

Although values for A have been obtained for steel anchorages embedded in concrete—Hawkes and Evans give $A = 0.28$—insufficient information exists at pre-

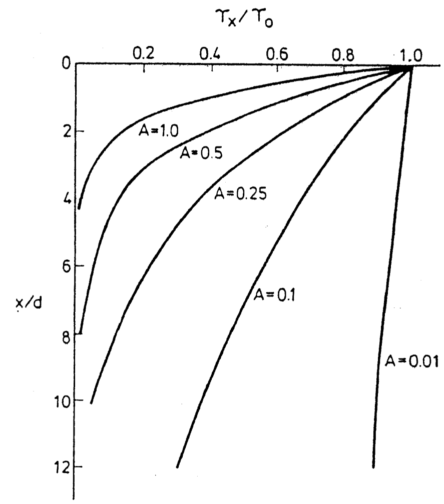


Fig. 16. Theoretical stress distribution along an anchor
(after Hawkes & Evans, 1951)

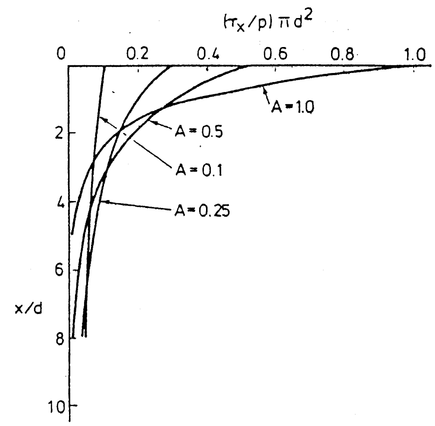


Fig. 17. Load distribution along an anchorage assuming AL/d is large
(after Phillips, 1970)

sent on the behaviour of cement grout anchors in rock to provide meaningful values for A. It is reassuring however, to find that the results in Fig. 17 are very similar to the results of Coates and Yu (1970) in Fig. 8 with E_a/E_r proportional to $1/A$, which suggests that the basic approach of Hawkes and Evans is applicable to rock anchors.

Magnitude of bond

Bars. In a rigorous investigation of the bond between concrete and steel bars, Gilkey, Chamberlin and Beal (1940) emphasise the following major points relevant to rock anchors:

1. Contrary to accepted belief, bond resistance is not proportional to the compressive strength of a standard cured concrete, there being some increase in bond but a reduction in the ratio of bond resistance to the ultimate compressive strength as the strength of concrete increases, especially for the higher strengths. To be specific, for the weaker concretes (UCS $< 21N/mm^2$) bond increases with the compressive strength. However, as the concrete strength exceeds this value, the increase in bond resistance becomes less, and within the strong concrete range i.e. UCS $> 42N/mm^2$, no added bond allowance is justified for added strength of concrete.
2. The bond developed by added length of embedment is not proportional to the additional length. The shorter the embedment, the greater is the average unit bond stress that can be developed by a plain bar. Therefore doubling the length of embedment as a means of increasing the anchorage does not actually double the amount of tension that the bar can resist by bond. On the other hand, additional embedment does add to the sum total of bond resistance.
3. Variations in age and type of curing seem to alter bond resistance much less than they alter the compressive strength of the concrete, bearing in mind that the strongest concrete gives the higher bond, but the weakest concretes have the highest ratio of bond to compressive strength.

Little information is available on the effect of spacing but Chamberlin (1953) conducted a series of tests with various types of bars to determine the effect of spacing on bond magnitude. For clear spacings of 1*d* and 3*d* differences in bond were not significant.

Wires and strand. Based on results obtained from almost 500 pull-out tests, Stocker and Sozen (1964) conclude:

(*a*) Due to the helical arrangement of the exterior wires, strand rotates while slipping through a grout channel, but the increase in bond is not significant. (Anderson *et al* (1964) also observed rotation of strand of about 15 deg during pull-out tests.)

(*b*) Bond magnitude increases by approximately 10 per cent per $6.9N/mm^2$ of concrete compressive strength, in the range $16.6\text{-}52.4N/mm^2$.

(*c*) Results from pull-out tests subjected to externally applied lateral pressures ranging from $0\text{-}17.25kN/m^2$ indicate a linear increase of bond strength with lateral pressure. In connection with this, concrete shrinkage is clearly important.

Effect of rust on bond

Gilkey *et al* (1940) also investigated the effect of steel surface conditions on bonding properties and found that:

(*i*) Deep flakey rust on bars, following 6-8 months exposure, lowers the bond, but wiping the loosest rust off finally produces a surface that will develop a bond equal to or greater than that which the bar would have developed in the unrusted condition.

(*ii*) Slightly rusted bars, following up to three months exposure, developed greater bond than unrusted or wiped rusted bars.

(*iii*) The loose powdery rust which appears on bars during the first few weeks of ordinary exposure has no significant effect on the bond properties of bars.

These findings have also been confirmed by Kemp *et al* (1968) for deformed bar, and Armstrong (1948), Base (1961) and Hanson (1969) for wire and strand.

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. Therefore any embedment length accommodating that interface automatically ensures a high factor of safety at the tendon/grout interface. A factor of safety of at least two is allowed by other designers.

While 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 bonded into the grout of an in situ rock anchor may be dissimilar to that of the tendon pull-out test used in concrete technology and from which most 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. In rock anchors, therefore, the mechanism of bond action depends on the respective elastic moduli of the steel and grout.

Little work has been done on multi-unit tendons with respect to their bond distribution. The use of spacers and centralisers, leading possibly to decoupling, also warrants investigation.

In general, recommendations pertaining to grout/tendon bond values used currently in practice for rock anchors commonly take no account of the length and type of tendon, tendon geometry, or grout strength, and for these reasons it is still advisable to measure experimentally the embedment length for known field conditions.

TENDON

Introduction

Accurate data on the mechanical properties of tendon components are readily available, but the choice of type of tendon and safety factors to be employed against rupture still demand assessment and judgement by the designer, especially in countries not covered by a code relating to anchors.

Tendons may be formed of bar, 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.

Largely as a result of recent developments in prestressing equipment and techniques, the use of strand appears to be increasing in popularity. A recent survey by FIP (1974) also confirms that strand tends to be more popular than wire, and the use of strand is now accepted even in countries where the basic material cost is greater. It is now widely recognised that the smaller the diameter of the tendon, the less is the cost of the material per unit of prestress force, but direct cost comparisons for the supply of tendon material in any country can be misleading since the real cost of the tendon also reflects cost of fabrication, installation and stressing.

Tendon characteristics

With regard to general characteristics it is of value to know that in Britain the production of prestressing tendons is governed by BS 4486:1969 (Cold Worked High Tensile Alloy Steel Bar), BS 2691: 1969 (Steel Wire), BS 3617:1971 (7 Wire Strand) and BS 4757:1971 (19 Wire Strand).

Following publication of CP 110:1972, permissible stresses are quoted in terms of the specified characteristic strength which is the guaranteed limit below which not more than 5 per cent test results fall, and none of these is less than 95 per cent characteristic strength. For wire and strand, the specified minimum strength is taken as the characteristic strength, which for practical purposes is termed 100 per cent fpu.

At home and abroad it is common to find tendon stresses quoted in such terms as elastic limit, 0.1 per cent proof stress and 0.2 per cent proof stress. Therefore to facilitate understanding and comparisons, some reconciliation is required between these terms and characteristic strength. In this connection it is noteworthy that in the preamble to the French Code (Bureau Securitas 1972) the term *Tg* is identified and defined as the elastic limit, measured as the 0.1 per cent proof stress, i.e. that point at which the permanent elongation reaches this value. The same note draws attention to the fact that this limit should not be confused with the 0.2 per cent proof stress adopted in the British Codes. Based on the advice of wire metallurgists the authors understand that the 0.1 per cent proof stress varies from 3-5 per cent below the 0.2 per cent proof stress which is defined as 87 per cent fpu in CP 110. Taking the average figure of 4 per cent below 0.2 per cent proof stress, then a 0.1 per cent proof stress is equivalent to 83.5 per cent fpu. This correlation may be employed when comparing safety factors in subsequent tables.

With respect to the values of elastic modulus quoted subsequently, it is known that an error of 5 per cent is possible, although the majority of results are within three standard deviations from the mean. Knowledge of this possible variation can be very important when interpreting load-extension graphs and for the same reason relaxation characteristics of tendons should be assessed carefully. Both aspects are detailed in Part 3 of this review, but it is of general interest to know that relaxation loss is a function of the logarithm of time. For example, the loss after one hour is

TABLE XII—TECHNICAL DETAILS OF BRITISH PRESTRESSING BARS

Item	*Unit*	*Bar diameter (mm)*									*Remarks*
		20*	22	25*	28	32*	35	40*	4x32	4x40	
Sectional area	mm^2	314.2	380.1	490.9	615.8	804.3	962.1	1256.6	3217	5026	In each case, the characteristic tensile strength is $1000N/mm^2$
Minimum breaking load	kN	325	375	500	625	800	950	1250	3200	5000	

*Recommended sizes

50-60 per cent of that at 100 hours, which in turn is about 80 per cent of that at 1000 hours. The loss at 1000 hours is also about half that at 5-8 years. Relaxation loss depends on the initial stress in the tendon and production history, and whilst tendons of exceptionally low relaxation properties can be produced, the anchor designer should remember that little advantage will be gained through their use, if for example creep in the ground is likely to be large in comparison.

1. *Bars*. CP 110 (1972) quotes detail supplied by McCalls Macalloy Ltd. (1969) on typical British bars in use (Table XII). The modulus of elasticity is about 165 000 N/mm², although CP 110 suggests a value of 175 000N/mm².

With regard to relaxation Antill (1965) found that the load loss for a typical alloy steel bar, initially stressed to 70 per cent UTS is about 4 per cent at 1 000 hours, and double that at 100 000 hours. For comparison the performance of bars relative to other tendon components is shown in Fig. 18. This information is provided for designers bearing in mind that CP 110 advises that an "appropriate allowance for relaxation" be made "for sustained loading conditions".

The use of bar anchors is very popular in Germany and North America, where bar sizes are available from 6.4mm (No. 2 bar) to 25.4mm (No. 8 bar) in steps of 3.2mm, and thereafter to 35.8mm (No. 11 bar) in slightly larger increments.

Bars tend to be used as tendons in fairly short low-medium capacity anchors mainly in single bar situations, but are increasingly used in certain sophisticated forms in Germany, where compression tubes and elaborate end bearing devices are incorporated. Groups of up to four bars have been used on occasions, but larger groups are rare although Berardi (1960) successfully used

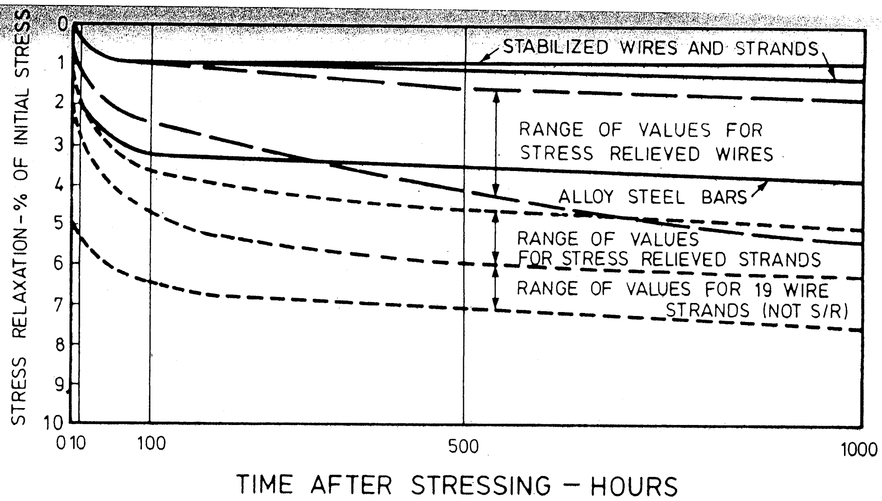


Fig. 18. Relaxation of British tendons at 20°C from initial stresses of 0.7 UTS

(after Antill, 1965)

20 No. 20mm plain bars for a 1 750kN test anchor.

2. *Wires*. Prestressing wire is manufactured from cold drawn plain carbon steel, and in a few countries, for example Germany, quenched and tempered (oil hardened and tempered or oil hardened) varieties predominate.

The ultimate tensile strength is inversely proportional to wire diameter, but also depends on the method of manufacture, and the steel specifications of the country concerned.

The major properties of British wire are summarised in Table XIII. CP 110 indicates that a typical value for the elastic modulus of wire and small diameter strands is 200 000N/mm².

It is noteworthy that Shchetinin (1974) reveals that Soviet industry produces wires of capacity 1 375-1 865N/mm² to meet the Soviet Code GOST, 7348-63. A popular choice for anchors is 5mm wire (1 670 N/mm²), with elastic modulus 184 000 N/mm² and 6.8 per cent relaxation at 1 000 hours. Wire tendons are recommended by Shchetinin on the basis that they eliminate suspected torsional and bending problems of strand anchors.

In general, tendons comprise between 10 and 100 wires (5-8mm diameter), dependent on the required anchor capacity, but 660 No. 5mm wires were employed at the Cheurfas Dam by Soletanche in 1934.

3. *Strand*. All strand is made from cold drawn plain carbon steel wire in Britain and seven wire strand is by far the most popular. Seven wire strands are stress relieved after stranding to produce a "normal relaxation" type, in two grades—regular

TABLE XIII—TECHNICAL DETAILS OF BRITISH PRESTRESSING WIRE

Wire diameter (mm)	*Characteristic strength (N/mm²)*	*Remarks*
Mill coil (BS 2691 Sect. 4)		
5.0*	1570*	Average E(N/mm²) = 192 000
	1670	0.2 per cent stress = 75 per cent specified
	1720	minimum strength.
4.5*	1620*	Average relaxation at 1 000 hours from
4.0*	1570*	70 per cent = 8 per cent ultimate at
	1670	20 deg C
	1770	
3.25*	1670	
	1720*	
	1770	
3.0	1670	
	1720*	
	1770	
2.65	1770	
	1870	
2.0	2020	
Prestraightened—Normal relaxation (BS 2691, Sect. 2)		
8.0	1470	Average E(N/mm²) = 201 000
	1570	
7.0*	1470	0.2 per cent proof stress = 85 per cent
	1570*	specified minimum strength.
	1670	
6.0	1470	Average relaxation at 1 000 hours from
	1570*	70 per cent = 3.8 per cent ultimate at
	1670	20 deg C
5.0*	1570*	
	1670	
	1720	
4.5	1570*	
	1670	
	1720	
4.0*	1570*	*Preferred sizes
	1670	
	1770	

TABLE XIV—TECHNICAL DETAILS OF BRITISH PRESTRESSING STRAND

Dia (mm)	*Minimum breaking load (kN)*	*Average E. (N/mm²)*	*Average relaxation at 1000 hrs from 70 per cent ultimate at 20 deg C (per cent)*	*Remarks*
Regular: normal relaxation (BS 3617 Sect. 2)				
6.4	44.5	198 000	5.6	The load at 1.0 per cent extension or
7.9	69.0	198 000	(7	0.2 per cent proof load ≡ 89 per cent
9.3	93.5	198 000	maximum)	actual breaking load (average).
10.9	125.0	198 000		The load at 0.01 per cent proportional
12.5*	165.0	198 000		limit ≡ 73 per cent actual breaking
15.2*	227.0	198 000		load. (average)
Regular: low relaxation (BS 3617 Sect. 3)				
9.3	93.5	200 000	1.1	The load at 1.0 per cent extension ≡
10.9	125.0	200 000	(2.5	89 per cent actual breaking load
12.5*	165.0	200 000	maximum)	(average).
15.2*	227.0	200 000		The load at 0.01 per cent proportional limit ≡ 80.5 per cent actual breaking load (average)
Super: normal relaxation				
9.6	102.5	197 000	5.5	Load at 1.0 per cent extension ≡ 85
11.3	138.0	197 000	(7	per cent actual breaking load
12.9*	184.0	197 000	maximum)	(average).
15.4*	250.0	197 000		Load at 0.01 per cent proportional limit ≡ 76 per cent actual breaking load (average)
Super: low relaxation				
9.6	102.5	198 000	1.15	Load at 1.0 per cent extension ≡
11.3	138.0	198 000	(2.5	90 per cent actual breaking load
12.9*	184.0	198 000	maximum)	(average).
15.4*	250.0	198 000		Load at 0.01 per cent proportional limit ≡ 79 per cent actual breaking load (average)
Dyform				
12.7*	209.0	198 500	1.1	Load at 1.0 per cent extension ≡
15.2*	300.0	196 500		92 per cent, 92 per cent and 91 per
18.0*	380.0	195 100		cent actual breaking load, respectively (average)
				Load at 0.01 per cent proportional limit ≡ 85 per cent, 85 per cent and 83 per cent actual breaking load, respectively (average)

*Preferred sizes

TABLE XVIII—DESIGN STRESSES AND SAFETY FACTORS WHICH HAVE BEEN EMPLOYED IN PRACTICE FOR STRAND TENDONS

Strand	Working stress (per cent ultimate)	Test stress (per cent ultimate)	Measured safety factor	Ultimate safety factor	Source
15.2mm	55	61	1.1	1.82	Britain—Ground Anchors Ltd. (1973)
15.2mm	58	80	1.37	1.71	France—Soletanche (1968)
12.7mm	48	57	1.2	2.1	Switzerland—VSL (1966)
12.7mm	30	73	2.43	3.3	Switzerland—Sommer et al (1974)
12.7mm & 15.2mm	60	—	—	1.67	Canada—Golder Brawner (1973)
12.7mm	65	80	1.23	1.54	Canada—Golder Brawner (1973)
15.2mm	50	80	1.6	2.0	Canada—Golder Brawner (1973)
12.7mm	52	78	1.5	1.93	USA—White (1963)
12.7mm	60	80	1.33	1.67	USA—Buro (1972)
15.2mm	59	79	1.34	1.69	USA—Schousboe (1974)
12.7mm	60	85	1.42	1.67	Australia—Langworth (1971)

TABLE XIX—PITCH OF TENDON SPACERS IN THE GROUTED FIXED ANCHOR ZONE

Pitch of tendon spacers (m)	Remarks	Source
0.5	Cheurfas Dam	USA—Zienkiewicz et al (1961)
0.5	3m fixed anchor	Czechoslovakia—Hobst (1965)
0.8	Multi-wire tendons	France—Cambefort (1966)
0.6	VSL anchors	Switzerland—Losinger SA (1966)
0.8-1.6	Multi strand tendons	Britain—Littlejohn (1972)
2.0	Multi strand tendons	Italy—Mascardi (1972)
0.5-2.0	Dependent on "stiffness" of tendon system	Germany—Stocker (1973)
1.5-2.0	Conenco (Freyssinet) anchors	Canada—Golder Brawner Assocs. (1973)
1.8	7.3m fixed anchor	USA—Chen et al (1974)
2.0	8m fixed anchor (12 No. 15.2mm strands)	Britain—Littlejohn et al (1974)
0.5	Multi-wire tendons	USSR—Shchetinin (1974)

to long term load losses—usually 10 per cent.

Tendon spacers

Spacers are used in both the free and fixed sections of multicomponent tendons. In the free length they may serve to centralise the tendon with respect to the borehole but their main function is to prevent tangling or rubbing of the individual bars, wires or strands. This is particularly important in long, flexible tendons, where, if the tendon is allowed to lose its design geometry, load may be dissipated through friction in the free length during stressing. In addition, extremely high stress concentrations may develop, particularly just under the top anchor head, where rupture of individual elements can easily occur. Spacers in this part of the anchor are hollow cored and between 4-8m apart.

In the grouted fixed anchor zone the spacers encourage effective penetration of grout between the tendon units, thereby ensuring efficient transmission of bond stress. In addition the spacer units should be designed to centralise the tendon in the borehole to (*a*) avoid contamination of tendon e.g. clay smear, and (*b*) give adequate cover of grout for corrosion protection and good grout bond at the borehole interface.

Spacers in this zone may also be used in conjunction with intermediate fastenings to form nodes or waves, in order to provide a more positive mechanical interlock between the tendon and surrounding grout. Whilst this method gives a tendon geometry which allows adequate penetration and cover of grout, it is important to note that the practice of unravelling strands followed by bushing of the wires gives a random geometry which cannot guarantee efficient load transfer.

With reference to the pitch of spacers, Table XIX gives an indication of the distances which have been employed in practice. In general it would appear that little work has been carried out on the influence of pitch or spacer design on load transfer in the fixed anchor zone.

Remarks

Whilst tendons are produced to a high standard and reliable minimum breaking loads are specified for use by the designer, few load/extension tests have been carried out on long tendons (10-30m) which are comparable in size to the free anchor lengths used in practice. Since interpretation of anchor load/displacement characteristics can be quite controversial in practice, particularly in the case of strand, 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 0.61m. The influence of tendon curvature, and splaying of multicomponent tendons near the top anchor head on stress/strain behaviour also requires clarification in view of the dearth of published information, at present. Top anchor heads will be discussed in Part 3 of this review.

GENERAL CONCLUSIONS

Although rock anchors have been used for over 40 years, it is difficult to justify technically certain aspects of contemporary design. Progress in the development and rationalisation of design has been slow, largely due to the scarcity of reliable laboratory and field experimental data relating directly to rock anchors.

As a result, practising engineers have been obliged to make reference to values and methods employed with apparent success in earlier designs, without fully appreciating or understanding their accuracy or reliability. ~~Bearing this in mind~~, it is perhaps understandable that the majority of designs are overconservative in certain aspects, if not in all. This dilemma is becoming increasingly acute now that engineers are being requested to design for circumstances where no exact precedents exist.

In view of the inconsistencies between theory and practice which have been highlighted in this design review, it is considered that more attention should be directed towards studies in the following areas:

1. *Uplift capacity.* There is little justification for the inverted cone method of assessing the ultimate resistance of withdrawal of the rock anchor system. However, until full-scale field tests are carried out to study modes of failure in relation to the geotechnical properties of rock masses, the present method of design, where rock shear strength is ignored, must be persevered with, as it is basically very conservative. Nevertheless, some standardisation on acceptable modes of failure, safety factors and allowances for unconsolidated overburden is now required.

2. *Fixed anchor.* A uniform distribution of bond stress is assumed in the vast majority of anchor designs, although this approach is only valid in the case of soft rock. In hard rock, the stress distribution is non-uniform, the highest stresses being mobilised at the proximal end of the grouted fixed anchor zone. The ratio E_{grout}/E_{rock} has a major influence on stress distribution, although the authors find that rock masses are seldom classified in sufficient detail for other potentially important parameters to be highlighted. The phenomena of debonding in rock anchors is not well understood, although it has undoubtedly been significant in certain high capacity anchors described. Values for the magnitude of bond at the grout tendon interface are usually abstracted from publications relating to reinforced and prestressed concrete. However, it should be noted that the boundary conditions existing in conventional bond tests, may be wholly different from these present in the rock anchor situation.

References

Abraham, K. and Porzig, R. (1973): 'Die Felsanker des Pumpspeicherwerkes Waldeck II'' Baumaschine und Bautechnik, 20 (6), pp. 209-220.

Adam, M. (1972): Reply to FIP questionnaire.

Anderson, G. F., Rider, J. H. and Sozen, M. A. (1964): ''Bond Characteristics of Prestressing Strand'' Prog. Report 13 of Project 1HR-10, Illinois Coop. Highway Res. Prog., Eng. Exp. Stn., Univ. Illinois, June, 157 pp.

Anon (1970): ''Rock Stabilisation in the Vicinity of the Intake Tower, El Atazar'' Report on test cables. Source unknown 8pp.

Anon (1972): ''Sea of Anchors and Bolts Keep Powerhouse Cavern Shipshape''. Eng. News Record, July 20, pp. 39-40.

Antill, J. M. (1965): ''Relaxation Characteristics of Prestressing Tendons''. Civ. Eng. Trans. Inst. Eng. Aust., 7 (2) pp. 151-159.

Armstrong, W. E. I. (1949): ''Bond in Prestress concrete''. J. Inst. Civ. Engng., *33*, pp. 19-40.

Associated Tunnelling Ltd. (1973): ''Report on Ground Anchor Tests— Thames Flood Prevention''. Unpublished (14 pp.) Lowton St. Mary's near Warrington, England.

Banks, J. A. (1955): ''The Employment of Prestressed Techniques on Allt-na-Lairige Dam''. 5th Int. Cong. on Large Dams, Paris, 2, pp. 341-357.

Barron, K., Coates, D. F. and Gyenge, M. (1971): ''Artificial Support of Rock Slopes''. Dept. of Energy, Mines and Resources, Mines Branch, Ottawa. Research Report R 228 (Revised), 145 pp.

Base, G. D. (1961): ''An Investigation of the Use of Strand in Pretensioned Prestressed Concrete Beams''. Research Report No. 11, 12 pp., Cement and Concrete Assoc., London.

Beomonte, M. (1961): ''Criteri Peril Calcalo e la Posa in Opera di Bulloni di Ancoraggio''. Geotecnica, *1* (Feb.), pp. 5-13.

Berardi, G. (1960): ''Ricerche Teoriche e Sperimentali Sugli Ancoraggi in Roccia''. Geotecnica, 6, 12 pp.

Berardi, G. (1967): ''Sul Comportamento Degli Ancoraggi Immersi in Terreni Diversi''. Univ. Genoa. Inst. Constr. Sc. Series III. No. 60, 18 pp.

Berardi, G. (1969): ''Sui Problemi Geotechnici Dellacittà di Genova con Particulare Riguardo Alle Pareti Rocciose''. Rivista Italiana di Geotecnica, *1*, 21 pp.

Berardi, G. (1972): ''Predimento per la Bonifica e Sostegno dei pendii a Mezzo di Chiodatura ed Ancoraggi''. 3rd Conf. on Problems of Foundation

Engineering, Turin, 16 March, 60 pp. 16 Nov.

Birkenmaier, M. (1953): "Vorgespannte Felsanker". Schweizerische Bauzeitung, 71 (47), pp. 688-692.

Boyne, D. M. (1972): "Use of Skin Friction Values in Rock Anchor Design", BSc(Hons) thesis, Engineering Dept., University of Aberdeen.

Bridon Wire Ltd. (1968): "Wire and Strand for Prestressed Concrete".

Bridon Wire Ltd. (1971): "Wire and Strand for Prestressed Concrete". Section No. 1, Edition No. 5.

Bridon Wire Ltd. (1971): "Dyform Prestressing Strand". Section No. 2, Edition No. 3. Bridon Wire Ltd., Doncaster, England.

British Standards Institution (1972): "The Structural Use of Concrete". CP 110, Part 1, BSI, 2 Park Street, London.

Broms, B. B. (1968): "Swedish Tieback Systems for Sheet Pile Walls". 3rd European Conf. S.M. & F.E.—Budapest, pp. 391-403.

Brown, D. G. (1970): "Uplift Capacity of Grouted Rock Anchors". Ontario Hydro Research Quarterly, 22 (4), pp. 18-24.

Brunner, H. (1970): "Praktische Erfahrungen bei den Hangsicherungen in Neuenhof", Strasse und Verkehr, 9, pp. 500-505.

Bujak, M., Glab, W., Moraczewski, K. and Wolski, W. (1967): "Preventative Measures Against the Rock Slide at Tresna Dam Site". 9th Int. Cong. on Large Dams, Istanbul, 1, pp. 1027-1035.

Bundred, J. (1973): "Insitu Measurements of Earth Pressure and Anchor Forces for a Diaphragm Retaining Wall". Symp. on Field Instr. in Geotech. Eng., London, pp. 52-69.

Bureau Securitas (1972): "Recommendations Regarding the Design, Calculation, Installation and Inspection of Ground Anchors". Editions Eyrolles, 61 Boulevard Saint-Germain, Paris-Ve (Ref. TA 72).

Buro, M. (1972): "Rock Anchoring at Libby Dam". Western Construction (March), pp. 42, 48 and 66.

Cambefort, H. (1966): "The Ground Anchoring of Structures". Travaux, 46 (April-May), 15 pp.

Cementation Co. Ltd. (1962): "Stressing". Technical Report No. 10 (unpublished), Croydon, England.

Chen, S. C. and McMullan, J. G. (1974): "Similkameen Pipeline Suspension Bridge". Proc. ASCE, Transportation Eng. J. 100 (TE1), pp. 207-219.

Coates, D. F. (1970): "Rock Mechanics Principles". Department of Energy, Mines and Resources. Mines Monograph No. 874, Ottawa.

Coates, D. F. and Yu, Y. S. (1970): "Three Dimensional Stress Distributions around a Cylindrical Hole and Anchor". Proc. 2nd Int. Conf. on Rock Mechanics, Belgrade, 2, pp. 175-182.

Comte, C. (1965): "L'utilisation des ancrages en Rocher et en Terrain Meuble". Bull. Tech. de la Suisse Romande, 22 (Oct), pp. 325-338.

Comte, C. (1971): "Technologie des Tirants". Institute Research Foundation Kollbrunner/Rodio, Zurich, 119 pp.

Conti, N. (1972): Reply to FIP questionnaire.

Da Costa Nunes, A. J. (1969): "Anchorage tests in clays for the Construction of Sao Paulo Subway". 7th Int. Conf. of the Int. Soc. for Soil Mech. and Found. Eng., Mexico, Paper 15-2, pp. 120-125.

Da Costa Nunes, A. J. (1971): "Metodas de Ancoragens". Parts (1) and (2), Associaçao dos Antigos Alumos da Politecnica. 50 pp.

Descoeudres, J. (1969): "Permanent Anchorages in Rock and Soils". 7th Int. Conf. of Int. Soc. Soil Mech. and Found. Eng., Mexico, Paper 15-17 pp. 195-7.

Deutsche Industrie-Norm (1972): Verprebanker für Vorübergehende Zwecke im Lockergestein: Bemessung, Ausführung und Prüfung. DIN 4125, Blatt 1, 1972.

Drossel, M. R. (1970): "Corrosion Proofed Rock Ties Save Building from Earth Pressures". Constr. Methods, May, pp. 78-81.

Eberhardt, A. and Veltrop, J. A. (1965): "Prestressed Anchorage for Large Trainter Gate". Proc. ASCE, J. Struct. Div., 90 (ST6), pp. 123-148.

Fargeot, M. (1972): Reply to FIP questionnaire.

FIP (1972): "Draft of the Recommendations and Replies to FIP Questionnaire (1971)". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1973): "Final Draft of Recommendations". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1974): "Questionnaire on Choice of Tendon". FIP Notes 50, pp. 4-8, Cement and Concrete Assoc., 52 Grosvenor Gardens, London SW1W 0AQ.

Feld, J. and White, R. E. (1974): "Prestressed Tendons in Foundation Construction". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 25-32, 7th FIP Congress, New York.

Fenoux, G. Y. and Portier, J. L. (1972): "Lamise en Précontrainte des Tirants". Travaux, 54 (449-450), pp. 33-43.

Gilkey, H. J., Chamberlin, S. J. and Beal, R. W. (1940): "Bond between Concrete and Steel". Reproduced in Eng. Report No. 26, Iowa Eng. Exp. Station, Iowa State Coll. Ames (1956), pp. 25-147.

Golder Brawner Assocs. (1973): "Government Pit Slopes Project III: Use of Artificial Support for Rock Slope Stabilisation". Parts 3.1-3.6, Unpublished, Vancouver, Canada.

Gosschalk, E. M. and Taylor, R. W. (1970): "Strengthening of Muda Dam Foundations using Cable Anchors". Proc. 2nd Cong. Int. Soc. Rock Mech., Belgrade, 3, pp. 205-210.

Ground Anchors Ltd. (1974): "The Ground Anchor System". (28 pp.) Reigate, Surrey.

Hanna, T. H. and Seeton, J. E. (1967): "Observations on a Tied-back Soldier-pile and Timber-lagging Wall". Ontario Hydro Res. Qtly., 19 (2), pp. 22-27.

Hanson, N. W. (1969): "Influence of Surface Roughness of Prestressing Strand on Bond Performance". J. Prestr. Conc. Inst., 14 (1), pp. 32-45.

Hawkes, J. M. and Evans, R. H. (1951): "Bond Stresses in Reinforced Concrete Columns and Beams". Structural Engineer, 29 (12), pp. 323-327.

Heitfield, K. and Schauerte, E. (1969): "Badentung der Geologischen Verhältnisse für die Böschungssicherung am Ehemaligen Hofolder Tunnel (Sauerland)". Z. Deutsch Geol. Ges., 119, pp. 270-284.

Hennequin, M. and Cambefort, H. (1966): "Consolidation du Remblai de Malherbe". Revue Generale des Chemins de Fer (Feb), 9 pp.

Hilf, J. W. (1973): Reply to Aberdeen questionnaire (1972), unpublished.

Hobst, L. (1965): "Vizepitmenyek Kihorgonyzasa". Vizugi Kozlemenyek, 4, pp. 475-515.

Hobst, L. (1969): "Stabilizace Svahú Pred petim". Invényrské Stavby, 9-10, pp. 353-359.

Irwin, R. (1971): Reply to FIP questionnaire.

Jackson, F. S. (1970): "Ground Anchors—the Main Contractor's Experience". Supplement on Ground Anchors, The Consulting Engineer, May.

Japanese Government, Construction Ministry (1964): "Foundation Treatment of Kawamata Dam" Proc. 8th Int. Cong. on Large Dams, Edinburgh, 1, pp. 187-207.

Jasper, J. and Shtenko, V. (1969): "Foundation Anchor Piles in Clay—Shales". Canadian Geot. Journal, 6, pp. 159-175.

Johannesburg City Engineer's Dept. (1968): "Cable Anchors; Design Procedures". Unpublished Memorandum (4 pp.).

Judd, W. R. and Huber, C. (1961): "Correlation of Rock Properties by Statistical Methods". International Symposium on Mining Research.

Juergens, R. E. (1965): "Cables Grouted into Bedrock Brace High Retaining Walls". Construction Methods, April, pp. 172-176.

Kemp, E. L., Brezny, F. S. and Unterspan, J. A. (1968): "Effect of Rust and Scale on the Bond Characteristics of Deformed Reinforcing Bars". J. Amer. Conc. Inst., 65 (9), pp. 743-756.

Klein, K. (1974): "Draft Standard for Prestressed Rock Anchors". Symposium on Rock Anchoring of Hydraulic Structures, Vir Dam, November 7-10, pp. 86-102.

Klopp, R. (1970): "Verwendung Vorgespannter Felsanker in Geklüftetem Gebirge aus Ingenieur Geologischer Sicht". Der Bauingenieur, 45 (9), pp. 328-331.

Koch, J. (1972): Reply to FIP questionnaire.

Koziakin, N. (1970): "Foundations for US Steel Corporation Building in Pittsburg, Pa". Civ. Eng. and Pub. Works Review, Sept. pp. 1029-1031.

Lang, T. A. (1961): "Theory and Practice of Rock Bolting". Trans. Am. Inst. Min. Eng., 220, pp. 333-348.

Larson, M. L., Willette, W. R., Hall, H. C. and Gnaedinger, J. P. (1972): "A Case Study of a Soil Anchor Tie-back System". Proc. Spec. Conf. on Performance of Earth and Earth Supported Structures, Purdue Univ., Indiana, 1 (2), pp 1341-1366.

Laurikainen (1972): Reply to FIP questionnaire.

Littlejohn, G. S. (1970): "Anchorages in Soils—Some Empirical Design Rules". Supplement on Ground Anchors, The Consulting Engineer (May).

Littlejohn, G. S. (1972): "Some Empirical Design Methods Employed in Britain". Part of Questionnaire on Rock Anchor Design, Geotechnics Research Group, Department of Engineering, University of Aberdeen (unpublished Technical Note).

Littlejohn, G. S. (1973): "Ground Anchors Today—A Foreword" Ground Engineering, 6 (6), pp. 20-23.

Littlejohn, G. S. and Truman-Davies, C. (1974): "Ground Anchors at Devonport Nuclear Complex". Ground Engineering, 7 (6), pp. 19-24.

Longworth, C. (1971): "The use of Prestressed Anchors in Open Excavations and Surface Structures". Australian Inst. Mining and Metallurgy (Illwarra Branch), Symposium on Rock Bolting, 17-19 Feb., Paper No. 8, 17 pp.

Losinger and Co. (1966): "Prestressed VSL Rock and Alluvium Anchors". Technical Brochure, Berne, 15 pp.

Losinger and Co. (1972): Reply to FIP questionnaire.

McCalls Macalloy Ltd. (1969): "Macalloy Prestressing Stressing Procedures". Publications 46 (July), 1st Edition, 24 pp.

Macleod, J. and Hoadley, P. J. (1974): "Experience with the Use of Ground Anchors". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors. pp. 83-85, 7th FIP Congress, New York.

McRostie, G., Burn, K. and Mitchell, R. (1972): "The Performance of Tied-back Sheet Piling in Clay". Can. Geotech. J. 9, pp. 206-218.

Maddox, J., Knistler, F. and Mather, R. (1967): "Foundation Studies for Meadowbank Buttress Dam". Proc. 9th Int. Cong. on Large Dams, Instanbul, 1, pp. 123-141.

Maijala, P. (1966): "An Example of the Use of Long Rock Anchors". Proc. 1st Int. Cong. Int. Soc. Rock Mech., Lisbon, 3, pp. 454-6.

Mantovani, E. (1970): "Method for Supporting Very High Rock Walls in Underground Power Stations". Proc. 2nd Int. Cong. Int. Soc. Rock Mech., Belgrade, 2, pp. 157-165

Mascardi, C. (1973): "Reply to Aberdeen questionnaire (1972)".

Mitchell, J. M. (1974): "Some Experiences with Ground Anchors in London". ICE Conference on Diaphragm Walls and Anchors", London (Sept.), Paper No. 17.

Mongilardi, H. (1972): "Consolidamento di una Scarpata Mediante Ancoraggi Pro fondi". Strade, 11, pp. 669-678.

Morris, S. S. and Garrett, W. S. (1956): "The Raising and Strengthening of the Steenbras Dam". (and discussion) Proc. ICE Pt. 1, Vol. 5, No. 1, pp. 23-55.

Moschler, E. and Matt, P. (1972): "Felsanker und Kraftnessanlage in der Kaverne Waldeck II". Schweizerische Bauzeitung, 90 (31), pp. 737-740.

Muller, H. (1966): "Erfahnungenmit Verankerungen System BBRV in Fels—und Lockergesteinen". Schweizerische Bauzeitung, 84 (4), pp. 77-82.

Nicholson Anchorage Co. Ltd. (1973): "Rock Anchor Load Tests: for Sheet Pile Bulkhead for the General Reinsurance Corp. Project, Steamboat Road, Greenwich, Conn., USA". Unpublished Report (8 pp.).

Nordin, P. O. (1968): "Insitu Anchoring". Rock Mech. and Eng. Geol, 4, pp. 25-36.

Ontario Hydro (1972): "Provisional Specification M 388 and Standard Specification M285, Toronto, Canada.

Oosterbaan, M. D. and Gifford, D. G. (1972): "A Case Study of the Bauer Earth Anchor". Proc. Spec. Conf. on Performance of Earth and Earth Supported Structures, 1 (2), pp. 1391-1400.

Parker, P. I. (1958): "The Raising of Dams with Particular Reference to the Use of Stressed Cables". Proc. 6th Cong. on Large Dams, New York, Question 20, 22 pp.

Parry-Davies, R. (1968): "The use of Rock Anchors in Deep Basements". Ground Engineering Ltd., Johannesburgh, South Africa, unpublished paper.

PCI Post-Tensioning Committee (1974): "Tentative Recommendations for Prestressed Rock and Soil Anchors". PCI, Chicago, USA, 32 pp.

Pender, E., Hosking, A. and Mattner, B. (1963): "Grouted Rock Bolts for Permanent Support of Major Underground Works". J. Inst. Eng. Aust., 35, pp. 129-150.

Phillips, S. H. E. (1970): "Factors Affecting the Design of Anchorages in Rock". Cementation Research Report R48/70 Cementation Research Ltd., Rickmansworth, Herts.

Pliskin, L. (1965): "Ancrages Précontraints dans le Rocher Système Freyssinet". Bull. Tech. de la Suisse Romande, 16 (7 Aug.), pp. 253-7.

Rao, R. M. (1964): "The Use of Prestressing Technique in the Construction of Dams". Indian Concrete Journal, August, pp. 297-308.

Rawlings, G. (1968): "Stabilisation of Potential Rockslides in folded Quartzite in Northwestern Tasmania". Engineering Geology, 2 (5), pp. 283-292.

Rescher, O. J. (1968): "Aménagement Hongrin—Léman—Soutènement de la Centrale en Cavernes de Veytaux par Tirants en Rocher et Béton Projeté", Bull. Tech. de la Suisse Romande, 18 (7 Sept.), pp. 249-260.

Reti, G. A. (1964): "Slope stabilised by Anchored Retaining Wall". Civ. Eng. (NY), 34 (4), pp. 49-53

Roberts, N .P. (1970): "Deformed Bars in Concrete". Concrete (July), pp. 306-8.

Ruttner, A. (1966): "Anwendung von Vorgespannten Felsankern (System BBRV) bei der Erhöhung der Spullersee—Telsperren". Schweizerische Bauzeitung, 77 (47), pp. 773-777.

Saliman, R. and Schaefer, R. (1968): "Anchored Footings for Transmission Towers". ASCE Annual Meeting and National Meeting on Structural Engineering, Pittsburg, Pa., Sept. 30-Oct. 4, Preprint 753, 28 pp.

Schmidt, A. (1956): "Rock Anchors Hold TV Tower on Mt. Wilson" Civil Engineering, 56, pp. 24-26.

Schousboe, J. (1974): "Prestressing in Foundation Construction". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 75-81. 7th FIP Congress, New York.

Schwarz, H. (1972): "Permanent Verankerung Einer 30m Hohen Stützwand in Stuttgarter Tonmergel durch Korrosions—Geschützte Injektionsanker, System Duplex". Die Bautechnik, 9, pp. 305-312.

Shchetinin, V. V. (1974): "Investigation of Different Types of Flexible Anchor Tendons (Rock Bolts) for Stabilisation of Rock Masses". Gidrotekniches-koe Stroitel' Stuo, 4 (April), pp. 19-23.

Soletanche Entreprise (1968): "La Surrelevation du Barrage des Zardezas sur l'oued Saf-Saf". Unpublished Report, 4 pp.

Sommer, P. and Thurnherr, F. (1974): "Unusual Application of VSL Rock Anchors at Tarbela Dam, Pakistan". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 65-66. 7th FIP Congress, New York.

South African Code of Practice (1972) "Lateral Support in Surface Excavations". The South African Institution of Civil Engineers, Johannesburg.

Standards Association of Australia (1973): "Prestressed Concrete Code CA35—1973, Section 5—Ground Anchorages". pp. 50-53.

Stocker, M. A. and Sozen, M. (1964): "Investigation of Prestressed Reinforced Concrete for Highway Bridges, Part V: Bond Characteristics of Prestressing Strand". Univ. Illinois Eng. Res. Station, Bulletin 503.

Stocker, M. F. (1973): Reply to Aberdeen questionnaire.

Suzuki, I., Hirakawa, T., Morii, K. and Kanenko, K. (1972): "Developpments Nouveaux Dans les Fondations de Pylons pour Lignes de Transport THT du Japon". Conf. Int. des Grande Réseaux Electriques à Haute Tension, Paper 21-01, 13 pp.

Thompson, F. (1970): "The Strengthening of John Hollis Bankhead Dam". Civ. Eng. (NY), 39 (12), pp. 75-78.

Universal Anchorage Co. Ltd. (1972): "Report on Rock Anchor Tests for Frigate Complex HM Dockyard, Devonport". Report No. 189, Universal Anchorage Co. Ltd., Egerton Street, Farnworth, Bolton, unpublished.

Universal Anchorage Co. Ltd. (1972): "Ground Anchors Simplify Excavation for Telephone Exchange Foundations". Contract Journal, April 6.

Voves, B. (1972): Reply to FIP questionnaire.

Walther, R. (1959): "Vorgespannte Felsanker". Schweizerische Bauzeitung, 77 (47), pp. 773-7.

White, R. E. (1973): Reply to Aberdeen questionnaire.

Williams, A. F. and Muir, A. G. (1972): "Stabilisation of a Large Moving Rock Slide with Cable Anchors". Report of County Roads Board of Victoria, Melbourne.

Wolf, W., Brown, G. and Morgan, E. (1964): "Morrow Point Underground Power Plant". Source unknown.

Wosser, T. and Darragh, M. (1970): "Tie-backs for Bank of America Building Excavation Wall". Civ. Eng. (NY), 40 (3), pp. 65-67.

Wycliffe-Jones, P. J. (1974): "Personal communications.

Zienkiewicz, O. C. and Gerstner, R. W. (1961): "Stress Analysis of Prestressed Dams". Proc. ASCE, Journal of the Power Division, 87 (PO1), Pt. 1, pp. 7-43.

Part 2: Construction

INTRODUCTION

IRRESPECTIVE OF THE care and conservatism applied to the design§ of an anchor system, thoughtless or careless constructional procedures can cause rock anchors to fail at very low loads. The majority of failures seem to be related to the grouting stage although some bond failures have clearly been due to poor tendon preparation. On a few occasions the drilling and flushing techniques may have been incorrect. Fortunately, failures have not occurred too often and these have usually been highlighted at the stressing and testing stage.

It is significant that although the technology of drilling and grouting can be highly complex, site techniques on the whole are left to skilled and experienced specialists, and close on-site inspection by supervising engineers has been relatively uncommon to date. Thus, rock anchoring after 40 years is still regarded as an art. Whilst it is appreciated that the highly variable ground conditions encountered in practice, giving rise to a large number of construction techniques, add to the mystique of anchoring, nevertheless it seems that the time is overdue for certain guidelines on construction practice to be presented for consideration by civil engineers.

The second part of this review discusses anchor construction techniques related to drilling, flushing, water testing, tendon preparation and installation, grouting and finally corrosion protection§. Since anchor construction is sensitive to poor workmanship emphasis is placed on quality control and close on-site supervision.

Aspects of anchor stressing and testing will be reviewed in the third and concluding part of this series of articles.

DRILLING

Introduction

In practice drilling rates often dictate anchor production rates and therefore influence in a major way overall costs. As a result major decisions to be taken by anchor specialists before each contract include

(i) The selection of the most suitable and efficient drilling method, and
(ii) The prediction of penetration rates.

With respect to choice of drilling method, the rock type, rate and scale of drilling operations, availability of plant, hole geometry and labour and drilling costs must all be assessed.

The prediction of drilling rates involves careful study of machine characteristics, bit and flushing medium properties as well as rock and borehole parameters. It is considered that a prior knowledge of drilling rates provides a sound basis for evaluating the feasibility of planned operations and for selecting alternative operational procedures if necessary.

The range and selection of drilling equipment and methods are described briefly, together with guide information on the prediction of drilling rates. The latter is perforce qualitative, simply because insufficient research has yet been conducted —or published—on the determination of "rock drillability indices". Drilling tolerances are mentioned in relation to current rock anchor practice.

Drilling methods

The major mechanical drilling systems in use are rotary, percussive and rotary percussive. Each system is characterised by the manner in which the bit attacks the rock, and a simple comparative analysis of the mechanics of various drilling systems can often reveal the inherent limitations of each and indicate the most promising system for a specific type of rock. For example a rock of high compressive strength, regardless of its abrasive properties, is likely to respond well to the crushing/chipping action of a percussion bit. On the other hand, a rock classified as hard because it is highly abrasive, but which is weakly bonded, may respond to percussive action more like a ductile material than a brittle one. For such a rock a percussion bit would do inferior work compared with a wear-resistant rotary drag bit. A current rule of thumb for the applicability of drilling methods for different rock categories is based on the resistance of rock to penetration, as shown in Table I.

Rotary drills

A rotary drill imparts two basic actions through the drill rod and bit into the rock —(i) axial thrust (a static action), and (ii) rotational torque (a dynamic action).

The resultant force applied to the rock is increased until rock fracture is induced and each machine has a point where an optimum axial thrust interrelated with the available torque can achieve a maximum penetration rate for a particular rock. Operating below the optimum thrust decreases the penetration and imparts a noticeable polishing or grinding action to the bit. Operating above the optimum thrust requires high rotational torque, and stalling of the machine is likely.

In general, rotary drills have a higher torque output than either percussive or rotary-percussive drills and require higher thrust capabilities. Types of machines and operating practice are described in detail in a US Army Report [1964].

Where specified, most core drilling is carried out using diamond bits which are available in two main forms—(a) "Surface set" bits with individual diamonds set in a metal matrix, and (b) "Impregnated bits" with fine diamond dust incorporated in a matrix.

The diamonds used for the surface set bits vary in both quality and size. Choice is governed by the rock to be drilled, but it can be summarised that "the harder the rock, the smaller the size and the higher the quality of the diamonds". Dixon and Clarke (1975) give specific recommendations on size of diamonds in bits related to type of rock. It is noteworthy that tungsten bits are less costly than diamond bits but are not regarded as suitable for drilling in very hard rocks.

When drilling with surface set diamond bits, Paone *et al* [1968] have shown that the most significant parameters affecting penetration rates are thrust and rotation speed of the drill, and the rock compressive strength, hardness, and quartz content.

Diamond drilling is not commonly employed in anchoring, partly for economic reasons, and partly due to the smoothness of the hole it creates, thereby leading to poorer rock-grout bond characteristics. Borehole roughness is undoubtedly increased by using percussive methods, but to date this does not appear to have been quantified.

For anchor construction in soft rock formations, such as stiff-hard clays and

TABLE I. APPLICATION OF DRILLING SYSTEMS

Method	*Resistance to penetration of rock*			
	Soft	*Medium*	*Hard*	*Very hard*
Rotary-drag bit	X	X		
Rotary-roller bit	X	X	X	
Rotary-diamond bit	X	X	X	X
Percussive	X	X	X	X
Rotary-percussive	X	X	X	

(After Paone, Under and Tandanand, 1968)

TABLE II. DRILLING METHODS AND EQUIPMENT RELATED TO GROUND CONDITIONS

Basic method	*Percussive*	*Percussive*	*Percussive*	*Rotary*	*Rotary*
Drill string	Standard coupled rods, separate anchor	Coupled rods also act as anchor	Coupled drill tubes and rods used simultaneously from same drive adapter, Atlas Copco Overburden Drilling method	Coupled flight augers	Standard rotary drilling tubes
Drilling machine	Wagon drill with drifter or crawler drill with independent rotation drifter. Compressed air powered.	Wagon drill with drifter or crawler drill with independent rotation drifter. Compressed air powered.	Special independent rotation drifter mounted on heavy wheeled chassis or crawler. Compressed air powered.	Standard auger drill capacity of torque and thrust dependent on hole size and depth. Diesel/ hydraulic power. Chassis powered wheel or crawler designed for drilling of shallow angle holes. Wheeled or skid mount possible.	Rotary rod drill or diamond drill. Performance about 2.7m.kN torque, 50kN thrust 0-500 r.p.m. Diesel/hydraulic power. Chassis powered wheel or crawler designed for drilling of shallow angle holes. Wheeled or skid mount possible.
Anchor	Muti-wire strand or single bar.	Special coupled rods	Multi-wire strand and single bar.	Multi-wire strand most common. Single bar also possible.	Single bar most common as in Bauer system. Multi-wire strand possible where ground is self-supporting.
Flushing medium	Normally air but water could be used.	Invariably water but air occasionally useful.	Water. Air used very rarely.	None	Water. Air used very rarely.
SUITABLE STRATA	Self-supporting rock only. Few metres of overburden possible with aid of stand pipe.	All materials.	All materials provided drill tubes are uncoupled when rock is encountered and drilling continued alone with rods.	All self-supporting soft material such as clay and chalk. Not rock. Not collapsible material such as sand and gravel unless casing is used.	All soft materials such as clay, sand and gravel. Also soft and medium rocks. Not hard rock.

(Modified after Mawdsley, 1970)

marls, augers are often employed. They fall into three broad categories:
(*1*) standard continuous flight augers for normal open/hole drilling,
(*2*) continuous flight augers with hollow couplings to permit water, bentonite or cement grout to be pumped into the bottom of the hole, and
(*3*) hollow stem augers with a removable centre bit to facilitate sampling through the centre of the auger during the drilling stage, and subsequently to permit homing of the tendon prior to withdrawal of the auger. Augers are available which can accept the standard U4 sampler tube, and on occasions this drilling method can be very attractive from a quality control point of view.

In general, a wide range of drill bits is available from auger tool manufacturers but experience is required in making the correct choice in practice. For example, a tungsten tipped finger bit is normally suitable for moderate to hard formations such as hard shale, siltstone, and soft decomposed sandstone whilst a fishtail bit is often ideal for boring clean holes through soft shale and stiff/hard clay.

Percussive drills

Percussive drills penetrate rock by the action of an impulsive blow, usually from a chisel or wedge-shaped bit: repeated application of a high intensity short duration force crushes or fractures rock when the blow is sufficiently large. Torque, rotational speed, and thrust requirements are significantly lower for percussive systems than they are for rotary or rotary percussive systems.

Hammer drills, in which the hammer remains at the surface, are used for drilling holes up to 125mm in diameter. Down-the-hole tools, (DTH) in which the hammer is always immediately above the bit, are used mainly for hole diameters ranging from 120 to 750mm.

Penetration rates of percussive drills are shown by Ryd & Holdo [1956] to be proportional to the rate at which energy is supplied by the reciprocating piston.

Rotary-percussive drills

These drills impart three actions through the drill bit:
(*i*) axial thrust of lower magnitude than that of a rotary drill,
(*ii*) torque, lower than a rotary drill but much higher than a percussive drill, and
(*iii*) impact.

The rotation mechanisms may be powered by the impact mechanism or by a separate motor, and the mechanism of rock failure is considered by White [1965] to combine the characteristics of both rotary and percussive mechanisms.

Choice of drilling method

The method of drilling is chosen primarily with respect to
(*a*) the type and capacity of the anchor, and hence the diameter and depth of the hole,
(*b*) the nature of the rock material and mass,
(*c*) the borehole surface roughness requirements,
(*d*) the accessibility and topography of the site,
(*e*) the availability and suitability of the flushing medium, and
(*f*) the drilling rate.

A guide to the choice of drilling method is given by Mawdsley [1970] who considers that in the majority of projects the most important factors affecting choice are the type of anchor and the strata to be drilled (see Table II). Parker [1958] writes that for holes up to 100m dia. and 60m in length percussive methods are preferable for most rock conditions. For deeper holes, which put a severe strain on percussive equipment, or poorer ground conditions, rotary methods are recommended. McGregor [1967] summarises in general terms the relation between rock type and diameter as shown in Fig. 1, and emphasises the differences (see Figs., 2 and 3) when drilling in soft friable rocks and variable strata. Where the rock has alternating hard and soft (collapsible) zones the use of a rotating eccentric bit has proved a successful innovation in recent years since it underreams the rock permitting the use of a uniform size of casing, as opposed to the more traditional use of telescopic casing which gradually reduces in size with increasing depth.

It is noteworthy that one of the disadvantages of the down-the-hole hammer (DTH) was illustrated recently at Muda Dam in Malaysia where two very expensive hammers were jammed at depth. Normally, the down-the-hole hammer is less prone to jamming than the ordinary percussive drill but when it does, the financial consequences are greater.

Drilling equipment

Irrespective of the method of drilling, there are certain desirable characteristics which are common to most rigs used in ground anchoring work. For instance, Mawdsley [1970] recommends the following items.

The rig should have powered traction so that it can easily be moved and positioned for each hole. When site floor conditions are bad the rig should be mounted on crawler tracks. An exception to the above is when the rig is mounted on another piece of equipment which is itself movable, for example, a floating pontoon.

The centre of gravity of the rig should be as low as possible as many anchor holes are drilled at shallow angles. The necessary drilling thrusts cannot be applied safely unless the rig is stable.

The rig should be capable of drilling at any angle from horizontal to vertical and should be able to perform as many drilling methods as possible e.g. rotary and auger.

In the view of the authors, the following practical aspects may also merit consideration:

Noise: It is noticeable that there has been a recent swing away from the use of percussive or rotary percussive drills, to rotary drills in built-up areas. This is primarily due to noise restrictions and a noise level of 75dBA at 15m is now specified in urban areas. In 5-10 years it is anticipated that rotary percussive drifters will be banned in built-up areas. In future planning therefore it is recommended that consideration should be given to hydraulically powered rigs.

Nevertheless, whilst percussive drills continue to be employed it is important for engineers to appreciate that exposure to high noise levels, usually above 90dBA, for extended time periods can produce physiological damage to the ear. On many construction sites, particularly in the UK, warnings of this potential hazard to drillers seem in the main to go unheeded.

Versatility: All rigs should be designed to accommodate a rotary head, rotary percussive. drifter, vibrodriver and down-the-hole hammer. Where high production is required, mechanical handling of drill rods and casing could be advantageous and use of drill racks, rod-changing units and hydraulic positioners merits consideration.

Prime movers: All prime movers to operate rigs should be "built-in" to give a compact, independent unit. For the vast majority of anchor applications a power supply of 50-60 h.p. is considered sufficient.

Mast movements: A sub-mast is required capable of rotating 90 deg. in elevation i.e. vertical to horizontal. The main mast, attached to the sub-mast through a turntable/sliding carriage, should be capable of rotating 180 deg. in plan.

The ability to (*a*) position the toe of the main mast at the hole location, (*b*) hold the main mast at any level from 0-2m above the around is considered important.

Hoist and feed rating: Bearing in mind possible use of vibrodrivers in the future to cope with unconsolidated ground overlying rock, a maximum feed rate of 10m/min may be desirable. A satisfactory hoist rate is 3m/min.; acceptable hoist capacity = 35kN; and acceptable feed capacity = 25kN.

Ideally, pressure gauges giving a measure of torque and feed capacity during drilling should be incorporated in the rig. These gauges could be monitored by an experienced driller or engineer to highlight changes in the strata, and thereby improve quality control.

Exhaust pollution: In the future, attempts should be made to design and specify prime movers which emit "clean" exhaust.

In spite of the above recommendations, it is noteworthy that for anchors installed directly into rock the traditional wagon drill with a percussive hammer may still provide the most economical solution in some circumstances.

In general, the correct choice of a drilling method and machine for an anchoring project is a critical factor in the eventual successful completion of a project and therefore the greatest care should be exercised in making that choice.

Drilling rates

Since the rate of drilling holes in rock depends on the nature of the material drilled and the drilling machine, it is desirable to have as much knowledge as possible on both the rock and the machine.

Regardless of origin, all rocks may possess complex secondary structures, banding or foliation, and the degree of fracturing and weathering, and bedding of the rock mass can affect the physical properties and the drillability of the rock. Consequently, although average or typical properties can be established for sound, unweathered specimens of rocks, in practice each site tends to be evaluated individually, and purely geological classifications of rocks offer little help in grouping rocks according to drillability. On the other hand classifying rocks on the basis of their physical properties, such as compressive and tensile strength, Young's modulus, scratch and impact hardness, toughness and others, is a major factor in establishing a suitable drillability scale. Nevertheless, no definite conclusion has been reached as to which are the most useful physical parameters to determine, and no single property correlates perfectly with drilling rate, although rock compressive strength remains a popular and useful parameter in the hands of the specialist.

Most recently, van Ormer [1974] has attempted to relate penetration rate to rock mass and material properties, and considers texture (porous to dense fine), hardness (1-10 on the Moh scale), breaking characteristics (brittle to malleable) and geological structure (solid to laminated). In each case the first named in the range sustains a faster drilling rate than the other extremes. Table III summarises the data pertaining to hardness, and the drilling rate for various rocks relative to 1.0 (for solid, homogeneous Barre Granite) is shown in Table IV. The latter table does not take into account the secondary structure of the rock mass—the influence of which, it is claimed, is best determined from experience. Differences between measured and predicted drilling rates based on physical properties of the rock are probably due to the ever present variation of these properties throughout the length of hole. Although rock material and mass anistropy is known to affect drillability, little work has been carried out to quantity its influence. In view of its importance however some effects are sumarised by van Ormer in Table V.

Whilst solid formations should provide good drilling, seamy, broken formations induce slow rates as tedious, careful supervision is necessary to avoid loss of flushing capacity, loss of drill string, and bit sticking.

From the standpoint of the anchor contractor, one of the simplest procedures at present for predicting penetration rates, particularly in percussive and rotary-percussive drilling, is to determine the coefficient of rock strength of the rock to be drilled. The test, which was first described by Protodiakonov [1962] and subsequently modified by the U.S. Bureau of Mines (Paone *et al*, 1968), consists basically of fracturing rock samples by impacting them with a falling weight. The resulting damage is measured by screening the broken sample. The test is relatively simple, does not require elaborate equipment and

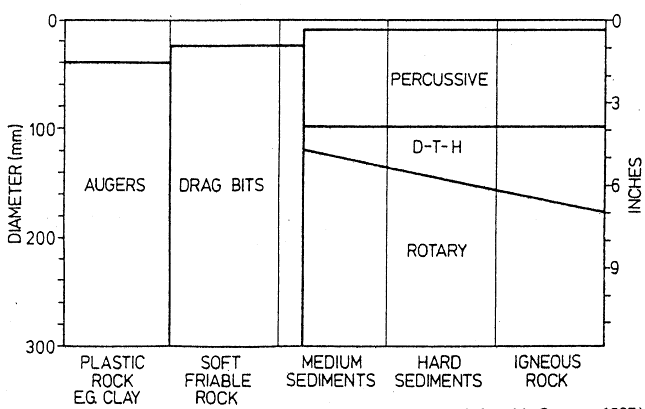


Fig. 1. Preferred methods of drilling different classes of rock and at different hole diameters. Depth of hole generalised

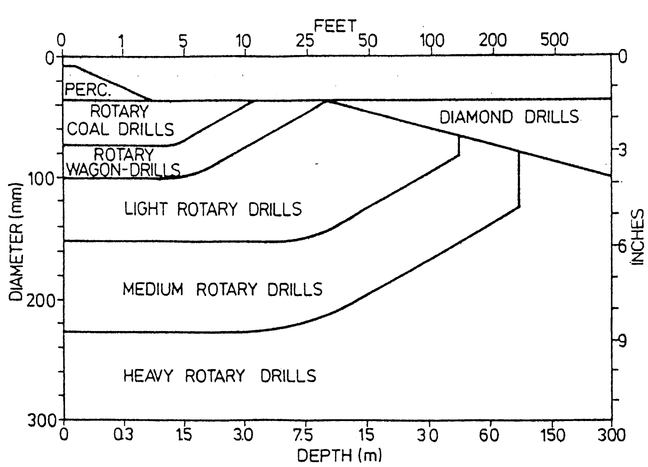


Fig. 2. Preferred methods in soft friable rocks (after McGregor, 1967)

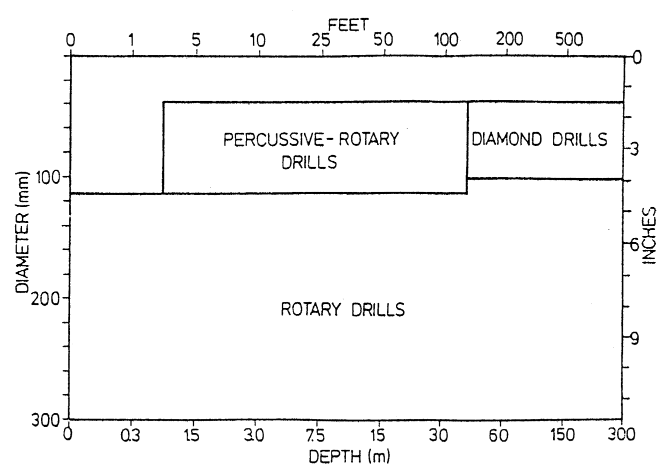


Fig. 3. Preferred methods in variable strata (after McGregor, 1967)

one man can carry the apparatus into the field and make several determinations in one day. Good results have been obtained in correlating field penetration rates with the coefficient of rock strength for rotary-percussive drills (Unger & Fumanti, 1972) and for percussive drills (Schmidt, 1972).

One major disadvantage, however, in using only the coefficient of rock strength for prediction is that no account is taken of drill power and machine characteristics.

Penetration rates, particularly for percussive drills, are a function of the air pressure supplied to the drill, the condition of the drill and the type and condition of the bits. Other technical factors such as flushing medium and bit diameter are also important, but to date have received little investigation. Since these parameters are usually difficult to measure with any degree of precision, especially in the field, it is not surprising that some discrepancies between calculated and measured rates are evident.

As a result it is now widely appreciated that a step that considers energy output of the drill must be included to further refine the procedure for predicting drilling rates. Whilst much work remains to be tackled Paone *et al* [1968] in a detailed account have already suggested a method of estimating penetration rate based on the quantity of energy required to cut a unit volume of rock and the energy output of the drilling system. It is also noteworthy that Paone *et al* [1969] have suggested using the coefficient of rock strength to determine the energy required to remove a unit volume of rock.

Flushing

It is vital to remove particles from the bit quickly and efficiently. Energy expended on grinding such fragments obviously cannot be used for hole production; comminution of the fragments also increases wear of the bit.

Commonly used flushing media are air, water or "mud"—usually being a colloidal suspension of bentonite in water. A distinction is also drawn between normal and reverse flush circulation. In the former, the flush is introduced via the rods and bit, and returns to the surface between the rods and the hole wall. In the latter, the opposite situation occurs.

Of the media listed, air is probably the most efficient scavenger, water the best coolant and mud the best lubricant. Air is the commonest fluid used for surface drilling with percussive machines, and with drag-bit and roller-bit rotary drilling in quarries. Air is best used in dry ground, although it can be used in very wet conditions provided ample air is available but offers little advantage over wet drilling. Underground, and in confined spaces generally, air is unsatisfactory unless used in reverse circulation, because of the health hazard of dust particles. Rock-drilling in confined spaces such as tunnels is therefore normally restricted to wet or suction drilling, the latter being one example of reverse circulation.

Water flushing is the standard method used for drilling in sticky ground (i.e. where there is a small inflow of water into the hole from the rock, only sufficient to combine with the cuttings to form a paste or where there are clayey layers), for drilling under the water table at depth, and for diamond drilling. The quantity of water used is not excessive—usually less than 4 litres per minute for conventional

TABLE III. HARDNESS OF SOME ROCKS AND MINERALS

Mineral or rock	*Hardness*	*Sratch test*
Diamond	10.0	
Carborundum	9.5	
Sapphire	9.0	
Chrysoberyl	8.5	
Topaz	8.0	
Zircon	7.5	
Quartzite	7.0	
Chert	6.5	Quartz
Trap rock	6.0	Quartz
Magnetite	5.5	Glass
Schist	5.0	Knife
Apatite	4.5	Knife
Granite	4.0	Knife
Dolomite	3.5	Knife
Limestone	3.0	Copper coin
Galena	2.5	Copper coin
Potash	2.0	Fingernail
Gypsum	1.5	Fingernail
Talc	1.0	Fingernail

Percussive drilling

Rotary drilling

(After van Ormer, 1974)

TABLE IV. DRILLING CHARACTERISTICS OF COMMON ROCKS

Characteristics	*Comparative drilling speed*	*Rock material*
Hardness— 1-2 Texture—Loose Breakage—Shatters	1.5 and up	Shales Schist Ohio Sandstone Indiana Limestone
Hardness— 3-4 Texture—Loose grained to granitoid Breakage—Brittle to shaving	1.0 to 1.5	Limestone Dolomites Marbles Porphyries
Hardness— 4-5 Texture—Granitoid to fine grained Breakage—Strong	0.6 to 1.0	Granite Trap Rock Most fine-grained igneous Most quartzite Gneiss
Hardness— 6-8 Texture—fine grain to dense Breakage—Malleable	0.5 and less	Hematite (fine-grained, grey) Kimberly chert Taconite

(After van Ormer, 1974)

Barre Granite is used as the standard for determining a comparative drilling speed of 1.0 because of its even texture, hardness, and consistent drilling.

TABLE V. EFFECT OF ROCK MASS STRUCTURES ON DRILLING RATES

Rock mass	*Nature of fractures*	*Drill rate*
Massive	——	Fast
Stratified	Perpendicular to drill rod; > 1.2m apart, clean	Fast Medium
Laminated	Perpendicular to drill rod; < 1.2m apart, clean	Medium
Steeply dipped	Small angle to drill rod, 1.2m apart, clean	Slow Medium
Seamy	Various inclinations to drill rod; close, open fractures	Slow

(After van Ormer, 1974)

anchor hole drilling. In spite of this wet drilling is often regarded as a messy and inconvenient method, whilst mud flushing is considered expensive and thought to require a great deal of preparation. Mud flushing is not common in rock anchor construction although it has been used successfully in France for open hole drilling through silts and sands overlying rock.

The type of flush employed may in cases improve the efficiency of hole formation. In weakly cemented sandstones for example, water flushing widens and cleans the hole and ensures a better bond at the grout/rock interface. However, in rock strata liable to deterioration from water action such as marls and chalks, water flushing where necessary should be kept to a minimum.

Regardless of the supposed efficiency of the flushing process, it is usual in anchor construction to leave a "sump" length for debris at the bottom of the borehole. In current practice, 0.3-0.7m is commonly added to the designed borehole length. After each hole has been drilled to its full depth and throughly flushed out in order to remove any loose material, the hole should then be sounded to ascertain whether "fall-in" or "blow-up" of material has occurred and whether it will prevent the anchor tendon reaching the required depth. If satisfactory the top of the hole should then be effectively plugged to prevent debris falling into it.

With regard to the logging of data relating primarily to ground water and flushing medium, it has been shown that local variations in ground conditions, over a few metres, can have marked effects on subsequent anchor performance—especially in soft rocks. Much qualitative data can be obtained on ground conditions by logging drilling rates and the degree of bit blocking, but a more sensitive record is often provided by observing changes in the amount and composition of flush return.

Other data relating to ground water, pressure and permeability can also be readily obtained if close liason is established and maintained with the driller. For example, the following should be noted:

(*i*) the depth at which ground water is first encountered in the hole,

(*ii*) any water added to the hole to assist drilling,

(*iii*) the level of water, and amount and diameter of casing in the boring at the end of the shift, and

(*iv*) the level of water when work recommences.

Alignment and deviation

In the drilling of rock anchor boreholes, it is important to maintain a true, straight hole, terminating in the expected, calculated position. Three causes of errors may be recognised:

(*a*) incorrect setting-up, with the drill pointing in the wrong direction at the start of drilling,

(*b*) misalignment, in which the drill is correctly lined up but the hole is out of line with the axis of the drill, and

(*c*) deviation in which the hole is started in the correct line but subsequently alters direction.

Correct setting-up of a drill is largely a matter of care and a good eye, but should always be aided by the use of a profile and spirit level. The use of a casing or drill rod guide plate at the base of the drill mast is advantageous.

TABLE VI. LIMITING FLOW RATES WHICH HAVE BEEN RECOMMENDED OR EMPLOYED TO DETERMINE THE NEED FOR WATERPROOFING

Source	Flow rate (recommended or employed)	Flow rate (gal/ft/min/atm)*
GERMANY		
(Brunner)	5 litres/min	0.00091
(Bomhard & Sperber)	1 litre/metre/min/10 atm	0.00670
SWITZERLAND		
(Moschler & Matt)	1 litre/metre/min/10 atm	0.00670
(Buro)	0.08 gal/ft/min/10 atm	0.00800
SOUTH AFRICA	0.075 gal/100 ft/min	0.00013
NEW ZEALAND	0.01 gal/ft/min	0.00167
AUSTRALIA	0.001 gal/in. dia/ft/min	0.00067
USA	0.001 gal/in. dia/ft/min (hole full plus 5 p.s.i.)	0.00063
MALAYSIA	0.003 gal/ft/min	0.00050
UK		
(Parker)	0.01 gal/ft/min	0.00167
(Falmouth)	0.25 gal/min	0.00021
(Devonport)	1 litre/metre/min/10 atm	0.00670

*Imperial units have been used in this table since the majority of references relate to contracts carried out prior to transfer to S. I. Units

Regardless of cause, misalignment is troublesome and can result in damage to the drill and string as well as causing jamming of the rods. Furthermore, McGregor [1967] notes that the rubbing of the rods on the wall of the hole may dislodge rock fragments whilst the resultant friction—especially in rotary drilling—can increase enormously the torque requirements. Resettlement of the rig when the drilling thrust is relaxed may also be a problem in soft ground and experience indicates that special care is required when drilling from free-floating platforms.

Deviation of the hole during drilling does not normally arise from a single circumstance. It may originate by using too thin rods, from excessive thrust, or by the bit following a fissure or other rock planar structure. Deviation is not usually a serious problem for DTH drills, but is exaggerated by the hole length in diamond drilling.

The above remarks have been primarily related to vertical downward-holes. With angled holes, the rods are apt to lie on the lower side of the hole and this has the effect of upturning the bit slightly. Hence angle holes often—but not invariably—tend to follow a shallow curve away from the vertical.

Wherever possible, drill holes should be planned so that they intersect the major rock discontinuities at as high an angle as possible. If this rule is not observed, then it is probable that a proportion of the holes will tend to deviate along the planes of the rock. In mica-schist for example, holes will follow the mica defined schistosity if originally drilled at, say, a 5 deg. angle to it.

It is therefore essential to set-up the drill with the greatest care and precision and to monitor the progress of the hole. It becomes progressively more difficult and costly to alter the direction of the hole after drilling has proceeded beyond a few metres.

Little guidance on maximum permitted deviations has appeared, but tolerances of 0° 28′ (Parker, 1958), 1° 10′ (Eberhard and Veltrop, 1965) and 0° 43′ (Littlejohn and Truman-Davies, 1974) may be compared with the less rigorous maximum of 2° 30′ permitted by the South African Code. Contractors often quote average deviations of 1 in 50 i.e. 1° 09′ and tolerances are usually relaxed in the fixed anchor zone. (Tolerance is measured as a deviation of anchor hole from the specified centre line divided by the length of drill hole).

A common method of inexpensively checking the deviation in a vertical hole is to lower a torch down it and observe by how much, if at all, the face is obscured at various depths. Alternatively, the deviation may be more accurately checked at regular intervals using a single-shot photographic or continuous reading borehole inclinometer.

WATER TESTING AND WATERPROOFING

On completion of drilling, the anchor borehole must be tested for "watertightness", since subsequent loss of grout from around the tendon in the fixed anchor zone is of prime importance in relation to efficient load transfer and corrosion protection. Reasonable threshold values for water loss or gain must be assessed which, when exceeded, dictate the need for waterproofing. In practice, it has been generally accepted that cement is not suitable for the treatment of fissures which are less than 250 microns wide although recent experimental studies suggest that the lower limit is closer to 160 microns for Ordinary and Rapid Hardening Portland Cements.

The authors believe that a logical approach is to establish the minimum width of fissure which will permit flow of cement at low pressure. The water flow per atmosphere which is caused by a single fissure of this width may then be specified as a threshold value which dictates the need for waterproofing.

It may be estimated that a single 160 micron fissure under an excess head of one atmosphere gives rise to a flow rate 3.2 litres/min (Littlejohn, 1975). It is therefore suggested that this order of flow should be considered as a reasonable threshold for water loss when Ordinary Portland Cements are employed in the neat cement grout. A lower fissure width of 100 microns gives a flow rate of 0.6 litres/min/atm. and this may be a more realistic threshold for minimal penetration when fine-grained cements are employed.

With regard to rock anchor practice, the magnitudes of water flow which have been permitted in various countries to date are listed in Table VI.

Clearly, great care must be taken in the interpretation of limiting flow rates, with particular regard to the length of section being tested. To avoid serious misinterpretation, it is recommended that permissible flow rates should be quoted simply in terms of litres/min/atm, no reference being made to flow per unit length of hole or stage.

In general, it is considered that water tests carried out over sections e.g. the fixed anchor, with the aid of packers are preferable to rate-of-fall tests carried out under atmospheric pressure from the surface, since more detailed information can be obtained over specific locations. Packer testing is not essential however and on many occasions rate-of-fall tests can be carried out more cheaply and quickly. In these situations packer testing may only be warranted if the acceptable water flows are exceeded.

On the practical side the hole must be thoroughly flushed with clean water from the bottom before testing, and during the test it may be of value to reduce the level of water in any adjacent holes so that any interhole connections may be more easily detected.

From a review of current world practice, it is clear that water-testing is not a routine procedure and even when waterproofing is carried out, generally acceptable water flows have not been established for rock anchor grouting. As a result, the following recommendations are presented for consideration.

(*a*) Waterproofing is required if leakage or water loss in an anchor borehole exceeds 3.0 litres/min/atm. The duration of the test should not be less than 10 minutes and in terms of the Lugeon coefficient the above flow is equivalent to 10*L*.

(*b*) Where there is a measured outflow or water gain (under artesion conditions) care should always be taken to counteract this flow by the application of a "backpressure" during the grouting stage. If the flow cannot be stabilised in this way waterproofing is required, irrespective of the magnitude of the water gain.

(*c*) Permissible flow is related to "excess head". Therefore the position of the water table in relation to the section being investigated must be established so that the driving or excess head inducing flow at the section may be calculated accurately. In fine fissures high applied pressures may induce turbulent flow, create high pressure gradients and open up the natural fissures. As a principle, changes in the local environment should be minimised and therefore the applied pressure inducing flow should be as small as possible.

(*d*) The flow rates in (*a*) are minimum values since they all pertain to single fissures. Clearly, larger limiting flow rates are acceptable if a number of fissures (thickness $<$ 160 microns) exist. This situation however must be confirmed by close examination of the borehole interface using a camera or close circuit television and/or multipacker injection tests.

In order to waterproof the hole against water loss, grout should be tremied into the hole from the base upwards. After a period of time (usually from 6 to 24 hours) the hole is redrilled and the water test repeated. The anchor construction proedure may only continue when the waterproofing criteria are satisfied. If the pregrouting is not successful on the first one or two occasions, then pressure grouting may be required to force the grout into the fissured rock mass and thereby stabilise the borehole wall against subsequent redrilling.

TENDON

Storage and handling

Longbottom and Mallett [1973] make a number of sound recommendations regarding this topic, on the basic assumption that anchor tendons must be protected against mechanical damage and severe corrosion on site.

Tendons must not be dragged across abrasive surfaces or be accessible to weld splash. Bars should be stored in straight lengths, and wires and strand in coils of diameter at least 200 times that of the tendon diameter. Kinked or twisted wire should be rejected, since experience has shown that bond and load/displacement characteristics can be adversely affected.

To avoid damage to protective sheathing, the ends of the tendon should be treated, after cutting to size, to remove very sharp edges. With respect to bars, care should be taken to protect the threads. Superficial damage to the threads can often be repaired by means of a file, but it is usually impracticable to recut or extend a bar thread on site because of the hardness of the steel.

Ideally, steel for anchor tendons should be stored indoors in clean, dry conditions. If this is impossible, the steel may be left outdoors for several months without serious corrosion, provided it is stacked off the ground and completely covered by a waterproof tarpaulin. Although the tarpaulin should completely cover the steel it should be fastened so as to permit circulation of air through the stack.

The humidity of the air, allied to possible atmospheric pollution (industrial and marine) is the major cause of corrosion during storage. There would appear to be little problem if the relative humidity is always less than 70 per cent, but severe corrosion occurs at levels in excess of 85 per cent. The worst conditions are experienced in marine tropical areas, where the average rate of corrosion is about three times that in a heavy industrial area in the UK. In such areas, wrappings should be impregnated with a vapour phase inhibitor powder, and in this case air through flow must be prevented.

Although it is known now that normal rusting actually improves the bond to grout, flakey, loose rust must be completely removed, and tendons which are severely pitted, particularly in the case of small diameter multi-wire strands, or at threaded sections of bars, should be rejected.

Fabrication

With respect to bar anchors, all threads must be thoroughly cleaned and lightly oiled, and it is important to ensure that bars are properly screwed into couplers, and that full thread engagement is obtained in nuts and tapped plates. To minimise corrosion, the tendon should not be left ungrouted for long after cleaning, especially if paraffin has been used.

Anchors with multi-strand or multi-wire tendons usually require more time for fabrication. If the strand is supplied already coated in PVC, then great care should be taken to degrease the intended fixed anchor length effectively, using solvents such as acetone, trichloroethylene or paraffin. Some contractors specify unravelling of the strand to facilitate effective cleaning; the wires are afterwards returned to their correct lay. This basic method is recommended and an efficient, if somewhat time-consuming refinement to the system has been developed by U.A.C. Ltd., who introduce small ferrules on to the central wire prior to relaying the strand. This produces nodes in each strand and undoubtedly increases the resistance to the strand-grout failure. Altrnatively, to eliminate the laborious and inherently risky job of attempting to completely remove a graphited bituminous grease which has been designed to resist easy removal, a machine has recently been developed (Littlejohn and Truman-Davies, 1974) to grease each individual strand and apply a protective plastic sheath only over the free length where it is required.

The fixing and location of spacers and centralisers must be done with care and precision, especially in the fixed anchor length where the tendon is usually formed into a roughly circular configuration with steel or polythene spacers and wire bindings. Attention should also be given to the bottom of the tendon and use of a sleeve or nose cone which will minimise the risk of tendon or borehole damage during homing is recommended.

Homing

Any method can be used provided that it will ensure that the tendon is lowered at a steady controlled rate. It is recommended that for heavy flexible tendons of total weight in excess of 200kg, mechanically operated pulleys or large drums (Littlejohn and Truman-Davies, 1974) be used to gradually unreel the tendon into the hole. It has been found that 200t capacity anchors, weighing about 16kg/m, are the largest that can be handled in restricted areas, e.g. dam crests, without elaborate handling equipment.

If the borehole grout is preplaced under water, grout dilution can occur if the tendon is lowered too quickly. The use of drums from which to unwind the tendon into the hole is preferable to the use of cranes, or (for vertical anchors) manhandling, as both these methods often create sudden bending of the tendon which may damage both steel and protection.

Immediately prior to homing, the tendon should be carefully inspected, and in certain situations the efficiency of the centraliser/spacer units may be judged by carefully withdrawing the tendon—prior to grouting—to observe damage or distortion, or the amount of smear.

In general the choice of the best methods of storage, handling, fabrication, and installation of anchor tendons is wholly an exercise in commonsense. Prestressing steel and fittings are valuable stores, and should be treated as such on site.

The most common and lowest basic cost material used for fixing and protecting rock anchors is neat cement grout. The influence of certain grout parameters on bond development has already been noted (Littlejohn and Bruce, 1975) and information on grout mixes and grouting procedures as used in rock anchor practice is now reviewed, and recommended quality controls are discussed.

Grout composition

Cement

The type of cement used will obviously vary from contract to contract as dictated by ground conditions and the installation programme. Thus, while Ordinary Portland Cement (Type I) may suffice in many cases, a sulphate-resisting (Type II), or a rapid hardening variety (Type III) may be required. In Britain, Ordinary and Rapid Hardening Cements must comply with BS 12 and High Alumina Cement with the relevant clauses of BS 12 and 195. It is recommended that high alumina cement be restricted to short term test anchors, in view of the use of high water cement ratios often necessary for pumpability.

Since cement surface areas (and therefore particle sizes) are normally controlled by specification, the most likely deterioration in cement quality may be due to age or poor storage, when partial dehydration or carbonation may lead to particle agglomeration and reduction in post-mix hydration. Although large sizes may be removed by sieving, it is likely that better control may be exercised by insisting on fresh cement, and by careful storage. Ideally cement should not be stored on site for more than one month, and must be kept below 40 deg. C, under cover. Cement should be used in order of delivery.

Water

Water which is suitable for drinking (except for the presence of bacteria) is generally considered suitable for cement grout formulation. Water containing sulphates (> 0.1 per cent), chlorides (> 0.5 per cent), sugars or suspended matter e.g. algae must be considered technically dangerous. High chloride content should be particularly avoided where the steel tendon is in contact with the grout.

Where there is some doubt as to the quality of the water, a test on the lines of BS 3148 "Tests for water for making concrete" may be carried out.

Water-cement ratio (w/c)

The proportion of water to cement in a grout rather than the quality of water is the most important determinant of grout properties. Excess water causes bleed, low strength, increased shrinkage and poor durability. The extent to which these (and also fluidity) are related to the w/c ratio of an OPC grout is shown in Fig. 4.

Table VII has been prepared to illustrate a range of w/c values recently used or recommended throughout the world, for neat cement grouts. Most ratios are between 0.40 and 0.45 which gives a grout with sufficient fluidity to be pumped and placed easily in small diameter boreholes, and yet retains sufficient continuity and strength after injection to act as a waterproofing and/or strengthening medium.

Admixtures

The use of inert "fillers" such as ground quartz, limestone dust, fine sand, clay, and even sawdust, has long been common, particularly in Europe. The resultant mixes have been used primarily to waterproof

TABLE VII. RANGE OF W/C RATIOS RECENTLY USED OR RECOMMENDED

W/C ratio				
Ordinary Portland	Rapid Hardening	High Alumina	Remarks	Source
0.4			Anti-bleed admixture permitted	Maddox et al (1967)
	0.45		U.A.C. anchors	Anon (1969)
	0.3		Anchors in Keuper Marl	Cementation Co Ltd. (1969)
0.4	0.4	0.4	Recommendation	Mullett (1970)
0.4–0.45	0.4–0.45		Fluidifier permitted	Buro (1970)
≃ 0.45			"Intrusion aid" permitted	Thompson (1970)
0.4			—	Gosschalk and Taylor (1970)
	0.46		Expanding agent required	Barron et al (1971)
0.38–0.43			Recommendation	Conte (1971)
0.4	0.4	0.35	Recommendation	Littlejohn (1972)
< 0.45	< 0.45	< 0.45	Recommendation	C.P. 110 (1972)
0.35–0.4			Admixtures permitted	Bureau Securitas (1972)
< 0.45	< 0.45		Recommendation	Mascardi (1972)
0.4	0.4		Recommendation	Ontario Hydro (1972)
≤ 0.5			Recommendation	South African Code (1972)
0.36–0.44			Recommendation	Stocker (1973)
0.38–0.44	0.38–0.44		Recommendation	Hilf (1973)
0.45	0.45		Recommendation	White (1973)
0.4–0.5			Expanding agents or retarders permitted	Golder Brawner (1973)
	0.45		—	Littlejohn and Truman Davies (1974)
0.4–0.45	0.4–0.45	0.4–0.45	—	Ground Anchors Ltd (1974)

TABLE VIII. COMMON CEMENT ADMIXTURES FOR ANCHOR GROUTS

Admixture	*Chemical*	*Optimum dosage (% of cement by weight)*	*Remarks*
Accelerator	Calcium Chloride	1—2%	Accelerates set and hardening
Retarder	Calcium Lignosulphonate	0.2—0.5%	Also increases fluidity
	Tartaric acid	0.1—0.5%	May affect set strengths
	Sugar	0.1—0.5%	
Fluidifier	Calcium Lignosulphonate	0.2—0.3%	
	Detergent	0.5%	Entrains air
Expander	Aluminium powder	.005 — .02%	Up to 15% expansion
Anti-bleed	Cellulose Ether	0.2—0.3%	Equivalent to 0.5% of mixing water
	Aluminium Sulphate	up to 20%	entrains air

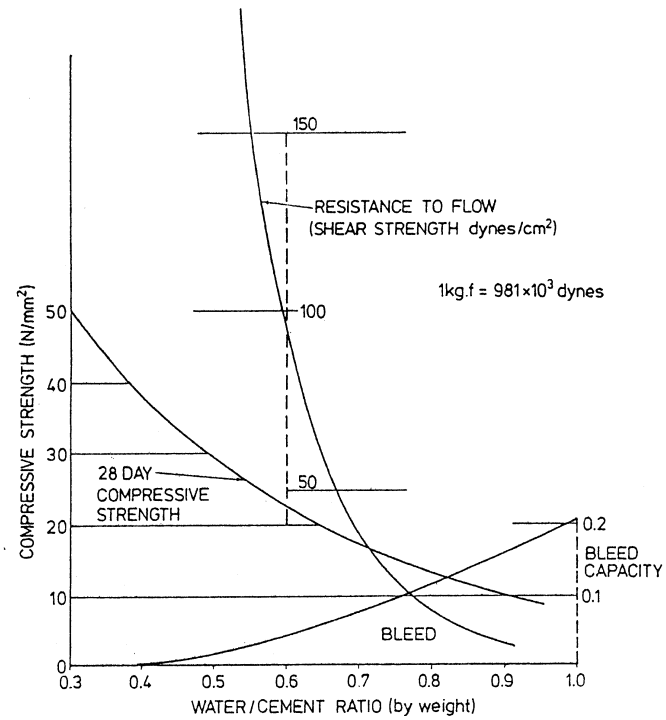


Fig. 4. (above). Effect of water content on grout properties

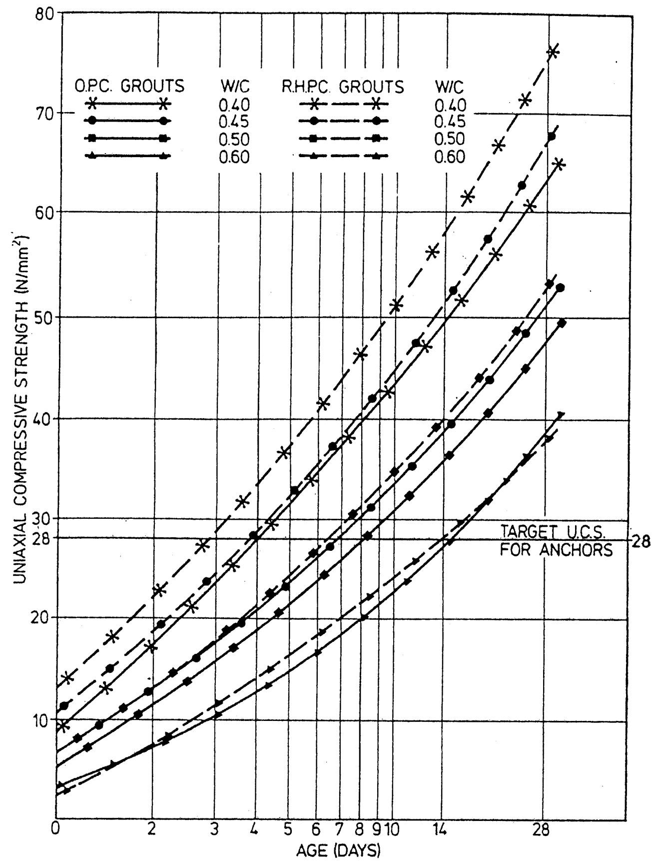


Fig. 5. (right). Gain in strength of set grouts

or consolidate boreholes prior to redrilling —a role in which neat cement grouts may be uneconomic. Such fillers are seldom employed however in grouts used for tendon bonding.

With respect to anchor grouts, chemical admixtures have often been employed particularly those to prevent shrinkage, to permit a reduction of the w/c ratio while ensuring fluidity, to accelerate or retard setting, and to prevent bleeding which in turn discourages corrosion. Table VIII lists common types of admixtures employed in grouts. Care should be taken however to ensure that the basic grout materials are compatible and except under carefully considered and controlled conditions, different types of admixture should not be included in the same grout. For example, admixtures such as calcium chloride should not be used with sulphate resisting, supersulphate or high alumina cement. Calcium chloride can also corrode steel in contact with the grout and to avoid this potential hazard the authors recommend that use of this admixture should be banned in anchor grouting.

Geddes and Soroka [1964] conclude that aluminium-based expanding agents improve grout workability while increasing the "confined" compressive strength (i.e. where expansion has been restrained on setting). This latter effect increases the bond capacity of the grout which has been illustrated experimentally by a reduction in bond transmission length. Leech and Pender [1961] have also favoured the use of aluminium powder in an amount of 0.005 per cent by weight of cement and they stipulate that bleeding was also inhibited. Pender *et al* [1963] advocate that a 2 per cent expansion of grout volume is desirable: this figure can be attained by using 0.002-0.005 per cent aluminium powder. However, a warning on the use of aluminium powder has been sounded by Moy [1973]. While confirming the findings of Leech and Pender, he emphasises the great sensitivity of grout mix properties to the amount of aluminium powder added —and its efficiency of dispersion and mixing. For example, slightly larger dosages of powder can give a markedly spongy and crumbly grout.

In Britain, some success has been achieved with calcium lignosulphonate as a grout fluidifier, when used at a concentration of 0.03 per cent by weight of cement. In this way a pumpable low w/c grout—0.3—can be satisfactorily produced for anchors, installed in water sensitive marls and shales.

In rock anchoring, grout bleed seldom receives consideration despite its great importance in corrosion protection. Antibleed additives based on cellulose ethers have been successfully employed (e.g. Maddox *et al*, 1967: 0.2 per cent by weight of cement), although slightly lower grout crushing strengths and higher initial grout viscosities result. They found from field tests that the final mix gave negligible settlement at the top of the tendon, and complete grout cover free from fissures or water filled lenses. Commercial products are readily available and Celacol M5000DS and Methocel 65HG4000 are recommended for consideration. Dosages are normally expressed as a percentage of the mixing water, rather than the cement, and vary according to the viscosity grade of the material. For example Celacol M5000DS and equivalent grades are normally added at a rate of 0.4-0.5 per cent by weight of water.

In general, considerable international agreement on the use of admixtures is apparent. For instance, the use of chloride bearing compounds is banned in Britain, Germany, France, Switzerland, Italy and the United States. CP 110 stipulates that admixtures may be permitted only when "experience has shown that their use improves the quality of the grout". Nitrates, sulphides, and sulphates are also banned, and total expansion should not exceed 10 per cent.

In Germany, the use of any additive is rare, and only those which increase workability of the grout are employed. Mascardi [1973] states that in Italy moderately expanding additives are used but air entraining or metallic expanding types are banned, as are rapid hardening agents.

Hilf [1973] considers that sand, and antibleed and expansion agents are acceptable in the United States, whereas White [1973] discourages the use of anything other than cement grouts. A very comprehensive survey of grout admixtures has been prepared by the American Concrete Institute [1971], and is recommended to the interested reader.

In summary, it may be concluded that the use of admixtures for grouts is still very much an art. Even the manufacturers have relatively little practical experience of their use for rock anchoring. Consequently, whenever a new mix is designed or adopted, the following must be recorded:

(*i*) water/cement ratio,
(*ii*) admixture concentration,
(*iii*) flow reading (through flowmeter, flow cone or viscometer),
(*iv*) crushing strengths (two cubes each) at 3, 7, 14 and 28 days, and
(*v*) notes on amount of free expansion or shrinkage, bleed and final setting time.

Even if the design is satisfactory, unless the cement and admixture is delivered on site ready mixed, very careful supervision of the grout mixing personnel is essential. Hence the general indication is that admixtures should be used only where absolutely necessary.

Grout crushing strength

Some grout properties have already been alluded to—pumpability, slight expansion on setting, a minimum w/c, and resistance to bleeding. In addition, the crushing strength requirements are of fundamental importance.

CP 110 states that grout used for prestressed concrete work must have a compressive strength in excess of $17N/mm^2$ at 7 days. Normally higher strengths are specified for stressing, and Littlejohn [1972] finds that $28N/mm^2$ is favoured in Britain. A survey of world practice reveals that this figure is in fact common in many countries, although Mascardi (Italy) feels that $35N/mm^2$ is necessary ($w/c < 0.45$) whilst PCI [1974] recommends a minimum value of $24N/mm^2$.

It is noteworthy that Thompson [1970] describes how satisfactory anchors were installed at the John Hollis Bankhead Dam, Alabama, with a grout of 28 day strength of $17N/mm^2$. However, this serves as a reminder that low strength grouts are only acceptable in rigid, competent rocks where "arching" mechanisms of the particulate grout can be mobilised, whereas high strength grouts are necessary in soft, yielding rocks.

In general a major disadvantage of cement grouts, even when admixtures are used, is the time required for the grout to develop full operational strength (see Fig. 5). Other problems are associated with its low tensile strength, brittle nature, and installation in adverse conditions. However, where time and bond length are not restricting factors—especially where large annular volumes are involved—no economic substitute to cement grout is available.

Mixing

The authors recommend that to ensure good practice, the following fundamental points should be observed.

1. The cement (and fillers where applicable) must be measured by weight.
2. Water should be added to the mixer before the cement (and fillers) and any admixtures should be added with great care usually during the latter half of the mixing time.
3. Although the mixing time depends on the type of mixer, the total time should not be less than 2 minutes according to CP 110.
4. Mixing by hand is to be strongly discouraged.

The equipment must be able to produce grout of uniform consistency, and should have two drums or tanks: one for mixing, the other for storage and delivery. In order to avoid heating of the grout, slow agitation only is permissible in the storage tank.

Rate of shear during mixing is particularly important and it is noteworthy that the most common type of grout mixer, comprising an impellor in a tank, combines two major effects which influence the efficiency of mixing—circulation and fluid shear. These are essentially incompatible, since a large slowly rotating impellor will produce a high circulating capacity and low shear rate, while a small rapidly rotating impellor will yield a high shear rate and low circulating capacity. For cement grouts of low w/c ratio shear rate is a critical factor in mixing and ideally impellor speeds of 1 500-2 000 rpm are required. In this connection an ideal type of mixer is the Colcrete double drum mixer which circulates the grout through a centrifugal pump. The grout is recirculated through a zone of high shear with sufficient impact to break down lightly bonded clusters or agglomerates, and provide maximum interdispersion of water and cement.

Where conventional paddle mixers are employed, field analysis indicates that the best results are obtained when the paddles are cut with slots, and where slotted baffle plates are fitted around the perimeter of the tank or drum.

Experience suggests that the actual mixing in the field is generally satisfactory, but that often the strainer between the two tanks is too small or easily clogged. In such cases, unstrained and lumpy grout overflows into the delivery tank and thence into the borehole. In addition exit points should be fitted at the base of tanks to avoid formation of cement cake at the bottom.

The use of rapid "snap-off" couplings permits the quick removal of obstructions which tend to form in bends of flexible pipes or at constrictions. It is noteworthy that rigid steel pipes do not allow the position of the obstruction to be quickly ascertained.

Finally it is an elementary yet important observation that a high standard of cleanliness of grout mixing and pumping equipment is usually associated with simpler and more efficient grouting operations.

Grouting methods

There are basically two distinct modes of anchor grouting, namely by two-stage or single-stage injection.

Two-stage grouting involves first injecting a "primary" mix to effect the bond between tendon and rock. After final stressing, a "secondary" phase is introduced, largely for the corrosion protection of the free length. In the one-stage system, both functions of the grout are simultaneously performed.

In two-stage injections the primary grout may be preplaced or postplaced with respect to the introduction of the tendon. Postplacing can be advantageous when dealing with large tendons and poor "slabby" rock, and is the only choice for very shallow or upwards-inclined anchors.

It is good practice to ensure that the primary grout extends for at least 2m above the designed fixed anchor length. This inhibits crack formation in the proximal end of the anchorage during stressing. Where the primary grout is preplaced, the tendon should be homed within 30 minutes of the injection. Even after the tendon has been correctly homed, problems have been experienced with grout/tendon bond development and opinions currently differ as to whether the tendon should be left static after homing (FIP, 1973) or vibrated (Standards Association, Australia, 1974).

Secondary grouting is usually accomplished with a mix of the primary composition although Mitchell [1974] recommends that to ensure complete freedom of tendon movement, an American practice of backfilling the free length with sand, sand and gravel, weak grout, or stone chippings, should be adopted.

At the present time the two-stage system is more common in practice, but has certain disadvantages:

(a) an additional interface is created at the top of the fixed anchor and is considered to be a prime target for corrosive agencies,

(b) the exact quantity and quality of the vital primary batch is difficult to judge without careful checking, and

(c) a two-stage method is intrinsically more time-consuming and laborious.

Single-stage methods are free from these problems. However it must be noted that unless the free tendon length is meticulously greased before sheathing all the load applied at the head will not be transmitted to the intended anchorage zone due to friction in the free anchor length.

On the practical side, before grouting commences, it is advisable to check the airtightness of all pipes involved, and the tremie pipe—flexible and usually 12-25mm in diameter—should be blown and flushed with water.

Both hole and tendon should be thoroughly water-flushed from the bottom upwards for at least 10 minutes prior to grouting. If the grout is to be postplaced the tremie pipe may be conveniently incorporated in the tendon, but terminating at least 150mm from the foot.

Grout should be tremied at a steady rate, and the pipe, if not incorporated in the tendon, may be withdrawn slowly during the operation. At no time must either the end of the tube be lifted above the surface of the grout or the level of grout in the pump storage tank be allowed to drop below that of the exit pipe, otherwise air may be drawn into the grout placed.

In the single-stage method or during the secondary phase of a two-stage injection, grouting should continue until grout of the same composition as that mixed has been emerging from the hole for at least 1 minute.

The Australian Code recommends that it is preferable to provide a standpipe during grouting so that grout shrinkage will occur in this pipe and not in the hole. In any case it is traditionally regarded as good practice, particularly in relation to dams, to "top up" anchor holes where necessary, a few days after the major grouting operation.

Grouting pressures

The general conclusion amongst specialist contractors is that high grout pressures are completely unnecessary for successful anchors in intact rock but useful for anchors in badly fissured rock. Analysis of the data received suggests that grouting

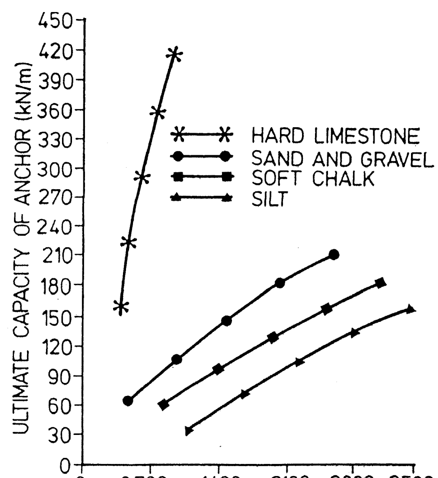


Fig. 6. Anchor resistance related to grouting pressure (after Soletanche, 1970)

TABLE IX. GROUTING PRESSURES RECOMMENDED AND USED FOR ROCK ANCHORS

Grouting pressure (N/mm²)			Source
Upwards sloping anchors	Downwards sloping anchors	Either	
		3.5	Holz (1963)
		0.5—2	Broms (1968)
	0.1		Buro (1970)
		0.69	Irwin (1972)
0.3	0.2—0.3		Conti (1972)
0.207—0.345	0.345—0.522		Koch (1972)
		< 2.5	F.I.P. (1973)

pressures normally lie in the range 0.28—0.70N/mm².

A.T.C. Ltd. (1973) experimented with different injection pressures when grouting anchors in chalk and concluded that there is no real benefit in employing grouting pressures of the order of 4N/mm². Practical and economic considerations often set the maximum grouting pressure at 3N/mm², and for the subsequent contract anchors, a pressure of 2N/mm² was used. Fig. 6 illustrates the relation of grout pressure to averaged anchor capacity, claimed for Soletanche "Tamanchor" (I.R.P.) system anchors. Others grouting pressures which have been used in practice are shown in Table IX.

It can be summarised that permissible grouting pressures are largely a matter of conjecture. They depend on the circumstances and geology of the site and "rules of thumb" should be proven at each site by in situ water or grout pumping tests, before being put into general use. As a starting point the most common rule for permissible pressure appears to be 0.023 N/mm² per metre of overburden.

Quality control

Variations in grout properties arise from three principal causes:

(*a*) inadequate mixing,

(*b*) variations in grout materials quantities and quality, and

(*c*) apparent variations arising from the testing procedure.

In order to obtain a satisfactory basis for grout mix design it is essential, prior to any anchor contract, that methods of storage, batching, mixing and testing of materials be rigidly defined and adhered to.

Mixing of cement grouts

Contact between cement and water leads to a prolonged sequence of exothermic reactions leading to complete hydration and ultimately final setting of the cement-water paste. There are normally four stages to this reaction:

(*i*) an initial highly exothermic reaction lasting 5-10 minutes,

(*ii*) a dormant period lasting up to 2 hours during which there is a low rate of heat evolution,

(*iii*) an increasing rate of reaction leading to final set after 6 or more hours, and

(*iv*) a continuing decreasing rate of reaction after setting.

During the dormant period, a cement grout should maintain a consistent physical state, when its properties can be measured and predicted. In order to obtain this consistent physical state when the cement is added to the water, sufficient mechanical agitation must be induced to fully disperse the cement grains. To achieve this and at the same time avoid false sets, mixing for a period of 5-10 minutes is normally required. Under most field applications this should be achieved by agitation during storage, and pumping and placement after mixing.

Variation in grout quality

Variations of material quantities and qualities from those specified in the grout design are largely a reflection of the standards of site organisation, equipment and supervision and as such are difficult to quantify. Neville [1963] has attempted however to define the quality of concrete mixes by relating the coefficient of variation of cube strength to the degree of site control, and it is considered that these standards (Table X) could apply to cement grouts.

TABLE X. VARIATION OF CONCRETE STRENGTHS

Degree of site control	Coefficient of variation = *Standard deviation* / *mean strength*
Best laboratory control	5
Best site control	10
Excellent	12
Good	15
Fair	18
Bad	25

(after Neville, 1963)

The best possible results obtainable when site control' approaches laboratory precision should have a coefficient of variation of 10. This will require:

(a) Obtaining cement, fillers and chemical admixtures from a reliable source,

(b) Storage of cementitious materials under dry and constant conditions,

(c) Accurate determination and monitoring of moisture content of fillers,

(d) Use of cement in fresh condition,

(e) Weigh batching of all materials (meter for water is acceptable),

(f) Controlled water/cement ratio,

(g) Adequate mixing rate and time of mixing,

(h) Immediate pumping and injection of grout after mixing, and

(i) Rigid supervision of all operations.

In practice the cement grout is expected to fulfil the dual role of fixing the anchor to the rock and protecting it against corrosion, often in "aggressive" environments. It is surprising, therefore, that the only common method of checking quality is by crushing a nominal number of cubes after the anchors have been constructed. Furthermore, samples are often carelessly taken, or not taken for every anchor.

Additional measurements are therefore recommended which permit the quality of the grout to be assessed before the grout is injected, thereby pre-empting the possibility of potentially expensive and/or dangerous errors occurring.

Measurement of important grout properties

Accuracy of measurement of grout properties is an important factor in determining the variability of grout properties in the field. Some property measurements, such as bleed, have been developed principally as laboratory measurements, for example, Powers float test and the ASTM method (see Powers, 1968). In the field, levels of bleed above 0.5 per cent are relatively easily detected in any sample contained in a wide, low container, and in anchors the actual magnitude of bleed is less important than the fact of its existence.

Laboratory measurements of grout fluidity in terms of shear strength and viscosity are normally carried out with a rotating disc or coaxial cylinder viscometer. Two instruments which are commonly used in the field are the Colcrete flowmeter (which expresses fluidity in terms of horizontal slump) and the Portland Cement Association cone (in terms of flow time). Various specialists and researchers have calibrated these instruments in terms of standard grout parameters e.g. w/c ratio, but for particular grouts it is the authors' view that the most direct information on fluidity is still best obtained from field pumping tests. Nevertheless flowmeter and flow cone data can be useful in assessing efficiency of mixing.

Check measurements of water/cement ratio can be made on site by measuring the specific gravity of the grout using a Baroid mud balance (see Table XI). Hydrometers are not recommended since at low water/cement ratios larger errors are introduced due to the thixotropy and solid structure of the grout.

TABLE XI. CALCULATED SPECIFIC GRAVITIES OF WATER/CEMENT GROUTS

Specific gravity	*Water/cement ratio*
2.10	0.3
1.95	0.4
1.84	0.5
1.74	0.6
1.67	0.7
1.61	0.8
1.56	0.9

In most grouts the hydrogen ion concentration is of value as an indicator of chemical contamination; pH is therefore another parameter which can be a useful control in practice and where a large number of site tests are planned, a battery or mains pH meter can be used.

With regard to the strength development characteristics of cement grouts, Fig. 5 indicates the curing times required by a range of grout mixes made from Ordinary Portland and Rapid Hardening Cement to attain the minimum strength of 28N/mm² before stressing. The results were obtained from 150mm grout cubes but 75mm cubes should give reliable results in practice. Care must be exercised, however, when attempting to correlate 75mm and 150mm cubes strengths. On demoulding, the larger cubes are invariably warmer, even when efficiently cured. In addition, the curing water takes longer to influence the centre of the larger cubes. Both these phenomena act to increase the early strength (1-7

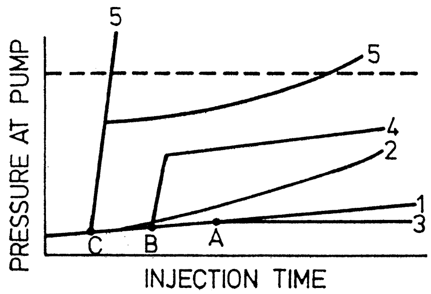


Fig. 7. Idealised representation of various grout injection time-pressure characteristics
(after Longbottom & Mallett, 1973)

days) but tend to depress the later strength of the larger cube sizes.

The recommended controls, including bleed measurements where corrosion protection by grout is vital, can be readily exercised during the actual grouting operation, which ideally should always be carried out the same day the fixed anchor section of the hole is drilled.

To ensure that injection pressures do not cause undue disturbance of the ground, the pump should be fitted with an effective control against pressure build-up. Pressure and pump speed may be considered as one control: the balance between the two is dictated by actual conditions. In this connection pressure gauges fitted with diaphragms are recommended to avoid contact with the grout. Pumping over distances in excess of 150m is strongly discouraged, as this can change the grout properties.

Monitoring the grout pressure during injection can provide useful information about the quality of the grout being pumped, and the efficiency of the operation. Idealised curves (see Fig. 7) for grouting progress are described by Longbottom and Mallett [1973].

Curve 1—Good grout, normal stiffening—"standard" mix.
Curve 2—Gradient greater than standard, possibly indicating that the grout is stiffening too quickly.
Curve 3—Indicates a fracture in the system at time *A*; leaking of the grout indicated by constant pressure.
Curve 4—Indicates partial blockage at *B*.
Curve 5—Serious blockage at *C*, possibly with stiffening. If the maximum pressure is exceeded, grouting should be stopped, and the system flushed.

It is concluded that problems associated with the crucial grouting operation will be eased if the equipment is kept clean and in good repair, adequate supervision and skilled labour is provided, and unnecessary complications (e.g. small amounts of admixture) are avoided. Data relating to the operation should be carefully recorded—w/c, type of cement, and/or additives, type of mixing and pumping equipment, mixing and delivery time, grout fluidity and strength, source and chemistry of mixing water, length of grout line, pressure and quantity of grout injection, air temperature, and the names of the operating personnel. Such data will help to pinpoint reasons for anchor malfunction, should it subsequently occur.

It is strongly recommended that specific gravity checks as well as flow cone or flow meter testing should be used to supplement the results of conventional cube crushing programmes—a retrospective source of data.

CORROSION AND CORROSION PROTECTION

Mechanisms and causes of corrosion

The corrosion of prestressing steel is largely electrolytic and Longbottom and Mallett [1973] list the pre-requisites as (*i*) an electrolyte having interfaces with (*ii*) an anode and a cathode which also have (*iii*) direct metallic interconnection.

The electrolyte is usually aqueous, and a mere surface film is adequate. Reactions are initiated as a result of inhomogeneities or impurities in the steel or grout, or by the presence of chlorides or other salts in solution.

The cathode has a higher electrical potential relative to the electrolyte than the anode, which is normally lower in the electrochemical table. The more common elements are arranged as follows:

(stainless steel)
Na Al Zn Cd Fe Ni *Sn Pb* H_2 *Cu Ag* O_2
(passivated *Al*)
Anodic <——————————> Cathodic

The general rule is that electrolyte action will be more severe between electrodes which are widely separated in the table than between those which are closer.

There are generally held to be three major mechanisms of corrosion:

(1) **Corrosion by pitting.** Under conditions of chemical and/or physical inhomogeneity in the steel or electrolyte, ionisation will occur at both anode and cathode, constituting a bimetallic cell (Fig. 8a).

(2) **Corrosion involving crack formation under tension** ("hydrogen embrittlement"). This is more a physical corrosion, mainly affecting highly stressed carbon steels. The best known cause of brittleness is nascent hydrogen (Fig. 8b). The cathode reaction:

$$2H^+ + 2e^- \rightarrow H_2$$

is favoured by acid environments, and the hydrogen so produced tends to disrupt the structure of the steel.

From a survey of reports on hydrogen embrittlement it appears that oil quenched and tempered steels are far more susceptible to hydrogen embrittlement than drawn types. There is however no unanimous opinion about the susceptibility of prestressing steel to hydrogen embrittlement in highly alkaline grout.

(3) **Corrosion involving oxygen.** Local concentations of oxygen at a *cathode* act to accelerate corrosion:

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$$

The reaction is favoured by alkaline conditions (see Fig. 8c) and oxygen concentrations at an *anode* lead to the formation of a protective, passivating layer of rust:

$$2Fe^{3+} + \tfrac{1}{2}O_2 + 4OH^- + (n-2)\,H_2O \rightarrow Fe_2O_3\,nH_2O$$

In the alkaline environment provided by a good dense grout, steel is passivated in this way. As Portier [1974] noted, however, rust so formed is easily removed by the circulation or infiltration of water, thus leading to progressive dissolution of the steel.

There are two main chemical controls on these reactions—water, and electrochemical potentials.

(*i*) *Water.* Regardless of the type of corrosion, it can only occur in an ionic medium, and, under natural conditions, water is the most widespread bearer. The renewal of water increases the risk, while humidity is an even more dangerous parameter. The factors are closely interdependent: the supply of oxygen; the intensification of the microcell effect by the formation of a cathode at the water/air interface; and the action of hydrogen embrittlement.

(*ii*) *Electrochemical potentials.* With respect to Fig. 9, in Region I there is formation of ferrous ions, and generalised dissolution. Hence it would appear that to avoid corrosion, it suffices to remain within pH 8.5—13.5, i.e. in the range created by grouts. However

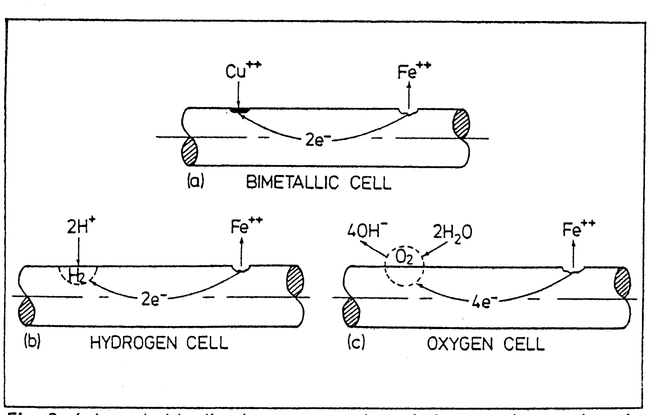


Fig. 8. (above). Idealised representation of three major modes of steel corrosion
(after Longbottom & Mallett, 1973)

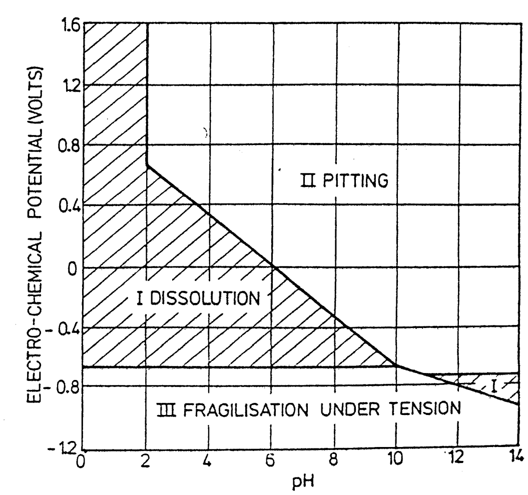


Fig. 9. (right). Relation of types of corrosion to pH and electrochemical potential
(after Caron, 1972)

this protection is very inadequate since it is known that (Region II), despite the passivating action of Fe_2O_3 formation, there may be corrosion by pitting under the influence of ions such as Cl^- when present in the cement. Also, Region III, corrosion with crack formation may occur.

Thus, although no domain is absolutely safe, the risks of corrosion can be simply reduced by:

(a) creating a pH environment of 9-12 in the grout. Chloride, sulphide, sulphate and carbonate ions all tend to lower the pH of the grout, enhancing electrolytic action,

(b) avoiding the possibility of harmful ions, e.g. chlorides, sulphides, sulphites, contacting the steel surface,

(c) selecting steels with low susceptibility to corrosion under tension, and eliminating from grouts anions which favour the passage of hydrogen e.g. SH^-, NO_3^-, CN^-, and

(d) preventing, as far as possible, the circulation of water.

Corrosion is thus aided by porous grout or concrete, and Rehm [1968] has found that in certain cases a cover of 25mm is insufficient. Therefore in anchors the porosity of the grout, and not simply its thickness of cover, should be stipulated.

Prestressing the steel may accelerate the rate of intensity of corrosion, although the elastic and strength properties of non-stressed steel are similarly affected. Quenched and tempered steels are far more susceptible to stress corrosion than cold drawn carbon steels of the type used in the UK for strand. Stress corrosion is more acute than ordinary corrosion for three main reasons—

(1) Stressing and releasing, if repeated, constantly destroys any protective oxide film,

(2) Stressing facilitates the development of micro-fissures, and

(3) Prestressing steel is, *a priori*, more susceptible than ordinary steel.

There is an increasing realisation that the failure of highly stressed materials under the influence of corrosion may be complex, and as yet it is impossible to be specific as to the conditions which will or will not give rise to stress corrosion. The only safe principle to follow is that if conditions could be dangerous—as in permanent ground anchors—then the whole design of the system should be orientated towards ensuring complete protection of the prestressing steel.

Whilst many of the problems of corrosion protection in prestressed systems in general are not present in ground anchor works Portier [1974] has pointed out that there are a number of corrosion problems specific to ground anchors, namely—

(i) Risks due to uplift pressure. Anchors may serve to stabilise foundation rafts, generally located underneath the water table and hence liable to uplift pressure. The slightest orifice serves as a drain cock and water may then flow along the tendon. This is particularly serious for strand anchors, although Soletanche Co Ltd., now use an epoxy pitch which is claimed to penetrate the tendon core and ensure absolute imperviousness, and British strand manufacturers appear confident about the penetration of corrosion resistant greases used at present with polypropylene sheathing.

(ii) Sealing. There are two contrary trends—either the risks are considered great and attempts made to protect the steel (as described below) or the risks are thought minimal and the tieback is immersed in the cement grout.

The latter method is older, and about 90 per cent of existing permanent anchors appear to have been so constructed, and whilst no failures have been observed no systematic records of corrosion have been taken.

(iii) The free length. This usually consists of a steel sleeve, or more often a plastic sleeve, which may easily be rendered impervious at the join. The tendon which passes inside is already protected by this sleeve, and also has additional protection from the cement filling the space between the sleeve and borehole wall. A problem is to prevent the formation of longitudinal paths (along which water can flow) along the axis of the sleeve. Various substances have been used for filling as cement grout does have certain disadvantages, and recent trends are towards synthetic substances which can impregnate the core of the tendon, while being at the same time flexible.

(iv) The head. While often being the most susceptible zone, it invariably receives least attention. It is vulnerable for many reasons: grout settling affects it, leaks emerge through it, mechanical and heat stresses create electric couples out of proportion with those of the sealing, and, it is in contact with the potentially corrosive atmosphere. One possibility is to ensure that on completion of the final grouting operation the top anchorage is completely encased in concrete. This however pre-empts the possibility of restressing the anchor at a future date. An alternative is to enclose the top anchorage in a steel or rigid plastic cover filled with grease or bitumen, again after final stressing. The PCI Recommendations [1974] advocate the "asphaltic painting" of all top anchorage hardware.

Classification of groundwater aggressiveness

It has been demonstrated that certain ions, both in the grout and in the groundwater initiate and sustain corrosion. Quantitative limits on aggressivity of environments have been drawn up by Bureau Securitas [1972] and FIP [1973]. Ground and mixing waters classed as aggressive are:

(1) *Very pure water.* It is termed aggressive if the concentration of CaO is less than 300mg per litre. Such waters dissolve the free lime and hydrolyse the silicates and aluminates in the cement.

(2) *Acid waters.* If pH is less than 6.5, they are considered aggressive as they may attack the lime of the cement. They are normally industrial waters, water with dissolved carbon dioxide, or water containing humic acids.

(3) *Waters with a high sulphate content.* These react with the tricalcium aluminate of the cement to form salts which disarrange the cement by swelling. Among these are (*a*) selenious water, with a high content of dissolved sodium sulphate, and (*b*) magnesian water, with a high content of dissolved magnesium sulphate. Waters with these salts are classed as very aggressive when the concentration of the salts exceeds 0.5g/litre for selenious water and 0.25g/litre for magnesian water. It is noteworthy that these values refer to stagnant water, and for flowing water the concentrations are 40 per cent of the above values.

Recommendations also refer to the aggressivity of the grout towards the steel of the tendon. In order to avoid "stress corrosion" of the tendon, the cement must not have a chlorine content, from chlorides, which exceeds 0.02 per cent by weight, and sulphur from sulphides, which exceeds 0.10 per cent by weight. These are provisional values only.

Any admixtures used must likewise contain no elements aggressive towards the steel or cement, and so the use of calcium chloride is forbidden.

Degree of protection recommended in practice

Methods used to protect rock anchor tendons reflect the following factors; the intended working life, the aggressiveness of the environment and the consequences of failure due to corrosion. Systems should be capable of effective protection against mechanical damage, as well as chemical, and should not therefore be impaired by the operations of fabrication, installation or stressing.

Three different situations can be delineated for the purposes of discussion, but in practice their distinction is often difficult.

(a). Temporary anchors in a non-aggressive environment. It is normally safe to assume that the cement grout will protect the fixed anchor length and the specified

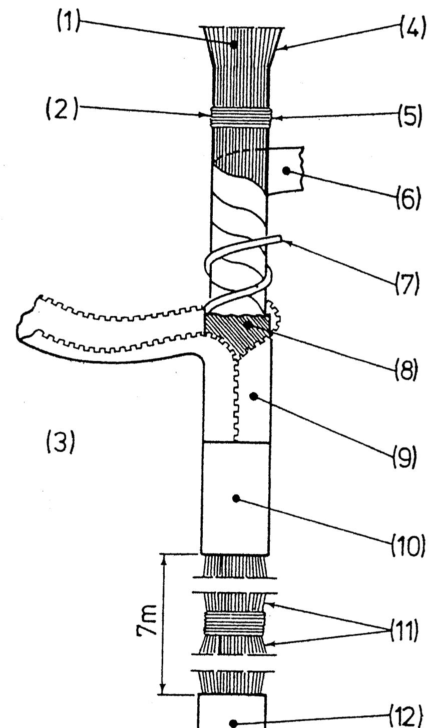


Fig. 10. Corrosion protection of tendons at Cheurfas Dam (after Cambefort, 1966)

(1) 630 5mm galvanised steel wires
(2) Average diameter of bound cable: 15cm
(3) Average diameter of finished cable: 20cm
(4) "Flint-kot" coating
(5) Bindings every 50cm
(6) "Flint-kot" coated tarpaulin
(7) Aloe rope
(8) Plastic mattress (mixture of grease and bitumen)
(9) Tarpaulin sheath with zip fastener
(10) Cement stopper sealing wires and tarpaulins
(11) Scraped wires
(12) White metal point
(13) Sealing tube

minimum cover for this type of anchor is not normally very large. Figures quoted by various engineers indicate a range of values for this type of anchor from 5 to 20mm. The need for some form of protection over the free length is not now disputed although it is not always enforced. White [1973] states that in the United States often no protection is provided even for a temporary anchor with a working life up to three years. Generally, however this is not so, and a combination of grease and tape is common practice. FIP [1973] recommend a grout cover of at least 5mm.

(b) Temporary anchors in an aggressive environment. The fixed anchor zone can still be satisfactorily protected with a good quality grout cover. However, the minimum cover now becomes more important and Matt [1973] has recommended that a minimum value of 30mm should be guaranteed. Greater importance is also placed on the assurance that this grout is not cracked: if this possibility cannot be excluded some additional protection system should be included. Protection of the free anchor length is still only a single protective system in most cases, plastic sheathing or greased tapes being the usual solution although grout or other protective coatings are also possible. The risk of failure due to corrosion of the tendon is greatly reduced if components of diameter in excess of 7mm are used. A minimum grout cover of 5mm is again recommended by FIP at present.

(c). Permanent anchors. These should always have protective systems designed assuming an aggressive environment: environmental changes during the life of the anchor cannot be anticipated and the possibility that the anchor will be exposed to an aggressive environment cannot therefore be excluded. It is now widely held that permanent anchors should be provided with a double corrosion protection system. It is recommended that, as far as possible, the protection should be made and checked under workshop or equivalent conditions. The chosen protection system should not adversely affect the handling of the tendon or the behaviour of the bond.

Over the fixed length, there is always grout cover, but it is common to provide an additional coating. The coating may be a high strength epoxy or polyester resin but any suitable material which has a proven resistance to the existing aggressivity and does not adversely affect the bond may be used. Sometimes it is considered sufficient to pregrout the anchor zone and inspect it before homing the tendon. The cover recommended by FIP is 5-10mm minimum.

The free anchor length is similarly doubly-protected. Grease packed plastic sheaths fitted under factory conditions are becoming a popular method. Various other elastic substances can also be used within a plastic tube; for example, bitumastic compounds like buto rubber, or greased tapes used within the sheath. The annular space outside the plastic tube is normally cement grouted but in some cases bitumen enriched grout is used.

Corrosion protection systems employed in practice

Numerous systems of protection against corrosion have been used—and in some cases abandoned—for rock anchors. Fundamentally, a distinction is drawn between systems for pre-protection and post-protection. The former are employed prior to homing, whereas the latter are effected after tendon installation.

With respect to systems of pre-protection sheathing is currently the most common method. PVC sleeving, or water resistant or greased tape is now almost standard protection for rock anchors. Greased tape in particular is easy to handle and apply with a 50 per cent overlap, and although the risk of damage during tendon installation is high, it does form an extremely efficient barrier to chemical attack. The grease should be supple to allow subsequent tendon extension during stressing without causing large friction losses, or being destroyed, and should thoroughly penetrate the tendon. With reference to sleeving, PVC or polypropylene sheathing may now be delivered to site already on the individual steel wires or strands, or it can be introduced in a separate process on site (Littlejohn & Truman-Davies, 1974).

Other pre-protection systems, which are described in an excellent article by Portier [1974] include:

(*a*) coatings providing cathodic protection,
(*b*) cathodic protection by electric current,
(*c*) synthetic, semi-rigid films,
(*d*) rigid synthetic anchor plugs, and
(*e*) metal casings under compression or tension.

Systems of post-protection are also numerous and consist basically of filling in-situ a sleeve over the free length, after tensioning. The substances used range from fluids, such as oils or water containing lime, to bitumens and cement, and the various materials have been described

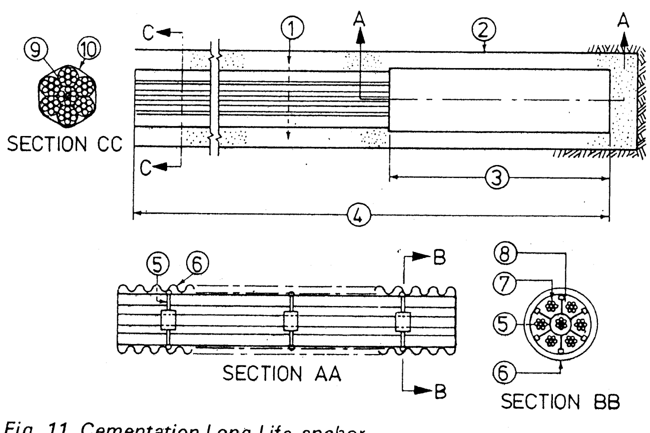


Fig. 11. Cementation Long Life anchor (after Golder Brawner, 1973)
1, Grout 2, Borehole wall. 3, Potted length. 4, Overall length. 5, Central spacers. 6, Corrugated sheathing. 7, Strands with polypropylene sheath removed and degreased. 8, Polyester resin. 9, Strands, each sheathed in polypropylene. 10, PVC adhesive tape binding

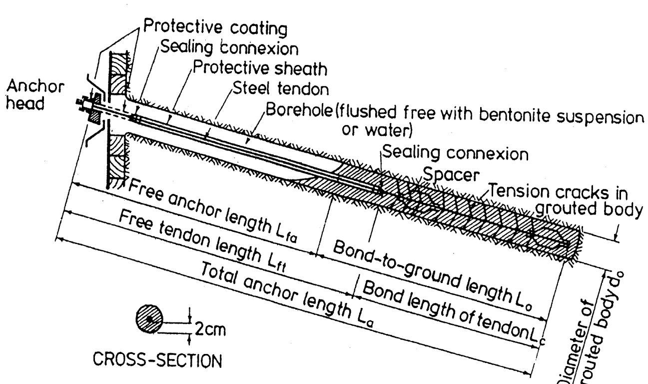


Fig. 12. Temporary anchor construction of Type A (after Ostermayer, 1974)

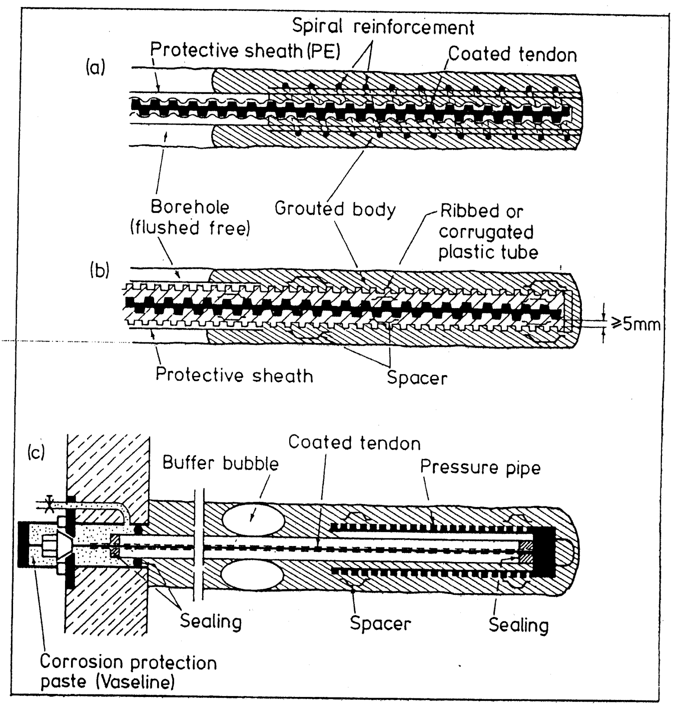


Fig. 13. Permanent anchor constructions; (a) Type A with coated tendon; (b) Type A with ribbed plastic tube; (c) Type B with pressure pipe (after Ostermayer, 1974)

and classified in some detail by Bureau Securitas [1972] according to duration of protection and aggressivity of the surrounding medium.

An interesting illustration of the development of corrosion protection systems is provided by comparing the anchors used at Cheurfas, in 1934 with a modern counterpart, As shown in Fig. 10 in the former, bitumen was liberally employed, and the wires were galvanised (except in the fixed length). A claim (Khaova *et al*, 1969) that about 11 per cent of the total tendon cross-sectional area has been lost due to corrosion in just over 30 years has been discredited recently by Portier [1974].

A sophisticated modern type is the Cementation Long Life Anchor (Fig. 11) in which the polyester resin not only ensures complete protection of the fixed anchor tendon length, but contributes a "deadman" effect to the whole anchorage system. The free length of the tendon consists of strands individually coated in grease and covered by polypropylene sheathing.

Ostermeyer [1974] discusses the classification of the sophisticated bar anchors most commonly used in Germany. For temporary anchors, Type A (Fig. 12) is generally used with only one stage of protection (sheath on the free length and at least 20mm of grout over the fixed length). Ostermeyer emphasises the importance of head protection and recommends that at least one coat of paint be applied to the head and the tendon above the sheathing.

Anchors of Type A (fixed under tension, Figs. 13a and b) and Type B (fixed anchor under compression, Fig. 13c) are used in permanent works. Such anchors have double protection—against both mechanical damage and chemical corrosion.

In Type B, the corrosion protection can be applied and tested under factory controlled conditions without difficulty. The protection on the whole length is examined electrically and then covered with a sheath. As the protection does not transmit load, a relatively elastic material can be used and paste or grease pressed into the annular space between sheath and tendon may be considered adequate.

In Type A, the application of a protection which remains undamaged during construction and stressing is difficult. For the anchors shown in Fig. 13a a synthetic coating is desirable which not only has an excellent bond with steel but, in addition, must also be flexible and strong enough to carry high bond stresses over a long period. When the coating is thick, the danger exists that the fixed anchor will be subjected to high bursting stresses. A spiral reinforcement is therefore provided to resist these stresses.

For the Type A anchor in Fig. 13b, the tendon is inside a ribbed plastic sheath. The annular space is filled with cement. Although this cement will crack, as in all type A anchors, the criterion of corrosion protection is considered to be fulfilled when the cement in the annular space is at least 5mm thick, and the sheath at least 1mm thick. When a ribbed plastic sheath is used, the danger of material fatigue is less than in cases where a protective coating has been directly applied to the tendon (Fig. 13a). The requirements of double corrosion protection are also met at the anchor head.

It is generally concluded that whilst the protection of rock anchors is a serious problem, it does not appear to be a crucial one at present and responsible engineers are clearly corrosion conscious. Nevertheless there is a growing need to establish standards of corrosion protection which will be accepted and used widely by consulting and contracting engineers.

References

ACI Committee 212 (1971): "Guide for Use of Admixtures in Concrete" J. Amer. Conc. Inst. (Sept.) pp. 646-676.

Anon (1969): "Ground Anchors for the World's Largest Ship Dock" *Contract Journal*, (June 19) pp. 850-851.

Associated Tunnelling Co. Ltd. (1973): "Report on Ground Anchor Tests—Thames Flood Prevention". Associated Tunnelling Co. Ltd., Lowton St. Mary's, near Warrington, England (Unpublished).

Barron, K., Coates, D. F. and Gyenge, M. (1971): "Artificial Support of Rock Slopes", Dept. of Energy, Mines & Resources, Mines Branch, Ottawa, Research Report R 228 (Revised), 145 pp.

Bomhard, H. and Sperber, J. (1973) "Der felsverankerte Spannleichbeton—Kragarm der Skiflugshanze Oberstdorf und Betrachtungen zur Anwendbarkeit und Beans pruchbarkeit von Konstruktionsleichtbeton". Beton-und Stahlbetonbau, 5 pp. 107-117.

British Standards Institution (1972): "The Structural Use of Concrete". CP 110, Part 1, BSI, 2 Park St., London.

Broms, B. B. (1968): "Swedish Tieback Systems for Sheet Pile Walls". Proc. 3rd European Conf. Soil Mech. & Found. Eng., Budapest, pp. 391-403.

Brunner, H. (1970): "Praktische Erfahrungen bei den Hangsicherungen In Neuenhof", Strasse und Verkehr, 9, pp. 500-505.

Bureau Securitas (1972): "Recommendations Regarding the Design, Calculation, Installation and Inspection of Ground Anchors". Editions Eyrolles, 61 Boulevard Saint-Germain, Paris-Ve (Ref. TA 72).

Buro, M. (1972): "Rock Anchoring at Libby Dam", Western Construction (March), pp. 42, 48 and 66.

Cambefort, H. (1966): "The Ground Anchoring of Structures", Travaux, *46*, (April-May) 15 pp.

Caron, C. (1972): "Corrosion et Protection des Ancrages Définitifs", Construction, (Feb), pp. 52-56.

Cementation Co. Ltd., (1962): "Anchor Stressing". Report No. 10, Cementation Co. Ltd., Mitcham Road, Croydon, Surrey. (Unpublished).

Cementation Co. Ltd., (1969): "Report on Tests Carried out on Ground Anchors in Keuper Marl". Cementation Co. Ltd., Mitcham Road, Croydon, Surrey. (Unpublished).

Comte, C. (1971): "Technologie des Tirants". Institute Research Foundation Kollbrunner/ Rodio, Zurich 119pp.

Conti, N. (1972): Reply to FIP Questionnaire (Unpublished).

Dixon, J. C. and Clarke, K. B. (1975): "Field Investigation Techniques", Chapter 3, p. 48 Site Investigations in Areas of Mining Subsidence Ed. F. G. Bell, Newnes & Butterworths, London.

Eberhardt, A. and Veltrop, J. A. (1965): "Prestressed Anchorage for Large Trainter Gate". Proc. A.S.C.E., 90, (ST6) pp. 123-148.

FIP (1972): "Draft of the Recommendations and Replies to FIP Questionnaire (1971)". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1973): "Final Draft of Recommendations". FIP Subcommittee on Prestressed Ground Anchors.

Geddes, J. D. and Soroka, I. (1964): "The Effect of Grout Properties on Transmission Length in Grout Bonded Post-Tensioned Beams". Mag. Conc. Res., 16, No. 47, pp. 93-98.

Golder Brawner Associates (1973): "Government Pit Slopes Project III: Use of Artificial Support for Rock Slope Stabilisation". Parts 3.1—3.6, Golder Brawner Assocs. Vancouver, Canada. (Unpublished).

Gosschalk, E. M. and Taylor, R. W. (1970): "Strengthening of Muda Dam Foundation using Cable Anchors". Proc. 2nd Cong. Int. Soc. Rock Mech. Belgrade, 3 pp. 205-210.

Ground Anchors Ltd. (1974): "The Ground Anchor System", Ground Anchors Ltd., Reigate, Surrey., (28 pp.).

Hennequin, M. and Cambefort H. (1966) "Consolidation du remblai de Malherbe". Revue Generale des Chemins de Fer (Feb.).

Hilf, J. W. (1973): Reply to Aberdeen Questionnaire (1972) (Unpublished).

Holz, P. (1963): "Prestressed support at Bancroft", Canadian Mining Journal, Nov. pp. 61-62.

Irwin, R. (1971): Reply to FIP Questionnaire (Unpublished).

Khaova, M. Montel, B. Civard, A. and Lauga R. (1969) "Cheurfas Dam Anchorage: 30 Years of Controls and recent Reinforcement". Proc. 7th Int. Conf., Soil Mech. and Found. Eng., Paper 15-12.

Koch, J, (1972): Reply to FIP Questionnaire (Unpublished).

Leech, T. and Pender, E. (1961): "Experience in grouting rock bolts" Proc. 5th Int. Conf. Soil Mech. and Found. Eng., 2, pp. 445-452.

Littlejohn, G. S. (1972): "Some Empirical Design Methods Employed in Britain", Part of Questionnaire on Rock Anchor Design. Geotechnics Research Group, Department of Engineering, University of Aberdeen (Unpublished Technical Note).

Littlejohn, G. S. and Truman-Davies, C. (1974): "Ground Anchors at Devonport Nuclear Complex" *Ground Engineering*, 7 (6) pp. 19-24.

Littlejohn, G. S. (1975): "Acceptable Water Flows for Rock Anchor Grouting", *Ground Engineering*, 8, (2), pp. 46-48.

Littlejohn G. S. and Bruce, D. A. (1975): "Rock Anchors—State-of-the-Art" Part I, Design. *Ground Engineering*, 8, (3) pp. 25-32; 8 (4) pp. 41-48.

Longbottom, K. W. and Mallett, G. P. (1973): "Prestressing Steels", The Structural Engineer, 51, (12) pp. 455-471.

McGregor, K. (1967): "The Drilling of Rock"; 1st Ed. C. R. Books Ltd., London (306 pp.).

Maddox, J., Kinstler, F. and Mather, R. (1967): "Foundation Studies for Meadowbank Dam", 9th Int. Conf. on Large Dams, Istanbul, 1, pp. 123-141.

Mascardi, C. (1973): "Reply to Aberdeen Questionnaire (1972)" (Unpublished).

Mawdsley, J. (1970): "Choice of Drilling Methods for Anchorage Installation". Supplement on Ground Anchors, *The Consulting Engineer*, (May, 1970) pp. 5-6.

Mitchell, J. M. (1974): "Some Experiences with Ground Anchors in London", ICE Conference on Diaphragm Walls and Anchorages, London (Sept.) pp. 129-133.

Moschler, E. and Matt, P. (1972): "Felsanker und Kraftnessanlage in der Kaverne Waldeck II", Scheweizerische Bauzeitung, 90 (31), pp. 737-740.

Moy, D. (1973): Private Communication.

Mullett, L. J. (1970): "Deep Fixings for Anchors". Supplement on Ground Anchors, *The Consulting Engineer*, (May, 1970) pp. 25-26.

Neville, A. M. (1963): "Properties of Concrete", Pitman, London.

Ontario Hydro (1970): "Provisional Specification M388 and Standard Specification M285", Toronto, Canada.

van Ormer, H. P. (1974): "Determining Rock Drillability", *Rock Products* (February issue) pp. 50-51.

Ostermayer, H. (1974): "Construction, Carrying Behaviour and Creep Characteristics of Ground Anchors". ICE Conference on Diaphragm Walls and Anchorages, London, (Sept.) pp: 141-151.

Paone, J., Unger, H. F. and Tandanand, S. (1968): "Rock Drillability for Military Applications", Final Contract Report AD 671671 for Army Research Office, U.S. Dept. of the Interior, Bureau of Mines, Twin Cities Mining Res. Centre Minneapolis, Minnesota.

Paone, J., Madson, R. and Bruce, W. E. (1969): "Drillability Studies. Laboratory Percussive Drilling". Bureau of Mines Report of Investigations 7300 US Dept. of the Interior, Washington.

Parker, P. I. (1958): "The Raising of Dams with Particular Reference to the Use of Stressed Cables". Proc. 6th Cong. on Large Dams, New York Question 20, 22 pp.

PCI Post-Tensioning Committee (1974): "Tentative Recommendations for Prestressed Rock and Soil Anchors", PCI, Chicago, U.S.A., 32 pp.

Pender, E., Hosking, A. and Mattner, B. (1963): "Grouted Rock Bolts for Permanent Support of Major Underground Works", J. Inst. Eng., Aust., *35*, pp. 129-150.

Portier, J. L., (1974): "Protection of Tiebacks Against Corrosion", Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors (pp. 39-53) 7th FIP Congress, New York.

Powers, T. C. (1968): "The properties of fresh concrete", J. Wiley & Sons Inc., New York.

Protydyakonov, M. M. (1962): "Mechanical Properties and Drillability of Rocks", Proc. 5th Symposium on Rock Mechanics, University of Minnesota, Minneapolis, U.S.A. (May, 1962) pp. 103-118.

Rehm, G. (1968): "Corrosion of Prestressing Steel", Proc. Sym on Prestressing, Madrid, (June).

Ryd, E. and Holdo, J. (1956): "Percussive Rock Drills—Their Contribution and Method of Operation". Manual of Rock Blasting (Chapter 12:01-1-35) Atlas Copco, Stockholm.

Schmidt, R. L. (1972): Drillability Studies—Percussive Drilling in the Field". Bureau of Mines Report of Investigations 7684. US Dept. of the Interior, Washington.

Soletanche Co. Ltd., (1970): "Other Types of Anchor". Supplement on Ground Anchors, *The Consulting Engineer*, (May, 1970) p. 13 and 15.

South African Code of Practice (1972): "Lateral Support in Surface Excavation", The South African Institution of Civil Engineers, Johannesburg.

Standards Association of Australia (1973): "Prestressed Concrete Code CA35—1973 Section 5—Ground Anchorages" pp. 50-53.

Stocker, M. F. (1973): Reply to Aberdeen Questionnaire (1972) (Unpublished).

Thompson, F, (1970): "The Strengthening of John Hollis Bankhead Dam", *Civil Engineering* (NY), 39 (12), pp. 75-78.

Unger, H. F. and Fumanti, R. R. (1972): "Percussive Drilling with Independent Rotation", Bureau of Mines, Report of Investigations 7692. US Dept. of the Interior, Washington.

Universal Anchorage Co. Ltd., (1972): "Report on Rock Anchor Tests for Frigate Complex H. M. Dockyard, Devonport". Report No. 189, Universal Anchorage Co. Ltd., Egerton Street, Farnworth, Bolton, England (Unpublished).

US Army (1964): "Scientific and Technical Applications Forecast, 1964, Excavation". Office of the Chief of Research and Development. Department of the Army, Washington, D.C. (250 pp.).

White C. G. (1965): "A Rock Drillability Index", DSc Thesis, Colorado School of Mines, Golden, Colorado. 156 pp.

White, R. E. (1973): Reply to Aberdeen Questionnaire (1972) (unpublished).

Part 3: Stressing and testing

INTRODUCTION

PRESTRESSING AN ANCHOR automatically tests the installation, confirms to a certain degree design safety factors, and ensures satisfactory service performance. This is equally true for prestressed anchors and those subsequently intended to act as "passive" untensioned members, and in both cases an initial stress history often enhances subsequent behaviour.

In addition, acceptance criteria based on standardised tests gauge the suitability and effectiveness of the installed anchor with respect to the intended application. Possible errors made in either the design or construction stages will be pinpointed immediately and potentially dangerous and expensive consequences avoided.

Incorporating these important precepts, this third part of the rock anchor review§ describes anchor stressing techniques, the monitoring and presentation of data, and provides guidance on the interpretation of stressing results. This basic information is intrinsic to anchor testing.

The authors believe that a standard approach to the testing and analysis of anchor behaviour should be established, relating to both short and long-term behaviour. Accordingly, the following basic types of test and quality control are recommended for consideration, and are described in detail:

1, precontract component testing,
2, acceptance testing of production anchors,
3, long term monitoring of selected production anchors,
4, special test anchors, and
5, monitoring of the overall anchor/rock/structure system.

A final section deals with aspects of long-term service performance, and reviews the relatively small number of case studies published to date. These highlight various parameters and phenomena which influence anchor behaviour in the long term.

STRESSING

Mode of stressing

There are basically two methods of applying stress to an anchor tendon:

(*i*) torque, applied via a torque wrench to some form of anchoring nut threaded on to a rigid bar tendon (Fig. 1a), and

(*ii*) direct pull, which may be applied to the tendon by a jack seated for example on a stressing stool or chair (Fig. 1b).

Torquing is normally restricted to small capacity (150kN max.) single bar tendons i.e. rock bolts of various types. In practice care must be taken to ensure that torsional stresses are not incidentally applied to the tendon, since they may combine with the tensile stresses and reduce the effective strength of the bar. This disadvantage can be alleviated by introducing a friction reducing material e.g. a molybdenum disulphide based lubricant, beneath the lock-nut prior to stressing.

The required torque to produce a specified load is usually expressed empirically in the form

Tensile load (kN) $= C \times$ torque (kN.m)

but whilst C may be defined within narrow limits under controlled laboratory conditions, experience suggests that variations of $\pm$ 25% can be expected for the value of C under field conditions. In addition the installed load is subject to variations due to a number of conditions related to control of alignment, friction between mating parts and size of bar tendon. Bearing in mind also that torquing is usually accomplished with the aid of an air driven impact wrench, the output of which is subject to variation in airline pressures, it is not surprising that the equipment needs frequent calibration and that good maintenance is vital. For reliable results therefore it is recommended that a calibrated hand wrench be used as a check in all cases. Nevertheless, the equipment is light, compact, easy to handle, and the stressing procedure is simple, and cheap. As a result the torquing method of stressing rock bolts is very popular in practice, and for the interested reader more detailed information can be found in the ISRM draft publication "Suggested methods for rockbolt testing" (1974).

By far the most common and indeed for the vast majority of anchors the only suitable method is stressing by direct pull. Strand is now much more commonly used than wire, and as a result multistrand and monostrand direct pull jacks are the most common systems used today in prestressing. Monojacking relates to single strand stressing and the individual tendon units are tensioned in turn (Fig. 2a). Multistrand jacks permit all the strands of the tendon to be stressed simultaneously. These jacks may be of solid or hollow ram design (Figs. 2b & c).

Practical aspects of stressing

In order to introduce the reader to some basic procedures and concepts, as well as the stressing jargon, the following description deals with practical aspects relating to anchor stressing in the field.

Top anchor movements should be kept ideally to a minimum. Therefore the bearing plate may be placed directly on to strong competent rock, or alternatively embedded in a mass concrete block to spread the anchor forces in the case of weak rock. For anchors with design loads in excess of 150kN it is important, prior to start of

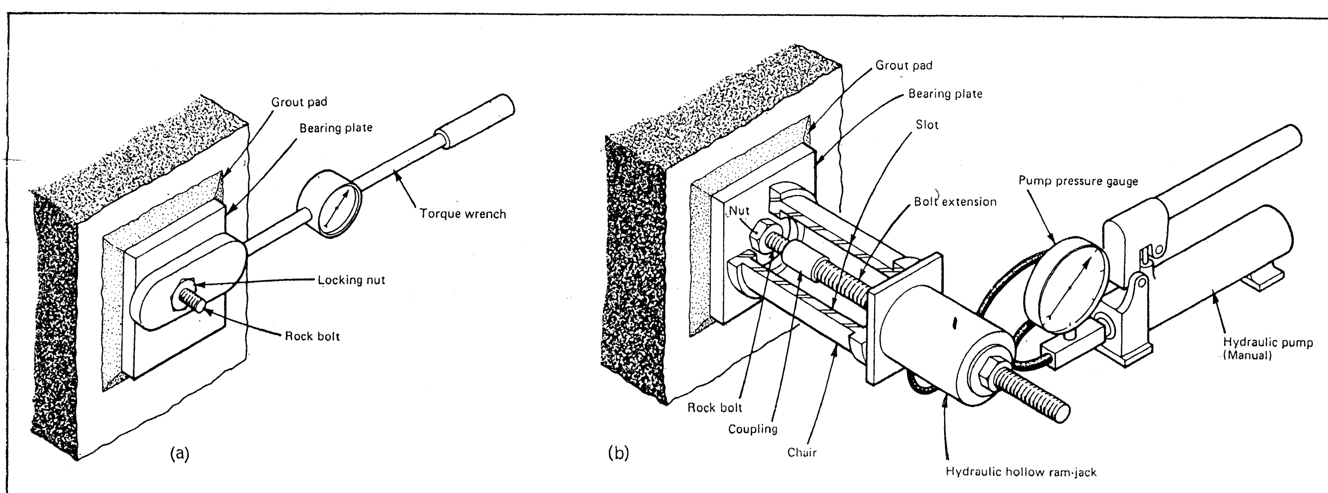


Fig. 1. Stressing by (a) torque wrench, and (b) direct pull

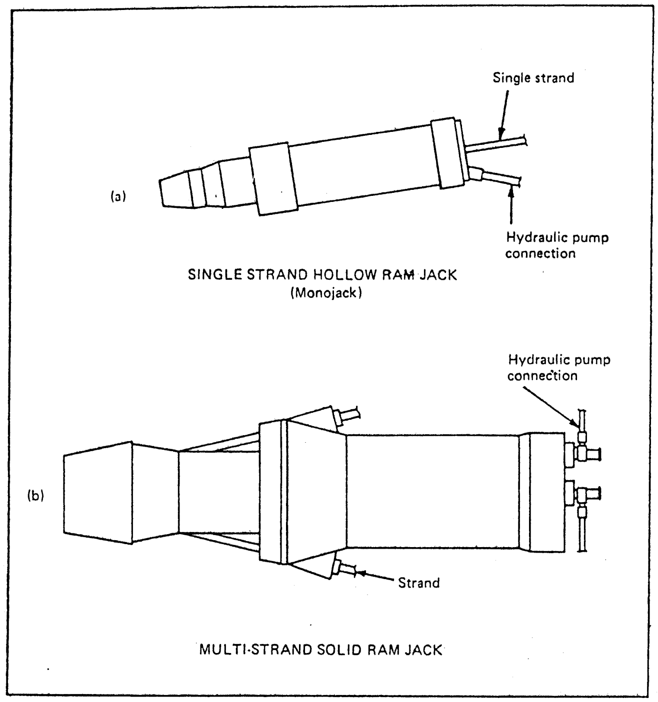


Fig. 2. Typical jacks for tensioning rock anchors; (c) (below) depicts a hollow ram multi-strand stressing jack
(photo, courtesy, Ground Anchors Ltd.)

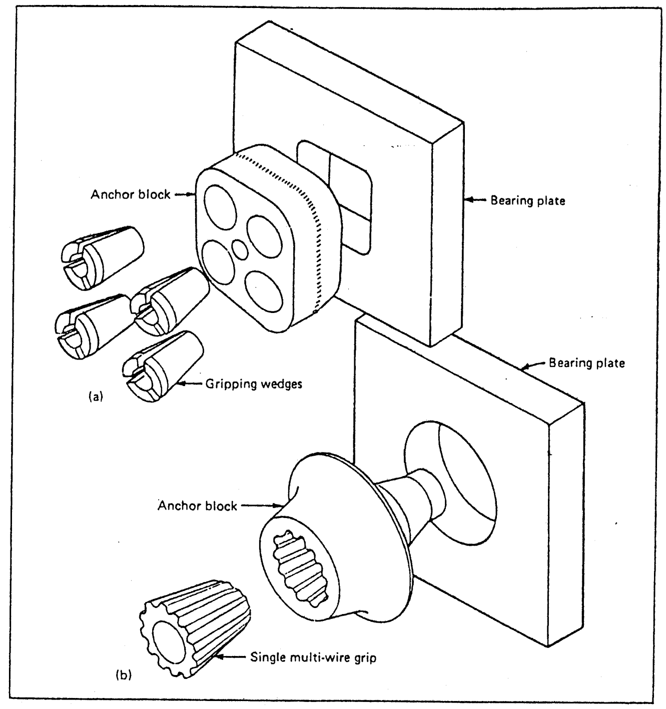
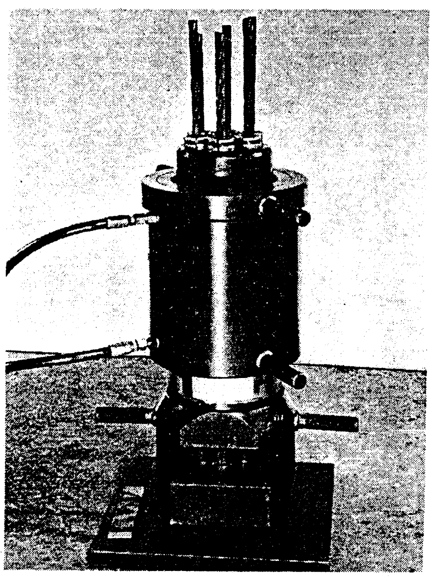


Fig. 3. Typical anchor heads for a strand or wire tendon



stressing, to check that the steel bearing plate has been correctly bedded centrally and normal to the tendon. This check can eliminate chaffing of the perimeter tendon components in the case of multiwire or strand tendons which splay outwards in the zone of the top anchorage or jack assembly (Fig. 3). This problem does not appear to be recorded for the case of parallel rigid bar groups.

Once the anchor grout has reached a specified strength, stressing may proceed. The authors recommend a crushing strength of 28N/mm² (see Part 2: Construction).

For solid bar or single unit tendons, the tensioning assembly may be fitted on to the tendon as soon as it has been thoroughly cleaned. For multi-unit tendons however it is important to verify that the wires or strands are not crossed within the free-length before fitting the anchor block and jack assembly (Fig. 4a). The correct alignment of strands is best accomplished by providing a form of comb grillage or fork (Fig. 4b), and the use of guide cords with caps is particularly beneficial on high capacity multi-unit tendons.

To simplify the description the remaining practical comments will relate primarily to multistrand stressing using a hollow jack.

If the tendon is to be subjected initially to a special test overload to prove its design capability, then the permanent grips are normally omitted from the anchor block at this stage of the work. The jack is now fitted over the strands and the temporary stressing units (Fig. 4a) are then assembled. The jack chair or stool which provides a support for the jack is placed centrally over the tendon and the side opening should be in a convenient position to allow the operator to inspect the anchor head during the tensioning operation (Fig. 2c).

The jack is now fitted manually or by a mechanical lifting device. Mechanical lifting and handling equipment is recommended for jacks weighing in excess of 80kg, and a guide relating the approximate weight of hollow ram steel jacks to their maximum rated capacity is given in Table I.

It is important, prior to stressing, to verify that the elongation at the top anchor will be be in excess of 30mm for the maximum load to be applied, otherwise the re-usable grips (wedges) in the temporary loading head (Fig. 4c) cannot be freed on destressing. Where extensions of 30mm or less are envisaged the jack piston should be advanced to 30mm before placing the temporary loading head. The re-usable grips must be lightly lubricated with high pressure grease prior to their fitting. These grips are finally homed to give a tight fit, by a gentle tapping with a special ring or U-shaped hammer. Stressing may now proceed.

TABLE I. APPROXIMATE WEIGHT OF HOLLOW RAM STEEL JACKS

Maximum-rated capacity (kN)	Approximate weight (kg)
200	20
500	40
1 000	80
2 000	150
3 000	200
4 000	300

It should be emphasised immediately that the space in front of the jack, and in line with the tendon axis must be kept free of personnel during the prestressing operation. Alternatively, a properly designed small aperture steel mesh cage should be provided for protection of the operators or passers-by.

A hand pump is the simplest means of pressurising the jack system to advance the ram but where many tendons need to be stressed and a high output is required a motor-driven pump is advantageous.

Bearing in mind that the stressing system may have been designed to operate at high pressures (quoted test pressures of 600 atmospheres by manufacturers are not uncommon), it is not always practical to monitor pump pressures below 40 to 50 atmospheres. The initial position of the jack piston is therefore noted at this pressure which is also considered sufficient to take up any slack in the tendon. The actual zero reading for the piston can be found by extrapolation when the ram extensions at subsequent higher pressures are noted (Fig. 4c). Throughout the stressing operation, both extensions and gauge pressures are recorded but this aspect is discussed in more detail below.

Where a test load has to be held for a period of time a slight fall in gauge pressure will be noted even though the extension of the piston remains constant. This

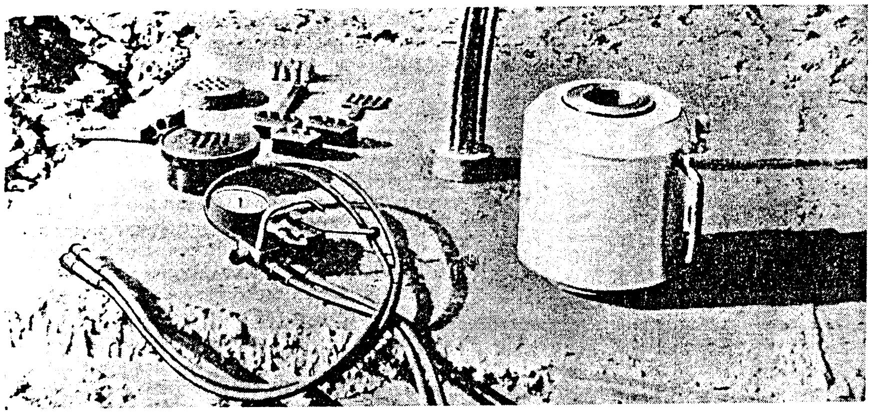
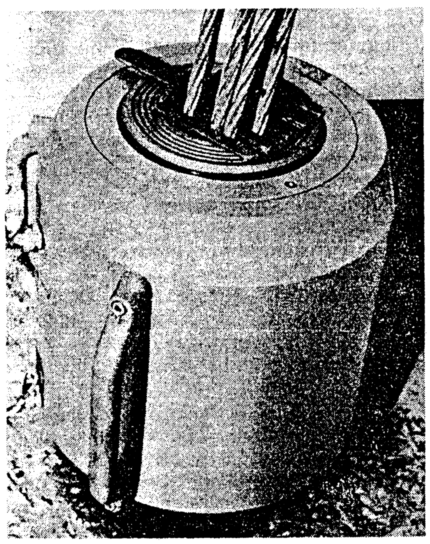
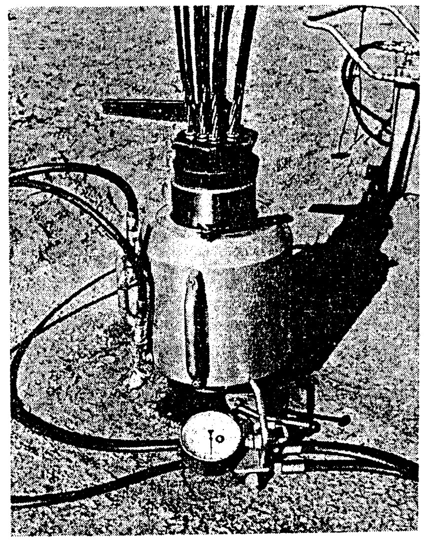


Fig. 4. (a) (above), Anchor block and components of jack assembly; (b) (below left), Fork for alignment of strands; and (c) (below right), Stressing through the temporary load bearing plate (photos, courtesy, Cementation Ground Engineering Ltd.)





loss is internal to the system and a gentle application of pressure to the original reading will in most cases produce the same extension as initially recorded. For long-term stressing a lock valve at the jack is recommended.

On completion of the initial stressing operation, the double-acting ram retracts and leaves the temporary loading head in position to allow its removal. Gentle tapping with a wooden or rubber mallet is usually sufficient to release the grips which should now be re-greased and kept in a clean condition ready for the next stressing operation.

In order that load can be finally locked into the tendon, permanent grips must be inserted into the permanent anchor block. This should be possible without the total removal of the jack and chair from the tendon, although the temporary grips will have to be removed until the permanent grips are fitted.

During stressing the chair provides a reaction head (Fig. 5) restricting the upward movement of the permanent gripping wedges. When the desired pump gauge reading is attained, the jack ram is retracted and immediately the wedges are drawn or pulled in around the tendon as it tries to retract, and so load is locked off. It is noteworthy that when this final load is considered insufficient (for reasons described below) the anchor may be restressed, and if necessary steel spacers or shims of various thicknesses inserted beneath the anchor block to raise the load at lock-off by increasing the tendon extension (Fig. 6).

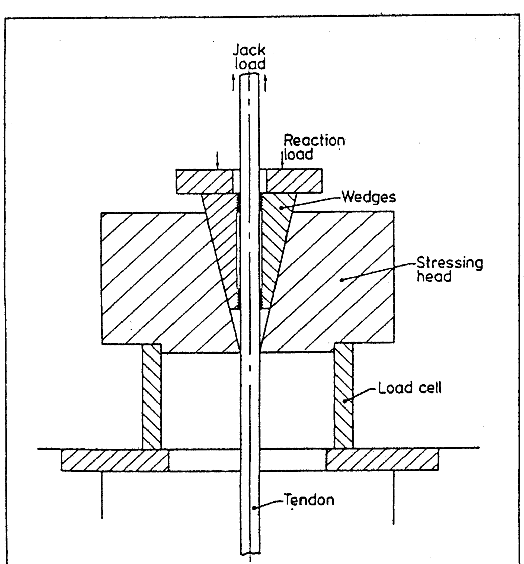


Fig. 5. Basic stressing mechanism at the top anchorage

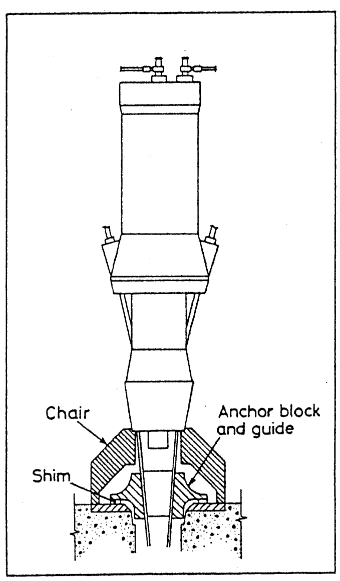


Fig. 6. Jack arrangement for shimming

Choice of stressing system

Multistrand stressing is swift and simple in operation once the jack has been correctly located, and requires relatively little data recording and back analysis in most cases. Nevertheless, multistrand stressing cannot provide a high degree of control over the behaviour of individual tendon units, or, at final lock-off, a guarantee of equal load in each unit. This is particularly important in anchors of free length less than 10m, where extensions are relatively small and so variations in the amount of wedge pull-in, for example, will represent proportionately larger load discrepancies than in a longer tendon.

Conversely, with respect to anchor block lift-off checks — detailed later — the multistrand jacking system alone can show the total load on the anchor in one stressing operation. Furthermore, for cyclic loading and unloading programmes, this system is easier and quicker to employ and gives more control, especially during the destressing stage. Some engineers also consider that a multistrand jacking system alone is capable of economically supplying prestressing loads in excess of 3 000kN. This view is based on the larger number of individual time-consuming stressing operations, and the larger spacing required to separate the strands in the anchor block if a monojack is employed.

On the other hand monostrand stressing is a relatively popular method for tensioning tendons of up to six strands, and close control over the force in each individual strand can be achieved. Since the development of high speed front gripping jacks, and bearing in mind the limited number of strands, the method is not unduly time consuming. In addition, most single strand stressing jacks are light and easy to handle, which is a major advantage on most sites.

There are however important points concerning monojack stressing operations which are widely recognised but remain largely unexplained. For example, when Mitchell (1974) monitored with strain gauges the load fluctuations in two adjacent strands of an anchor tendon, he observed that the load in the first tensioned strand decreased steadily during the stressing of the adjacent strand (Fig. 7). This effect was in fact exaggerated because in this experiment the load was not incrementally applied to each strand in sequence as recommended in practice. Nevertheless the results clearly justified Mitchell's subsequent advice that after application of a nominal seating load to each strand, the remaining load should be applied in four or five equal increments to each strand in turn, in a specific sequence to ensure a uniform distribution of load

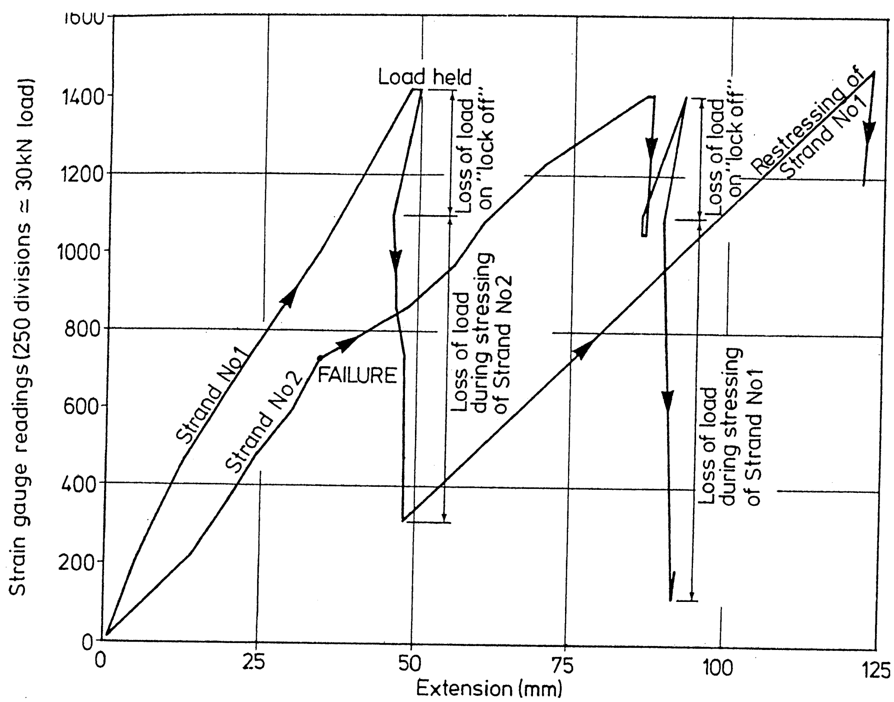


Fig. 7. Interference between two strands during monojack stressing (after Mitchell, 1974)

across the tendon. Mitchell also found that in a six strand tendon, at the completion of each stage of incremental loading, the greatest and least load losses monitored always occurred on the first and last strands loaded, respectively. This phenomenon has also been personally observed by Barley (1974) and the authors. In practice, after the final increment of one stressing sequence, the uneven distribution of the loading can be minimised by conducting a final stressing round to bring all strands up to the required load.

In general, it is wrong to recommend one stressing system over the other. Realistic comparisons, made to effect a choice, should only be attempted when the stressing and testing specification, and the environmental considerations e.g. accessibility, are known.

Whichever system is used, it is important in many cases to verify that the applied prestress is actually being resisted by the grouted fixed anchor zone, and further that the method of applying the tensile load is relevant to the particular application. For example the load may be applied remotely through a simply-supported beam, or by prestressing through a plate or pad bearing directly on the rock overlying the fixed anchor being tested. In the latter case, the tensioning procedure may simply prestress the rock and/or grout column between the fixed anchor and pad. This may have serious consequences if the test is supposed to check the stability of the pad against uplift, if it performs in service as the footing of a transmission tower for example. No work has been published on this phenomenon in rock, to the authors' knowledge, but current research being conducted by the Universal Anchorage Co. Ltd., and the Geotechnics Research Group suggests that, for shallow anchors installed in horizontally bedded flaggy sandstone, the load is resisted locally by the rock mass in the grouted fixed anchor zone, where the slenderness ratio (depth to top of fixed anchor/hole diameter) exceeds 15.

Monitoring procedures

The prestressing of any anchor, either production or special test, presupposes the graphical plotting of anchor load against tendon extension. Such a plot facilitates judgement as to the anchor's competence and efficiency. Therefore, it is most important to be familiar with the parameters to be investigated, and methods of their measurement, presentation, and interpretation.

The parameters

The two basic parameters are, obviously *load* and *extension*. The former is self evident, being the actual amount of prestress locked into the tendon at any one time. The tendon extension, however, involves other measurements, not always recognised as being significant in load — extension analyses. An extension, as measured *before* lock-off may be regarded as the "gross extension". At lock-off in the case of a wedge grip type top anchorage (Fig. 3a), pull-in of the wedges (and strand) will occur until the system is "tight". *After* lock-off, there may be movements due to bedding-in of the top anchor block and bearing plate, deflection of the structure, and/or permanent displacement of the fixed anchorage, in addition to the elastic extension of the tendon under load.

Long term monitoring may necessitate the recording of ground or air temperature, as variations in *temperature* will affect tendon prestress, and instrumentation such as vibrating wire gauges.

With regard to the recording of load-extension data Mitchell (1974) has recommended in practice that the details should be noted over four or five equal increments during loading or unloading cycles. However, Hanna (1969) considers that for a load extension diagram to be of "engineering use" it is essential that the load increments are small e.g. 10-20% of the working load (T_w). In this connection the Nicholson Anchorage Co. (1973) describe the stressing of test anchors at Greenwich, Connecticut, in six equal increments, after an initial seating load.

In general, it would appear that in any one stressing stage, at least five load increments should be monitored in routine production anchor tests. In special tests however, where a more basic analysis is being attempted, extensions should be monitored at load increments equivalent to 10% or less of the maximum load for each stage in the stressing investigation.

The various levels of measurement sophistication understandably reflect the time, money and personnel available. For *load* measurement, load cells have been installed on occasions to monitor anchor performance in both long and short term experimental programmes. Such cells are expensive, relatively fragile, and require regular care and maintenance if reliable performance is to be guaranteed.

Hanna (1973) discusses load measurement in considerable detail and this reference is strongly recommended to the interested reader, since many load cells are described which are applicable to anchor situations. By way of introduction Hanna indicates that the choice of load cell is usually controlled by three factors:

(*i*) cost,
(*ii*) environment e.g. access, temperature, humidity, susceptibility to damage, and
(*iii*) nature of load and accuracy required.

In summary, the major types of cell applicable to anchors are

(*a*) mechanical — based on
- proving ring systems (up to 2 000kN)
- force measuring blocks (up to 10 000kN), and
- cup springs (greater than 4 500kN),

(*b*) strain gauged elements (up to 5 000kN), and
(*c*) vibrating wire systems (up to 10 000kN).

Other methods involving photoelasticity, hydraulics and springs have also been used in practice. In all cases at least 1% accuracy is preferable and, regardless of the cell type, eccentric loading of the cell should be either assessed or prevented. The upsetting effects of eccentricity on load cell readings in the field are well illustrated by McLeod & Hoadley (1974) referred to later. It is also imperative that load cells are calibrated prior to and after use in the field.

An alternative and cheaper method for measuring anchor load is to use the prestressing equipment available, together with a destressing stool or chair. The method is applicable to both individual strands or the tendon as a whole. In both cases the principle is the same: a feeler gauge of specified thickness (0.1mm) is inserted under the anchor block or individual grip unit upon stressing through the stool to a certain load. The jack pump pressure at the earliest moment of insertion is recorded, and the minimum load at "lift-off" is thus evaluated. This initial residual load is commonly referred to as the "lock-off" load. The method is very common in practice, if somewhat crude, but an accuracy of $\pm$ 2% can be obtained by a careful operator. In the case of a single unit tendon the accuracy can be improved since the access to the tendon often permits the moment of "lift-off" to be registered by a dial gauge reading to 0.01mm (Fig. 8). In this connection it is noteworthy that the Czech draft code (1974) suggests that the jack calibration accuracy should be $\pm$ 1% as measured from two gauges. In the case of torquing, the lift-off load is related to the reading on the hand torque wrench when the locking nut is just in motion.

In a similar way to load measurement there are a number of levels of sophistication in measuring the tendon *extension*. The

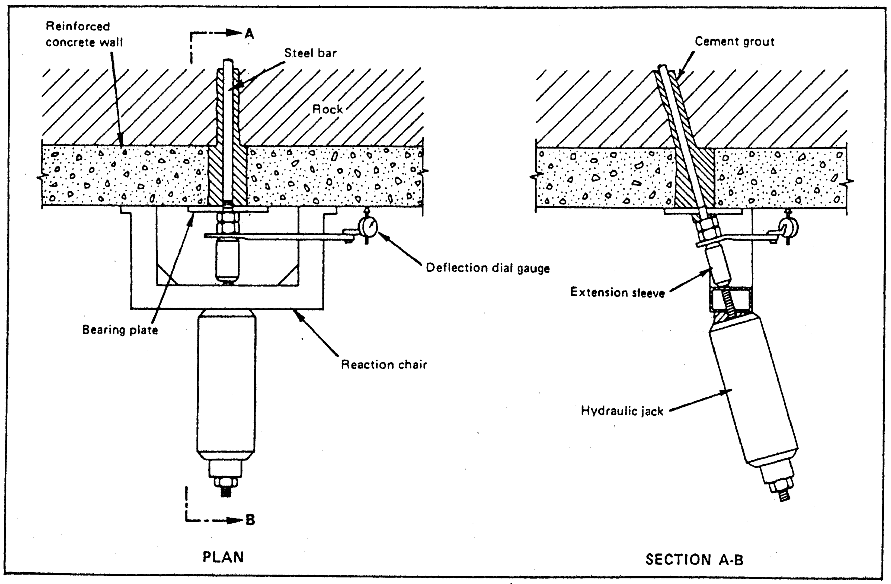


Fig. 8. Jack arrangement for mono-unit stressing and measurement of residual load
(after da Costa Nunes, 1966)

simplest, and least accurate, method is to measure the jack ram extension. Even if the correct null extension point is noted — when the jack has fully gripped the tendon or strand — there is no guarantee that the jack extension thereafter is the same as the strand extension. This is particularly the case where slip of the strand relative to the temporary grip wedges on or in the jack occurs. Usually, therefore, the true tendon extension is overestimated by this method.

A preferable method of measurement is the one whereby a piece of adhesive tape or some other means is used to mark all or a representative number of strands at some distance above the permanent load bearing plate. The difference between this distance in the unloaded condition and that measured at successive load increments provides the basic data for a load-extension graph. For single strand stressing, this distance is measured after lock-off at each load increment when the jack has been removed. Where a solid ram multistrand jack is used no lock-off or jack removal is required at intermediate load increments. In the particular case of hollow ram multistrand stressing it may be more convenient to measure the distance between the strand mark and the temporary load bearing plate. This approach permits an accurate measurement of gross extension without removal of the jack, provided that the distance between the temporary and permanent bearing plates is recorded. These distances are usually measured with a stiff steel rule, and an accuracy of $\pm$ 1mm can be attained. In this connection the Czech draft code stipulates an accuracy of $\pm$ 0.1mm.

More refined methods, often associated with special test anchors, include the use of dial gauges attached to a simply supported datum beam, in order to monitor movement of the temporary bearing plate. In very special cases, strain gauges of either mechanical or electrical types are installed.

Remote survey is the method of accurately determining the *movement of the permanent load bearing plate* and should be considered whenever possible. Knowing these movements, gross extensions can be corrected to give extension data dependent solely on tendon elasticity and fixed anchor movements. The Czech draft code stipulates that precise observations be made of vertical and horizontal movements of the structure and those of the rock. Also, the supports for all measuring instruments should be such that they are independent of the structure and not influenced by deformations produced by the prestressing operations. Usually for anchors in competent rock, and prestressed against a properly designed bearing plate system, top anchorage movements provide a very small proportion of the total tendon elongation. PCI (1974) recommends that bearing plate movements greater than 13mm should be taken into consideration. There is no disagreement with this statement but the authors believe that the significance of the actual value of movement can only be appraised when the free length of the anchor is known. For example, a plate movement of only 5mm would be sufficient to lose 20-25% of the initial prestress in the case of a free length of some 4m. In general however where the top anchorage movement represents less than 5% of the tendon extension it is usually ignored in the routine stressing of production anchors.

A direct, as opposed to interpretive, method of measuring the amount of *fixed anchor movement* involves the embedment of a wire in the fixed anchor. The wire is decoupled over the free length and extends out of the top anchorage assembly. With the wire loaded in tension, simply to keep it taut, the wire movement indicates fixed anchor movement (Fig. 9). Alternatively a redundant tendon unit may be used in place of the wire. This method has been used successfully by Liu & Dugan (1972).

Another parameter involving measurement on the tendon is the *strand wedge pull-in* at lock off. It should be emphasised however that this parameter is solely monitored as an indirect means of establishing the amount of lock-off loss and the resulting residual load at that time.

By careful measurement, the amount of strand wedge pull-in can be estimated to at least $\pm$ 1mm accuracy. With a multistrand stressing system the difference between extensions immediately before and just after lock-off is the amount of pull-in. With monostrand stressing, this amount can be readily judged by close observation of the strand near the jack nose during the lock-off operation.

If accurate monitoring is required it is considered advisable to measure in the field the amount of wedge pull-in and express it as a distance in mm, rather than as contributing a certain prestress loss, since the magnitude of this loss is directly proportional to the free length of the tendon in question.

This point can be illustrated by reference to details of two test anchors reported by Barron *et al* (1971) and shown in Table II.

Recent research conducted jointly with the Universal Anchorage Co. Ltd. has led the authors to conclude that the amount of wedge pull-in increases linearly with load in the strand, after a comparatively large initial pull-in at loads up to 30kN/strand. At 200kN/strand for 15.2mm Dyform, the amount may be as high as 6mm but mostly averages between 2-4mm in fair agreement with Fenoux and Portier (1972) who estimated 2-3mm.

It has also been found that the amount of wedge pull-in is less in monostrand compared with multistrand stressing. This is due to the practice of tapping home the individual grip wedges immediately prior to lock-off, in the monojacking operation.

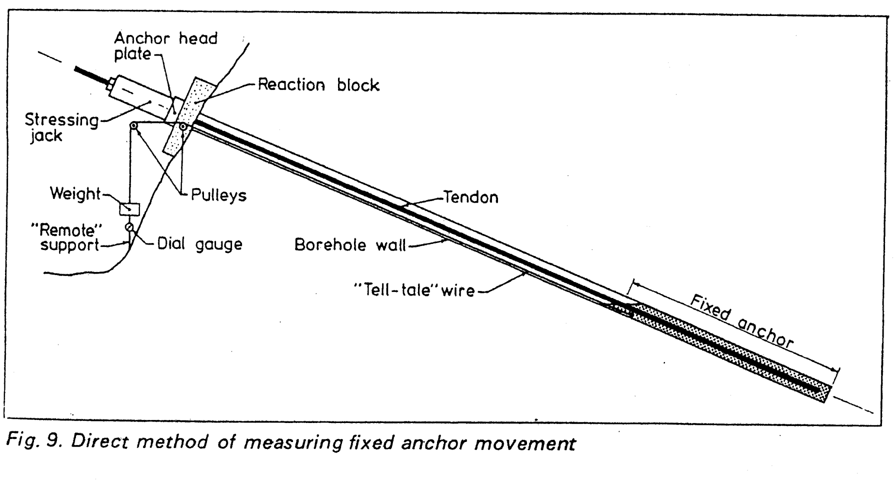


Fig. 9. Direct method of measuring fixed anchor movement

Presentation

All data relating to the stressing operation should be collected and carefully preserved. The list of items given in Table III is recommended for inclusion in a full stressing record. The data describe the rock anchor, jacking equipment and personnel, in addition to the load/movement readings which should be recorded during stressing, as already described.

There is limited published data on the stressing records recommended for torquing but a brief list of requirements is suggested in the ISRM draft document

TABLE II. LOCK-OFF LOSSES (after Barron *et al.* 1971)

Anchor	Free length	Applied load	Load after lock-off	Lock-off loss
1	3.96m	1 352kN	937kN	30.7%
2	10.67m	1 427kN	1 256kN	12.0%

"Suggested methods for rockbolt testing" (1974).

Although the final graph of load against extension will be based on corrected data, the original monitored data should also be presented on the stressing record since this information will not only provide historical data and facilitate back-analysis, but it will permit interpretation by other analysts.

When plotting the load against extension, another variable to define clearly is the point of origin of the graph. In most cases, the "zero" extension is recorded after the application of a certain seating load to the tendon, and not actually at zero load. The seating load is supposed to take up the slack in the tendon and jack, and compensate for friction and other losses in the jack/pump assembly thereby giving a more accurate measure of load-extension data.

For instance Larsson *et al* (1972) begin extension readings at 12% T_w — "to take up slack" — but also assume a zero extension of 2.5mm. On the other hand Longbottom and Mallet (1973) simply recommend starting at 10-20% T_w and N.A.C. Ltd. (1973) commonly begin reading from 10% T_t. The biggest seating load published to date is 25% T_w on anchors at the Frigate Complex, Devonport (Short 1975).

Most anchor codes e.g. Czechoslovakia and Germany advise reading from 10% T_w although P.C.I. (1974) recommends a start from 10% T_t. In the authors' view it would appear more realistic to try and gauge the actual seating load required for any particular anchor/jack assembly in order to optimise the measurement of residual displacements, e.g. due to fixed anchor movement at zero load. Nevertheless, the above recommendations are simple and although zero readings are extrapolated the method is probably adequate for routine short term testing.

The final presentation of load-extension should indicate the maximum possible measurement error in each parameter. Thus, when the line corresponding to the extension of the theoretical tendon length is drawn from the relationship

$$\text{extension} = \frac{\text{length} \times \text{load}}{E_{steel} \times \text{cross-section area}}$$

a meaningful and sensible comparison between actual and theoretical extension characteristics is permitted.

Similarly, a graph of load against time should have superimposed the theoretical relaxation curve for the tendon in question, as computed from the manufacturer's data. In this connection it is noteworthy that elevated temperatures occurring naturally or artificially e.g. adjacent to concrete nuclear reactor vessels, considerably increase the rate of loss. It is not generally appreciated that for wire and strand at 40°C the relaxation losses are at least 50% greater than at 20°C.

Interpretation

The fundamental property of the load-extension curve to be adjudged is its elastic behaviour, whether linear or non-linear. Due to limits on the accuracy of the monitored data collected, it is rare to obtain a perfectly linear plot, even for the most efficient anchor. However, if the deviation from linearity is both marked and consistent in trend, it is most likely that this is due to one or both of two factors:

(*i*) debonding in the fixed anchor at the grout/tendon interface, and
(*ii*) fixed anchor movement.

The latter phenomenon is unusual in all but the weakest rock strata, but unless some form of direct measurement (Fig. 9) has been incorporated, it can only be confidently dismissed by cyclically loading the anchor at least once to ensure that the load-extension characteristics of the anchor are reproducible.

Assuming allowance has been made for the top anchorage and fixed anchor movements, an interpretation can be made with respect to the amount of partial or total debonding within the fixed anchor zone, by calculating the effective free length to produce the true elongation of the tendon actually monitored at different loads. In practice, this analysis is facilitated by drawing construction lines, equivalent to the extension of different free lengths, on the load-extension graph (Fig. 10). During the initial loading of an anchor the characteristic trend of the measured load-extension curve is to approximate to lines of short free length initially, but to progressively intersect lines of longer free length with increasing load.

Cyclic loading not only highlights fixed anchorage movement, but generally facilitates back analysis, and confirms the degree of reproducibility of the elastic load-extension characteristics. It should be noted that when drawing straight, theoretical extension lines on such diagrams involving cyclic loading, a family of these lines should be drawn through each new zero load point, following the last loading cycle thereby eliminating the permanent set produced in the anchor by previous stressing.

A refined cyclic method is described by Fenoux and Portier (1972), which they consider to be systematic, easily conducted, and economic. The principle is that by careful destressing and restressing, without real change in tendon elongation, a value of load equivalent to twice the total frictional effects in the anchor can be deduced.

The method and interpretation is shown in Fig. 10. Assuming section X-Pm and X-Pb are sensibly parallel, the line X'-Y' represents the true values of loads corresponding to measured extensions since losses due to friction have been compensated. The point R, defined by X' and Y' and $\Delta l'$ gives the true final load sustained by the anchor. The method also permits lock-off losses to be readily determined.

Different failure modes within the anchor may be recognised during stressing and from close analysis of load-extension data. For example, a continuous cumulative permanent displacement indicated either by rapid load loss or from a cyclic loading plot usually indicates interface failure in the fixed anchor. Whether this is rock-grout or grout-tendon failure may be verified by loading each tendon unit with a monojack and comparing load-displacement characteristics.

Discrepancies between the theoretical and actual extensions are more often the rule than otherwise. Commonly, the amount of discrepancy permitted on any one site reflects the allowable anchor movements bearing in mind proximity of adjacent structures, the load safety factors, acceptable errors in measurement, and the consequences if failure occurs.

P.C.I. (1974) states however, as a general rule, that, where the measured and theoretical elongations have more than a 10% difference, "investigation shall be made to determine the source of the discrepancy".

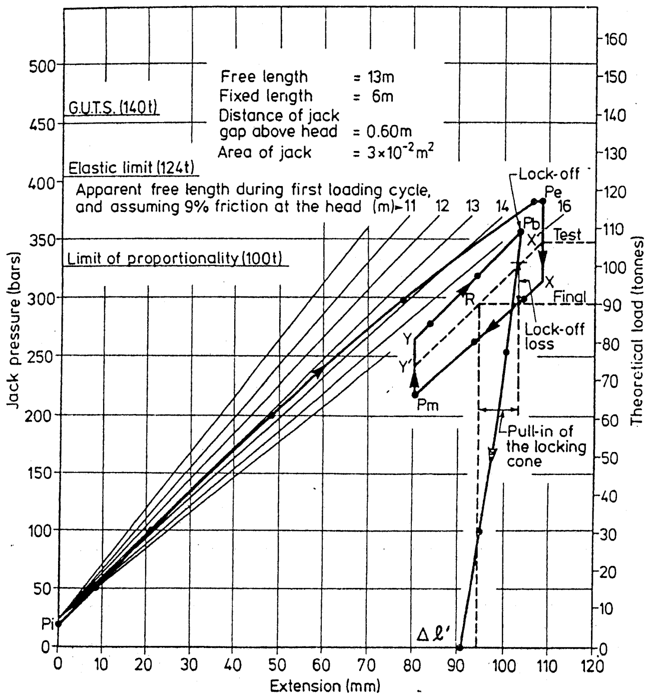
Numerous potential sources of error can be listed. For instances, as noted in Part I — Design, the E values given by the manufacturer for his prestressing steel, and based on short lengths may be in error.

Furthermore, Janische (1968) found that in extension measurements on long lengths of strand (100m) the extension for any particular applied load varied considerably, yielding E values in the range 180 000 — 220 000N/mm², averaging 196 000 ± 9 000 N/mm². Variations of this order were noted in strands for prestressing the Wylfa nuclear reactor, but even more relevant was the observation that the elongations of tendons were comparatively much greater than their constituent strands,

$$E_{strand} = 183\,000 - 195\,000\text{N/mm}^2$$
$$E_{tendon} = 171\,000 - 179\,000\text{N/mm}^2$$

TABLE III. RECOMMENDED ITEMS FOR INCLUSION IN STRESSING RECORD

General classification data			
Project	Contractor	Engineer	Inspector
Date	Time started	Time completed	Stressing personnel
Anchor No.	Free length	Fixed anchor length	Rock type
Tendon type	E value of steel	Working load (T_w)	Test load (T_t)
Jack type	Area of piston	Maximum rated capacity	Date of last calibration
Pump type	Pressure gauge range	Pressure gauge accuracy	Date of last calibration
Type of top anchorage assembly	Lock-off mechanism	Initial seating pressure	Strand pull-in
Data monitored during stressing			
Permanent bearing plate movement	Tendon extension	Jack pressure	Tendon pull-in at lock-off



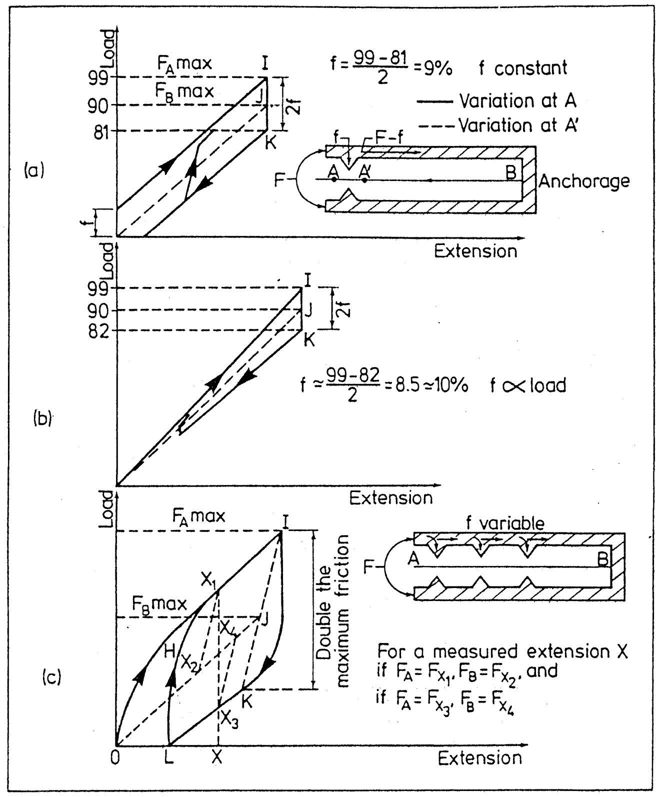
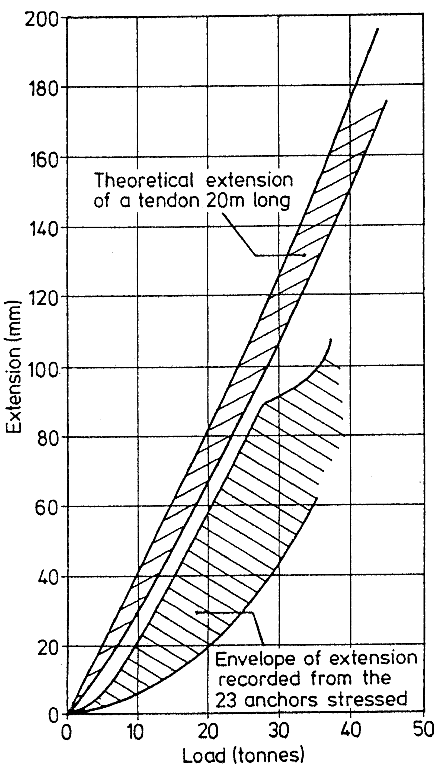


Fig. 10 (above) "La Méthode du Cycle" (after Fenoux & Portier, 1972)
Fig. 11 (below) Extension-load diagram illustrating the influence of friction (after Hennequin & Cambefort, 1966)
Fig. 12 (right) Influence of type of friction on form of load-extension graph (after Fenoux & Portier, 1972)



Janische attributed this to the possibility that with the stressing of multistrand tendons taking a longer period in the field than the testing of individual strands a plastic deformation occurs in the steel in the former application giving it a larger extension and so, apparently, a smaller E value.

Further information is supplied by Leeming (1974) who felt that instead of a possible maximum variation of $\pm$ 5% (three standard deviations from the mean), quoted by manufacturers, the total variation is more probably $\pm 7\frac{1}{2}$%. He also highlighted the difference between valuation of E when testing long and short specimens, by noting that the value for a 137m specimen was some 9% less than that for a short test piece of the same strand.

On a less sophisticated level, overdrilling or underdrilling of the hole will alter the free length in practice, and the accuracy and reliability of the recording — as distinct from the accuracy of the instrumentation — should always be considered.

Friction is another major source of error. Even if allowance for friction losses in the jack is made — some manufacturers quote a figure of 1% over the whole loading range — friction still occurs along the free length, particularly in long sheathed tendons surrounded by a protective grout surcharge column, and around the grip assembly of the top anchorage.

Such friction will act to reduce the measured extension simply by dissipating a proportion of the applied load which can act over the total tendon length. This results in an extension corresponding to a free length apparently less than is actually present. For example, Hennequin & Cambefort (1966) describe stressing details from a contract near Paris. They noted that the measured extensions were markedly lower than those estimated theoretically, and concluded that on average, only about 70% the total applied prestress was transmitted over the whole tendon length (Fig. 11). Such frictional losses can often be overcome simply by overloading by an amount particular to each anchor type.

Fenoux & Portier (1972) have also discussed friction in anchor systems and detail three types:

(*i*) constant value,
(*ii*) proportional to load, and
(*iii*) variable.

Each type acts on the load extension graph form as shown schematically in Fig. 12. Friction around the top anchorage is thought to have two distinct sources:

(*a*) between tendon and grout due to the bending of the tendon units under the bearing plate; this is of the order of 3-6% but can be alleviated by efficient lubrication; and
(*b*) between tendon and bearing plate, which may be up to 50% if the bearing plate and anchor block are badly positioned.

Commonly however, up to 10% total frictional losses in the top anchorage assembly may be expected.

Data on errors in prestressing measurements have been supplied by Longbottom & Mallett (1973). This information suggests that the difference between the observed and the theoretical force may be as much as 15% when dealing with rock anchors (Table IV).

TABLE IV. ESTIMATED ERRORS ASSOCIATED WITH THE PRESTRESSING OF ROCK ANCHORS

Source	Variation
Different type of manometer	$\pm$ 1%
Typical manometer error	$\pm$ 2%
Internal jack friction	$\pm$ 2%
Error in reading extension	$\pm$ 1%
Stress-strain & production tolerance of tendon	$\pm$ 6%
Calculation error	$\pm$ 3%

However since these values will rarely act together, a more likely error is estimated to be $\pm$ 5%.

Remarks

It is encouraging to observe the increasing use of prestressing to load anchors, thereby subjecting the overall system to a stress history and so improving subsequent performance in service.

Measurement of anchor load in the field is generally regarded as a simple operation, although more regular calibration of jack and pressure gauge equipment would undoubtedly lead to a higher degree or precision.

Accurate monitoring of extensions is the exception rather than the rule, because these measurements in the field are often considered to be awkward or time-consuming, and in any case, less important than the ultimate attainment of anchor loads.

Insufficient attention is paid to the interpretation and consideration of the monitored load-extension data. As a result there has been little progress in the understanding of basic anchor behaviour with particular regard to component movements of the overall anchor system.

In spite of the background technology available in the field of prestressed concrete, there is currently a lack of awareness concerning the sources of discrepancies between the theoretical and field results for rock anchors.

During the stressing operation safety standards would be considerably improved by the use of protective barriers and warning signs.

TESTING

Precontract component testing

Prior to use on site, manufactured components such as the tendon and top anchor assembly units should be tested in an independent testing establishment to guarantee component safety factors and ensure efficient performance. Alternatively, it may be acceptable on occasions when employing a standard form of component, to obtain test certificates from the manufacturers in order to facilitate or substantiate the choice of appropriate components.

With regard to the testing of the tendon steel, manufacturers should be requested to supply load-extension characteristics for each reel or batch of material delivered. In the UK, testing and the supply of test certificates and stress/strain diagrams should be carried out in accordance with BS 2691 "Steel wire for prestressed concrete" and BS 3617 "Seven-wire steel strand for prestressed concrete". Useful guidance will also be found in FIP "Recommendations for approval, supply and acceptance of steels for prestressing tendons".

To confirm that the specified minimum stress/strain values have been met, the permanent extension method is used by manufacturers in routine testing. In the case of steel the non-proportional elongation, quoted in the definition of proof stress*, is equal to the permanent elongation which remains after the proof load has been removed. Provided the permanent elongation is less than that defining the proof stress (e.g. less than 1.0%), then the specification has been met.

The normal test procedures is as follows:

(*1*) An initial tensioning stress of 10% of the specified minimum tensile strength is applied to the test piece (gauge length = 0.6m)
(*2*) The extensometer is set at zero,
(*3*) The load is increased to the specified proof stress, and held for 10 seconds,
(*4*) The total extension is noted,
(*5*) The load is reduced to just below initial stress, and then increased to the initial stress,
(*6*) The permanent extension is noted, and
(*7*) By plotting the results, the modulus of elasticity can be calculated making use of the proportional stress/strain relationship.

Very little has been published on the effect of low temperatures on the ultimate strength of steel tendons. For 1 570/1 720 N/mm² steel wire a slight increase in strength occurs as the temperature falls. Sub zero temperatures (Farenheit scale) would, however, be necessary to produce a 5% increase in tensile strength, without the elongation being affected.

Apart from any question of the effect of temperature change on mechanical properties, it is useful to remember that a change in temperature of 1°C will produce a change in stress in a fixed wire of the order of 1.9 to 2.2N/mm². For applications where a significant range in temperature may be recorded in the anchorage zone, it is clear that provision of a coefficient of thermal conductivity will facilitate the analysis of test results.

Data on fatigue resistance of prestressing steels is also limited, and the manufacturers do not supply endurance diagrams for their products as a routine procedure. As Longbottom (1974) has stated, the provision of such data requires the investigation of a series of stress ranges each about a series of mean stresses (see for example FIP "Recommendations for approval, supply and acceptance of steels for prestressing tendon").

In practice ground anchors are seldom subjected to pulsations of stress of any magnitude relative to the prestress, but if in a particular case significant alternations of stress are predicted, these can be accommodated in the design of the tendon and top anchor components, and by prestressing to the service load plus the fluctuating stress. The successful application of prestressed concrete and steel in railway and highway bridges in resisting impact and fatigue (Lee, 1973) is ample evidence that satisfactory solutions can be produced. Eastwood (1957), Baus & Brenneisen (1968) and Edwards & Picard (1972) have described the fatigue strength of rolled threaded bar anchorages, prestressing strand and some types of wedge grip top anchorages.

With reference to the top anchorage system, which may be regarded as a combination of the tendon, grips, anchor block and load bearing plate or waling acting together, both the grip components which secure the bar, wire or strand within the top anchorage and the complete top anchorage assembly should be tested in accordance with BS 4447 "The performance of prestressing anchorages for post-tensioned construction". Useful guidance is also given in FIP "Recommendations for acceptance and application of post-tensioning systems".

The British Standard describes three methods of testing prestressing anchorages for prestressing applications.

(*i*) Test of load efficiency of the anchored tendon, consisting of a short term static tensile test on the proposed anchorage attached to the tendon. The load efficiency

$$\left(\frac{\text{test failure load}}{\text{average UTS of tendon}}\right)$$

must not be lower than 92%, where the average UTS of the tendon is determined in accordance with BS 18 "Methods of testing metals" and BS 4545 "Methods for mechanical testing of steel wire", as appropriate.

The characteristic strength of the anchored tendon is calculated as the characteristic strength of the tendon times the actual efficiency. In this test limits of percentage elongation are also stipulated.

(*ii*) Test of dynamic behaviour of the anchored tendon where a fluctuating force between 0.60 and 0.65 fpu at a frequency not exceeding 10Hz is applied for a minimum of 2×10^6 cycles. Loss of initial cross-sectional area of the tendon due to fatigue must not exceed 5%. It is considered that this dynamic test is only relevant where the anchor application involves fluctuating stresses which are transmitted to the tendon.

(*iii*) Test of force transfer to the load bearing block, consisting usually of a short term static compressive test on the complete top anchorage assembly to ensure that the load bearing block can continuously support a minimum force of 1.1 fpu.

It is suggested that the test of force transfer to the load bearing block of the form described in BS 4447: 1973 should be applied to all types of top anchorage assembly so that bearing plates, walings, and the additional reinforcment placed in a concrete diaphragm wall are subject to the same design and performance checks that are currently applied to reinforced concrete load-bearing blocks in prestressed concrete. The design of load-bearing blocks is currently covered by the recommendations of CP 115 "The structural use of prestressed concrete".

Bearing in mind the application of rock anchors in excavation engineering it is noteworthy that the German DIN 4125: 1972 stipulates that the anchor head should be in a position to bear secondary stresses imposed by unforeseen flexure with adequate safety e.g. by deformation of the excavation structure or by angle deviation from the planned axial direction of the tendon.

With reference to jacking equipment the authors are unaware of any codes which specify test procedures. In the light of discussions with jack and pump manufacturers it is recommended that all jacks and ancillary equipment should be tested in the factory to a proof loading or pressure equivalent to at least 1.25 times the rated capacity. Overloading above the maximum rated capacity must not be permitted in the field and the choice of jack should be such that the rated capacity can accommodate 85% of the characteristic strength of the largest tendon (largest tendon unit for a monojack) in the group of anchors being considered.

When new equipment is delivered certificates concerning proof testing, internal losses and load-pressure conversion charts or factors should be supplied by the manufacturer.

To ensure that the monitored data is accurate, pressure gauges, like the equipment, must be well maintained and calibrated regularly. It is recommended that the gauges should be calibrated for the start of every contract, and then checked on site against a control gauge at monthly intervals or every thirty production anchors depending on usage. Independent calibration of jack equipment is recommended every three months.

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*The proof stress is defined as a stress which is just sufficient to produce under load, a non-proportional elongation equal to a specified percentage of the gauge length. The 0.1% proof stress is therefore obtained from the graph by marking off parallel to the straight line (or line of proportionality) a second line at a distance equal to 0.1% extension. The point of intersection of this offset line with the curve gives the proof stress.

Acceptance testing of production anchors

SHORT TERM ACCEPTANCE tests on all production anchors highlight potential difficulties pertaining to service behaviour and provide measured safety factors related to the design working load. These tests are associated with the initial stressing operations and normally include quality control observations over a period of up to 24 hours.

As a first priority, the testing procedure must yield a measured safety factor as determined by overloading for a short period. Such overloads, however, must be compatible with the allowable stresses and safety factors permitted in the country concerned. The relevant details are discussed in Part 1—Design (Table XV)§, and these suggest an encouraging trend towards standard safety factors throughout the world at the present time.

To check the measured performance against that predicted by calculation, it is essential that a load-extension graph be plotted for each anchor, in the manner discussed in Part 3—Stressing§.

In addition, an attempt should be made on either preliminary test anchors or on early production anchors to obtain an indication of fixed anchor movement, since this information allows the analyst to assess a component of permanent displacement which in turn permits a reasonable estimate of the degree of debonding, if any.

Finally, it is necessary to ensure that the service load locked-off after stressing is stable. The alternative methods employed in practice are monitoring loss of prestress with time, and monitoring creep displacement of the anchor with time.

Acceptance testing of temporary anchors in Germany is covered by DIN 4125 (1972). This standard concentrates solely on soil anchors but it is considered relevant to describe the recommendations in this review since the tests are rigorous and have been carefully devised. In addition, important principles are introduced which may well be stipulated for rock anchor testing in the future, particularly in the case of highly weathered materials, or fractured rock masses with interstitial clay.

Each production anchor is subjected to an initial load T_o equivalent to 0.1 T_y (T_y = yield strength of the tendon, assumed to be the 0.1% proof load which is equivalent to 83.5% fpu) after which it is stressed in one operation to 1.2 T_w (T_w = specified working load) and held for at least 5 minutes in non-cohesive soils, and 15 minutes in cohesive soils, whilst tendon extensions are monitored at the top anchorage (Type I test).

Where the spacing between grouted fixed anchor zones is less than one metre, a check on interaction may be necessary. This will involve several adjacent anchors being loaded and observed simultaneously.

For the first ten anchors, and thereafter one in ten of all subsequent anchors, a slightly more rigorous approach is taken and the extensions must be monitored from a fixed datum, at load increments equivalent to 0.4 T_w, 0.8 T_w, 1.0 T_w and 1.2 T_w, due account being taken of strand slippage (Type II test). At the maximum test load the observation times are as stated for the Type I test, and on destressing to the initial load (T_o), an indication of the permanent extension is provided. In the case of prestressed anchors, the working load is subsequently applied and locked-off.

For the Type II test the results are plotted as shown in Figs. 13 a & b and at 1.2 T_w (Point X) where unloading is first carried out, the elastic component (Δ_{xe}) and permanent component (Δ_{xp}) of the total displacement Δ_x can be distinguished. The curve, $T_o\ X_e$ in Fig. 13b is taken as an approximate path for the elastic displacement.

It is further specified that at least 5% of the anchors must be tested up to 1.5 T_w, bearing in mind that the maximum test load cannot exceed 0.9 T_y (Type III test). At the maximum test load the observation times are as stated for the Type I test.

In general, the acceptance regulations are met for Type I tests, when at a load of 1.2 T_w the displacements stabilise within the observation time, and when the elastic extension curve lies between two boundary lines plotted on the load-extension graph.

The upper boundary line (a) corresponds to the tendon extension equivalent to the free length plus 50% of the fixed anchor length, or 110% of the free length in the case of a fully decoupled tendon with an end plate or nut. The lower boundary line (b) corresponds to 80% of the free length of the tendon. It is important to emphasise that account should be taken of sources of error as already described in Part 3—Stressing, and generally it is merely recommended that the observed load-extension line should be compared with the calculated theoretical extension due to the elastic extension of the free length of the tendon.

The permanent displacement, calculated with the aid of the approximate elastic extension line $T_o\ X_e$ should conform closely with the results of the basic test but the permanent displacement (Δ_p) must not be greater than that observed for the basic test over the load range T_w to 1.2 T_w (see "Special test anchors").

For Type II and III tests, the acceptance conditions are met when at maximum test load the creep displacement stabilises within the observation time, and when the free length of the tendon and permanent displacement have been proved in a similar way to the Type I test, through back-analysis of the observed extensions.

In the case of permanent anchors, generally regarded as having a service life in excess of two years, current thinking in Germany is illustrated in the Draft DIN 4125 (1974) which has been published for comment. In this document, it is suggested that each anchor should be tensioned from the initial load T_o to 1.5 T_w, with a preliminary reading at T_w. The anchor is then unloaded to T_o, the permanent elongation is measured, after which the anchor is retensioned to T_w.

For the first ten anchors, and thereafter one in every ten, the test load is to be applied at stages, 0.4 T_w, 0.8 T_w, 1.0 T_w, 1.2 T_w and 1.5 T_w. Unloading then occurs in the same stages to T_o, before T_w is re-applied.

The displacements occurring at 1.5 T_w should be measured 1, 2, 3, 5, 10, and 15 minutes after lock-off. The specified observation period of 15 minutes should be extended if displacements occurring between 5 and 15 minutes are greater than 0.5mm, and monitoring should be continued until a clear estimate of the creep rate is possible. An observation period of 5 minutes is considered sufficient in frictional soils, provided that the displacements are smaller than 0.2mm.

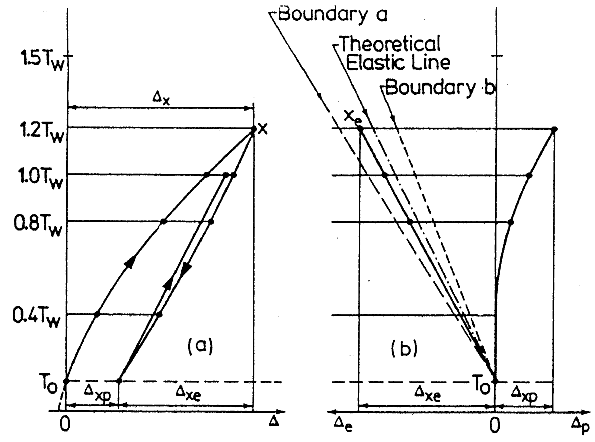
The results of these measurements compare favourably with test anchor results, and a comparison of elastic extensions and the creep rates is usually sufficient. The acceptance test is considered to be satisfactory if the elastic extensions fall between the two boundary lines (a) and (b) previously described. Further, the creep should be less than 2mm at a load of 1.5 T_w (see "Special test anchors").

With regard to acceptance testing in France, Bureau Securitas (1972) states that overloads of 1.2 T_w and 1.3 T_w should be applied to temporary and permanent production anchors, respectively. In the case of permanent works, where anchors are in service for more than 18 months, it is further suggested that 5% of all anchors could be tested to 1.5 T_w. No maximum permissible stress is specified for the steel tendon, but the Bureau warns that great viligance is required when the elastic limit is exceeded (83.5% fpu), and normally the test would be stopped if the extension reached 150% of the extension at the 0.1% proof stress.

Accurate estimation of load losses e.g. through friction, is emphasised when plotting load-extension data, and an accuracy of not less than 3% is stipulated for manometers. Tensioning by stages starts at 0.15-0.20 T_t and at least five stages are recommended in order to draw accurately the load-extension diagram. In frictional soils the test load is held for 1-2 min. During this time the displacement should not exceed 1mm and the observed free length of the tendon, based on back analyses of the load-extension diagram, should lie between the theoretical free length and the theoretical free length plus 50% of the fixed anchor length. For anchors with a working life less than nine months, an observed free length equivalent to 90% theoretical free length is accepted. If these tests are satisfactory the service load is locked off plus an allowance for losses.

In cohesive soils, the test load is held for five minutes, and the curve of displacement with respect to time should compare closely with the performance of anchors subjected to creep tests (see "Special test anchors"), in addition to complying with the extension criteria described above.

In Czechoslovakia the draft standard for prestressed rock anchors (Klein, 1974) stipulates the test loading of all temporary anchors to 1.2 T_w in cycles as shown in Fig. 14: (a higher test loading for permanent anchors is expected but yet to be specified). The maximum permissible stress in the steel tendon is the 0.2% proof stress which is equivalent to 87% fpu. The observed displacements are separated into elastic and permanent portions and the observed elastic displacement at 1.2 T_w should lie between the boundary lines (a) and (b) as specified in DIN 4125 (1972). The permanent displacement due to the increase in load from T_w to 1.2 T_w should not exceed by more than 10% the permanent displacement obtained in the basic anchor test over the same load range (see "Special test anchors"). With regard to creep under a constant service load, it is stipulated that the displacement should not exceed 0.135mm/m of free tendon for every tenfold increase in time. To simplify measurements on production anchors, the draft Code suggests that constant time intervals should be chosen for the observations, and that changes in displacement must not increase in these time intervals. For the specific time intervals in Fig. 15, the displacements must be less than 0.02mm/m of free tendon, and for acceptance testing, the total period of observation must be at least ten minutes. Finally, the creep displacement is compared with the results



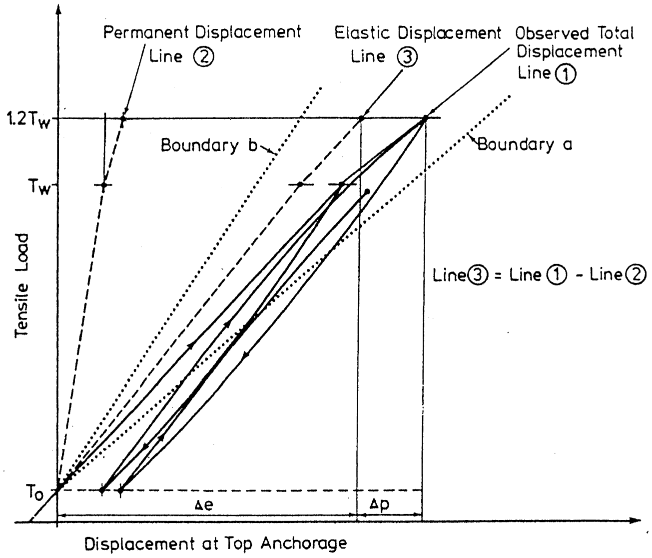


Fig. 13 (above). Stressing programme for acceptance tests
(after DIN:4125-1972)

Fig. 14 (right). Working diagram for acceptance tests
(after Czech Draft Code, 1974)

from basic tests.

On every site, it is specified that the first three production anchors and 5% of the remainder should be subjected to a more rigorous test loading to 1.4 T_w and 1.5 T_w for temporary and permanent works, respectively. A service life of less than two years is considered temporary.

The FIP final draft (1973) suggests that the tensile stress in the tendon must never exceed 0.9 T_y (75% fpu, assuming T_y is equivalent to the 0.1% proof stress) and all production anchors should be tested to 1.2 T_w and 1.3 T_w for temporary and permanent works, respectively. A service life of less than two years is considered temporary.

Details of the acceptance test are shown in Fig. 16 and extensions are monitored at load increments equivalent to 0.15-0.20 T_t. For soils and rock not susceptible to creep the test load is held for 2-5 min., and the anchor is accepted if:

(*i*) no noticeable displacement (approx. 1mm) is observed during the period of observation, and

(*ii*) the measured total displacement at the top anchorage is in reasonable agreement with the results of the "extended acceptance" test (see below).

For soils and rocks susceptible to creep, the observation period at constant test load must be long enough to enable the relationship between creep displacement and time to be ascertained, and a minimum period of five minutes is specified. The anchor is locked off at the required service load if the measured total extensions and creep displacements conform closely to those of the "extended acceptance" test.

At the beginning of a contract, it is recommended that between three and ten production anchors should undergo an "extended acceptance" test. The stressing programme is shown in Fig. 17 and this test is applied to approximately 10% of the production anchors constructed thereafter.

In this test the anchor is accepted if:

(*a*) the displacement of the anchor under test load has stabilised within the observation period, and

(*b*) the measured elastic tendon extension corresponds to the calculated elastic extension.

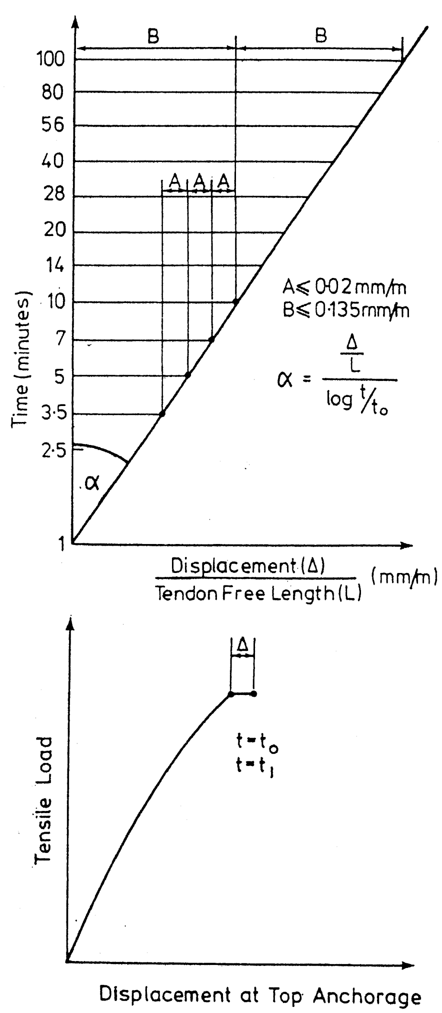


Fig. 15. Working diagram for acceptance criteria for creep displacement
(after Czech Draft Code, 1974)

In connection with (*b*), the calculated free tendon lengths based on the observed elastic extension of the tendon must not exceed the free tendon length plus 50% of the fixed anchor length or 110% of the free length, or be less than 90% of the free tendon length.

Current practice in Italy has been revealed by Arcangeli and Tomiolo (1975) of Rodio. From an initial seating of 0.10 Rak (Rak = characteristic tensile rupture stress of steel), extensions are recorded at 0.15 Rak intervals up to 0.85 Rak. This load is applied usually for 10-15 minutes until creep losses in the steel are negligible (less than 0.1mm in 5 min.). Thereafter following destressing down to 0.3 Rak in 0.15 Rak increments, the anchor is restressed to 0.85 Rak before locking off at the required load.

All anchors are tested in this way to provide a measured safety factor of 1.3 and to compensate for frictional effects and lock-off losses the procedure of Fenoux and Portier (1972) is used. In general the results from each site or geotechnically distinct anchor area are analysed and compared statistically to verify the service conditions of the installations.

In the United States, PCI (1974) suggests the test loading of every anchor to at least 1.15 T_w. During the test loading the prestressing load in the tendon should not exceed 80% fpu. The maximum test load is usually applied for up to 15 minutes, and extensions should not diverge by more than 10% from the calculated values, otherwise an investigation is required. For temporary anchors in rock (up to three years where there is no apparent danger of corrosive attack) it would appear that extension measurement is not usually required. With reference to losses of prestress during service, PCI states that meaningful lift-off

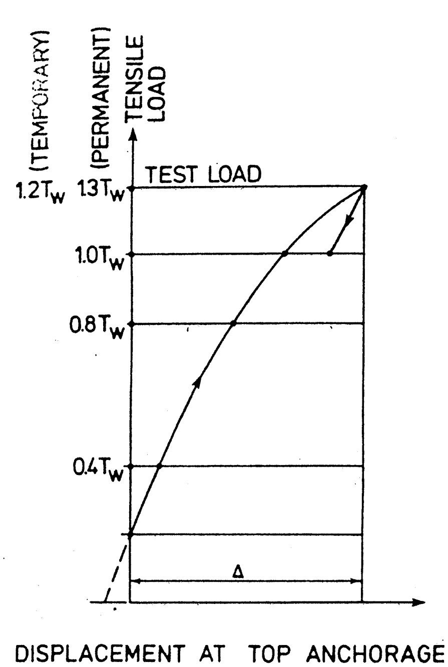


Fig. 16. Stressing programme for acceptance tests (after FIP Draft, 1973)

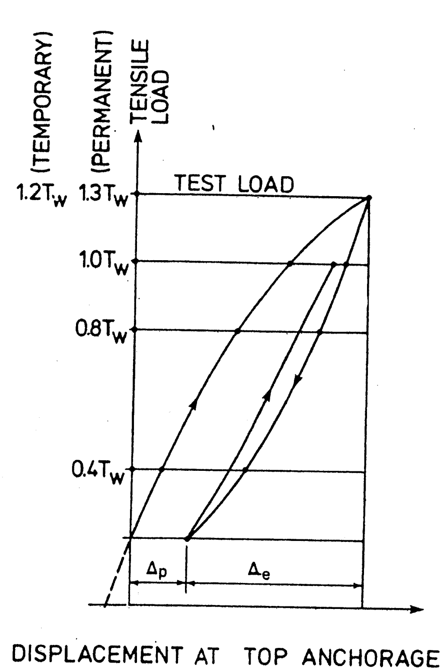


Fig. 17. Stressing programme for extended acceptance tests (after FIP Draft, 1973)

checks can be carried out after 24 hours and that in most cases of rock anchors the primary time dependent loss is steel relaxation.

In Britain, CP 110 (1972) permits tensile testing to 80% of the characteristic tensile strength (fpu) of the steel tendon and the authors' recommendations on safety factors related to acceptance tests are reaffirmed in Table V.

The above recommendations are gradually being adopted in Britain, but for temporary and permanent anchors the most common method in current practice consists of test loading in increments up to 1.25 T_w with a minimum observation period of five minutes at this maximum test load. The anchor load is then reduced to zero before restressing in increments up to a lock-off load of 1.10 T_w (Littlejohn, 1970). Tendon extensions are monitored but since the movement at the top anchorage during the initial loading stage may comprise fixed anchor displacement, tendon extension, wedge pull-in, bearing plate and structural movement, the interpretation and analysis of the data are usually restricted to the load-extension graph obtained during the second loading cycle.

The observed extension should compare closely with the value estimated from the free length of the tendon and the permissible discrepancy on any site varies, the value often being directly related to the accuracy of the measurements and parameters used in the calculation.

In order to give some insight into service behaviour of anchors in Britain, emphasis to date has been placed on monitoring loss of prestress with time which is a simple alternative to the German and French practice of measuring creep displacement. A lift-off check is carried out immediately after lock-off to measure the actual residual load in the anchor. This residual load, which is usually 1.10 T_w, is then checked after 24 hours. Bearing in mind the errors in measurement referred to in Part 3 "Stressing", a loss of up to 5% is acceptable in practice. If the load is less than 0.95 T_w, the anchor should be replaced or otherwise dealt with as agreed with the Engineer.

Where the anchor load lies between 0.95 T_w and 1.05 T_w, the tendon should be retensioned to 1.1 T_w, and retested after a further period of 24 hours. If the anchor, after three such tests, still fails to retain a load of 1.05 T_w, the anchor should be derated or replaced, as agreed with the Engineer. In the former case it is recommended that the load be reduced until no prestress losses are observed, over a period of at least one week, based on daily readings. A safe working load may then be established equal to 62.5% and 50% of this reduced stable load for temporary and permanent applications, respectively (see Table V).

If a component of a multi-unit tendon fails during the stressing stage, a reduced anchor capacity, in proportion to the number of components left, may be agreed with the Engineer, unless the individual components have stresses in service which are below the limits specified. In this situation it may be possible to upgrade the load in each component to compensate to some extent the loss of the redundant component. For example, a tendon consisting of 10 No. 15.2mm Dyform strands might be required in Britain for a permanent anchor with a working load of 1 400kN. In this case each strand would be resisting only 140kN (46.7% fpu) and could therefore be upgraded to 50% fpu (150kN) to give a safe working load on the anchor of 1 350kN if one strand failed. The same approach may be applied if gripping wedge failure occurs and fresh wedges cannot be fitted.

In South Africa the Code of Practice "Lateral Support in Surface Excavations" (1972) stipulates a test load of 1.25 T_w for every prestressed anchor. This load is maintained for a period of not less than ten minutes to test the anchorage, and is then reduced to a load of 1.1 T_w.

Between 24 and 48 hours after lock-off, the tendon is retensioned until the anchor block just lifts off the permanent load bearing plate, and the residual load at this point is recorded. If this residual load is greater than 0.80 T_w but less than 1.05 T_w the tendon should be retensioned to 1.10 T_w and then retested 24 hours later. If after three such retests at 24 hour intervals, the tendon still fails to maintain a load above the working load it should be condemned and replaced, or derated as approved by the Engineer.

In the case of tendons which are to be permanently protected against corrosion by grouting, they may be grouted after the 24-48 hour test but not later than seven days after this test. Such fully bonded tendons are not subjected thereafter to further tests.

In this connection Parry-Davies (1968) emphasises the advantages of leaving the tendon ungrouted over a period of, say, 12 months in order to facilitate tests.

He further reasons that since the working strain in the tendon is only a small fraction of the ultimate strain, a generous safety factor against "catastrophic collapse" is provided. Since a small extension of the tendon supporting a basement excavation, for example, will relieve excess forces

TABLE V. RECOMMENDED SAFETY FACTORS AND TEST FORCES IN BRITAIN

Item	Anchor category	
	Temporary (Life < 2 years)	Permanent
Design or working force (T_w)	62.5% fpu	50% fpu
Test force (T_t)	78% fpu	75% fpu
Measured safety factor	1.25	1.5

which may build up, the safety of the system lies not so much, therefore, in the ratio of actual stress in the tendon to ultimate stress, but in the ratio actual strain to ultimate strain.

Remarks

The value of overloading an anchor to give a measured load safety factor and to impose a stress history which can improve subsequent behaviour, is widely appreciated.

With reference to the interpretation of load-extension data, however, important differences in acceptance criteria are apparent, and clearly use of boundary lines which reflect permissible discrepancies must not be employed inflexibly or without a basic understanding. For example, an anchor with negligible fixed anchor movement but which has apparently debonded along half the fixed anchor might be judged acceptable. An anchor in which only 80% of the applied load has seemingly been transferred to the fixed anchor zone might also be considered satisfactory. These two extremes illustrate that ill-considered use of the extension criteria could be misleading and potentially dangerous expecially when considering the corrosion risk (debonding at tendon interface) or the overall stability (inadequate load beyond potential failure plane).

It is suggested that while load-extension boundary lines are favoured in practice, care and attention is required in interpretation. To alleviate problems of interpretation, the authors recommend that all production anchors should be subjected to at least one stage of cyclic preloading. The analyst should then concentrate on the load-extension plots of the second and any subsequent load cycles, from which most of the initial non-recoverable movements have been removed e.g. plate "bedding-in". In this way, closer correspondence between theoretical and observed extensions should be apparent, easing the analysis of anchor performance.

If discrepancies are still considered significant, on-site discussions are necessary to decide the appropriate action, which may lead to acceptance, derating or replacement of the anchor, depending on the circumstances and the consequences of failure.

Long-term monitoring of selected production anchors

Long-term monitoring over periods in excess of 24 hours checks service behaviour and acts as a control to verify that anchor performance is satisfactory. Furthermore, the collection of data relating loss of prestress or creep displacement to time, type of rock, and anchor load and geometry, will improve understanding of the service behaviour of anchors and could well lead to future refinements in design. In the short term, such data establish if overload allowances applied to the working load at initial load-off, are adequate and realistic.

Long-term losses within the anchor are due to a combination of steel relaxation and anchor creep (see "Service behaviour of production anchors"). The relaxation characteristics of prestressing steel are well known and readily available from manufacturers. Less is known about creep in rock anchor systems largely because basic information regarding the magnitude and distribution of stresses in the fixed anchor zone is not available. Nevertheless, in weathered rock or fractured rock with clay infill, creep losses may be significant and an estimation of the amount to be expected should be gauged from test anchors installed well in advance of full-scale production.

Where test anchor results are not available and the rock is of poor or variable quality, it has been recommended in Britain that periodic checks of anchor stress should be carried out on production anchors as follows:

(*i*) 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.

(*ii*) On a large contract where the consequences of failure are severe, the first ten anchors should be checked weekly for one month, then monthly for the next three months.

(*iii*) Subject to satisfactory results after four months, 5% of all production anchors should be checked at six months, and again at 12 months.

The permissible variation in anchor load is usually $\pm$ 0.1 T_w and restressing is only carried out after careful consideration. For example, in the case of a retaining wall tied back by several rows of anchors installed in a weak shale, loss of prestress due to consolidation of the shale in the fixed anchor zone may be observed without accompanying movements of the retaining wall. In these circumstances remedial measures may not be required.

Bureau Securitas (1972) considers that although the ground anchor tie-back system is now a safe and thoroughly tried and tested method, it is absolutely necessary to plan a monitoring or control procedure which will detect possible failures in time. As a result, periodic monitoring of permanent anchors for a period of at least ten years is compulsory in France.

During the first year, monitoring takes place at intervals of three months, at six month intervals in the second year, and thereafter at yearly intervals. As already indicated, the Bureau classifies anchors according to basic geometry and type of ground at the fixed anchor. In each category, the minimum number of anchors to be monitored is:

10% of production anchors (total installed, 1-50)

7% of production anchors (total installed, 51-500)

5% of production anchors (total installed, over 501)

The Bureau further states that the control apparatus must be reliable, simple, and have an adjustable sensitivity; it need not be a measuring device, and a limit device capable of detecting load losses of between 15 and 25% is adequate. In this connection the authors would add that the control apparatus should also be capable of monitoring prestress gains, particularly in the case of anchors for retaining walls.

In selecting the production anchors to be observed, the FIP Draft Recommendations (1973) indicate that for "extended acceptance" tests, an initial number of 3-10 anchors should be monitored, followed by a percentage of all others—usually 10%.

It would seem that the South African Code "Lateral support in surface excavation" (1972) recommends the most rigorous approach at the present time, namely that each anchor should be tested at the following intervals after stressing unless it is to be permanently protected against corrosion by grouting:

(i) Not less than 24 hours and not more than 48 hours.

(*ii*) Seven days if the 24/48 hour test is satisfactory.

(*iii*) One month if the 7 day test is satisfactory.

(*iv*) Monthly intervals for the first six months and thereafter at three monthly intervals if the first monthly test is satisfactory.

After 12 months, all tendons remaining in service should be tested at intervals laid down by the Engineer; in no case should such intervals exceed six months.

As an alternative to the measurement of loss of prestress, creep displacement may be monitored since test results in Germany and France have indicated that, under constant load, the stabilisation of displacements of the tendon, the fixed anchor, and the ground in the vicinity of the fixed anchor proceeds linearly, when displacement increments Δ are plotted against the logarithm of time. The displacement increments increase with increase of load and when the stresses at the fixed anchor/ground interface approach the ultimate strength of the ground the displacements accelerate in relation to time on a semi-logarithmic scale.

On the basis of these observations certain authorities clearly consider that the displacements may be considered stabilised when, for a constant applied load, the displacements are successively smaller, or that they do not increase more than linearly when plotted on a semi-logarithmic scale against time (see "Special test anchors" in Germany).

In current practice where an attempt is made to gauge the long-term performance, this commonly consists of one lift-off check but the time of observation varies considerably e.g. at 24 hours (Buro, 1972, Mitchell, 1974), 72 hours (Australian Standard, 1973), 7 days (Gosschalk and Taylor, 1970, Chen and McMullan, 1974) or 28 days (Morris and Garrett, 1956). Certainly few production anchor checks are as thorough as those executed by McLeod and Hoadley (1974), all anchors being checked at 3, 7 and 21 days, and 100 out of 1 800 by load cell each day for six months.

Remarks

For economic as well as operational reasons the time involved for the stressing and control of anchors on a construction site should be minimised. The question remains whether it is realistic, or indeed possible, to judge the long-term load holding capacity of the anchor on the basis of a short-term test. Although prestress losses due to lock-off, friction and steel relaxation are predictable, the creep behaviour of different types of rock due to anchor loading is largely unknown. Field experience indicates that such losses may be significant in heavily weathered rock, or fractured rock with clay infill.

A prestress loss of up to 5% in 24 hours or a creep displacement of up to 4mm in 72 hours has been used as an upper threshold of acceptability in practice, but these figures are rather arbitrary and should be regarded as provisional.

Only when creep losses are monitored over long periods for a variety of anchor loads and geometries, and for a wide range of classified rock types, will an accurate predictive capacity be available. In the meantime, therefore, it is recommended that periodic checks of anchor stress or creep displacement should be carried out on anchors whenever possible, and every effort should be made to publish the field data obtained in the form of case histories.

Special test anchors

In cases where there is no prior experience of anchoring in a particular rock, special tests should be carried out to optimise or check design assumptions, and also to pinpoint any important practical considerations relating to construction and stressing. In rocks susceptible to creep, the duration of the test should be sufficient to establish a safe working load for minimal creep, or to permit assessment of an overload allowance or restressing programme to accommodate creep losses. In all rocks an attempt should be made to test these anchors to failure so that actual safety factors can be determined.

It is interesting to note that Štefanko and de la Cruz (1964) use the terms "Dynamic" and "Static" when summarising types of test, as follows:

(i) **Dynamic:** progressive and continual loading of the anchor until failure is induced. Such tests provide data on the ultimate capacity of certain elements e.g. grout/tendon bond or rock /grout bond, and usually these ultimate values are simply factored to provide suitable working parameters. In Europe such tests are referred to as "basic" or "suitability" tests, and they must be carried out on specially installed anchors which will not subsequently be employed in service.

(ii) **Static:** load-time relationships are determined to investigate the anchorage effectiveness. Such "decay" tests are more time-consuming and costly, and are not yet as widely conducted as would appear advisable. Anchors undergoing this type of test can be used as production anchors if required.

In Germany, the basic suitability of any ground anchor system is ascertained from *basic tests* on at least three anchors in recognised types of ground (DIN 4125: 1972). The construction, testing and subsequent excavation of the anchors must be monitored by a recognised professional institution which also classifies the ground.

Approximately one week after grouting, stressing is carried out and top anchor displacements are measured from a remote datum for different loads above the initial seating load ($T_o \not> 0.1\ T_v$). Proceeding from this initial value T_o, load increments equivalent to 0.15 T_v are applied until failure, or until the yield stress of the tendon is reached (Fig. 18a). After the load increment equal to 0.3 T_v and thereafter at each successive higher load increment, the tendon is unloaded to T_o to provide data on permanent displacements, and to enable calculation of the effective free length of the tendon. The top anchorage displacements occurring at loads below T_o are not measured.

Before each unloading operation displacements are observed under constant load in non-cohesive soils until the movements stop, but for at least five minutes. At 0.6 T_v the load is held for 15 minutes and the associated displacement Δ_1 is noted (Fig. 18b). At 0.9 T_v the observation time is increased to at least one hour (associated displacement = Δ_2). In cohesive soils the observations at 0.6 T_v and 0.9 T_v are continued until the displacement during the last two hours is less than 0.2mm. If the working load (T_w) is less than 0.6 T_v the maximum applied test load should be at least 1.5 T_w (observation time at least 1 hour), and the working load (T_w) should be applied for at least 15 minutes.

All applied loads should ideally be measured with the aid of load cells, and the displacements via dial gauges accurate to 0.01mm.

During the basic test the actual shape, length and character of the complete anchorage is determined by excavation after the stressing stage. Particular attention is paid to the grout-tendon interface and central position of the tendon in the grouted fixed anchor zone.

On plotting the load-displacement results, the measured displacements at the top anchorage are divided as for acceptance test analysis into elastic (Δ_e) and permanent (Δ_p) portions (Fig. 18). For a specific anchor load (Point *X*) as shown in Fig. 18a, the total displacement is Δ_x with an elastic component Δ_{xe}, and permanent displacement Δ_{xp}. In Fig. 18b the elastic and permanent components of displacement are plotted for each load increment, and the failure load is readily observed as being 0.94 T_v. However in this case, the upper load limit specified might be 0.9 T_v, if this was the maximum load step at which the displacements under constant load clearly stabilised during the observation period.

If the upper load limit is not reached in the basic test, the largest test load applied is taken as the upper limit, but never greater than T_v.

Following the basic test a report is produced which describes fully the ground conditions, anchor characteristics and stressing results. The upper load limit is quoted for the observed free and fixed anchor lengths. In the case of the observed free length, the curve of the elastic displacement Δ_e (Fig. 18b) should lie between the boundary lines (*a*) and (*b*) (see "Acceptance testing").

It is noteworthy that any anchor system chosen for a contract must also be subjected to three *suitability tests* at the construction site, if the local ground is different to that of the basic test, or if the drilling procedure or borehole diameter is substantially different from the basic test. In contrast to the basic tests however, the anchors in suitability tests are not excavated after stressing.

For permanent soil anchors in Germany (Draft DIN 4125: 1974) the basic tests are similar to those already described for temporary anchors with the following variations.

The tensile load is applied in the stages specified in Table VI commencing at T_o. When each stage of loading has been reached, the load is subsequently reduced to T_o, so that elastic and permanent displacements can be judged.

The anchors should be stressed to 0.9 T_v if the failure load of the grouted fixed anchor is not reached at an earlier stage.

In order to determine the limit load for minimal or acceptable creep (T_k), the displacement must be measured under constant loading prior to the removal of each load e.g. after 1, 3, 5, 10 and 30 minutes, and recorded as shown in Fig. 18c. The required minimum observation periods are shown in Table VI but these periods can be extended if necessary until the trends are clear and the creep *K* Δ related to the displacement of the anchor, can be determined. In addition, it is recommended that if the creep is greater than 1mm for a course grained soil, then the longer mini-

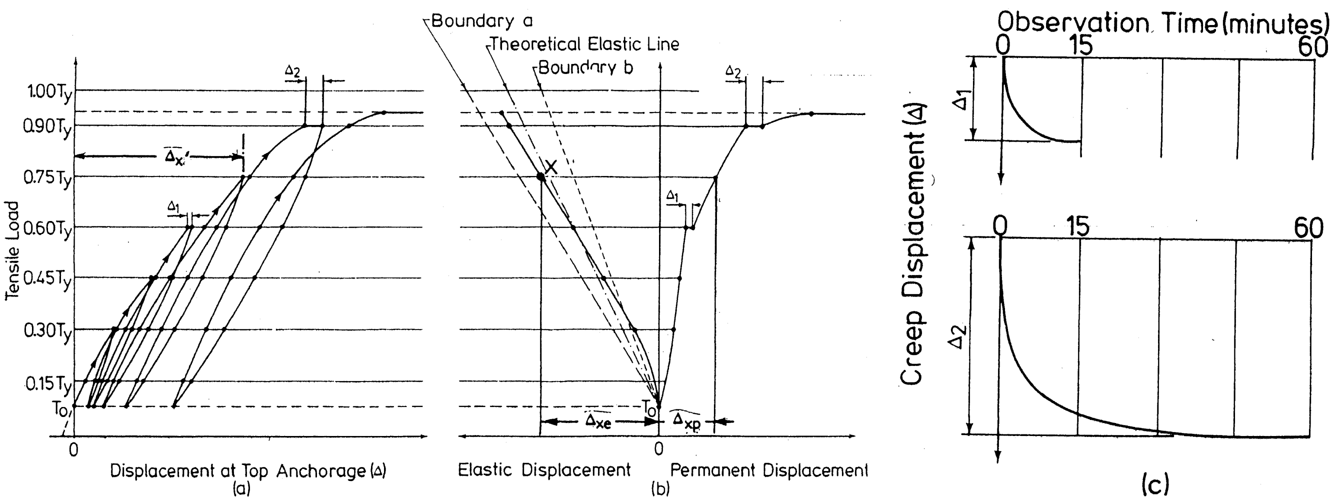


Fig. 18. Stressing programme for basic or suitability tests

(after DIN 4125-1972)

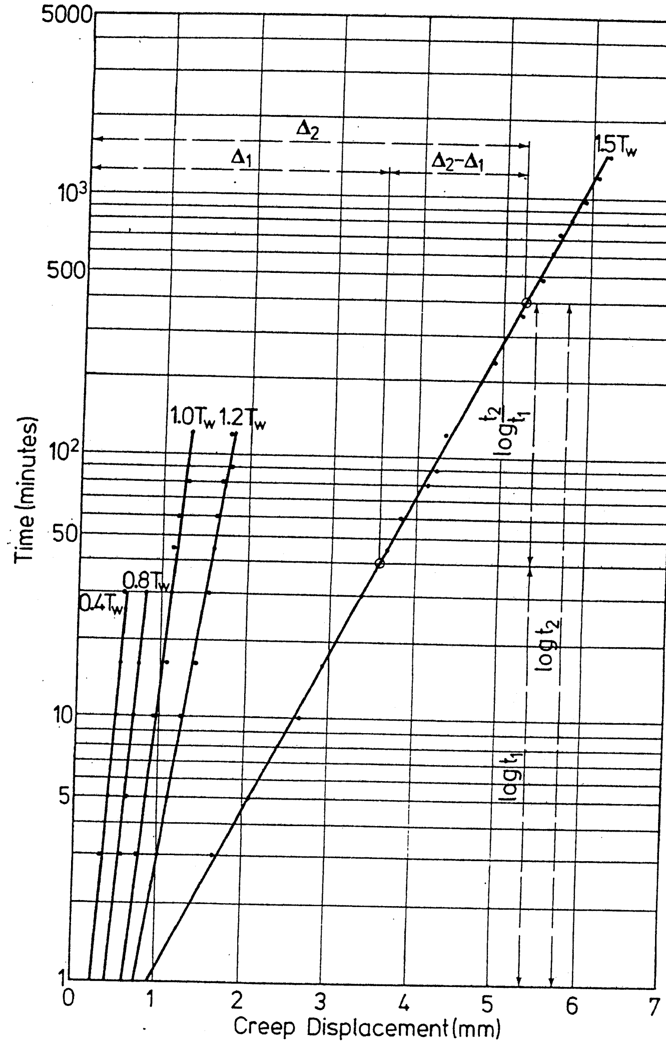


Fig. 19 (left). Method for the determination of K_Δ
(after Draft DIN 4125-1974)

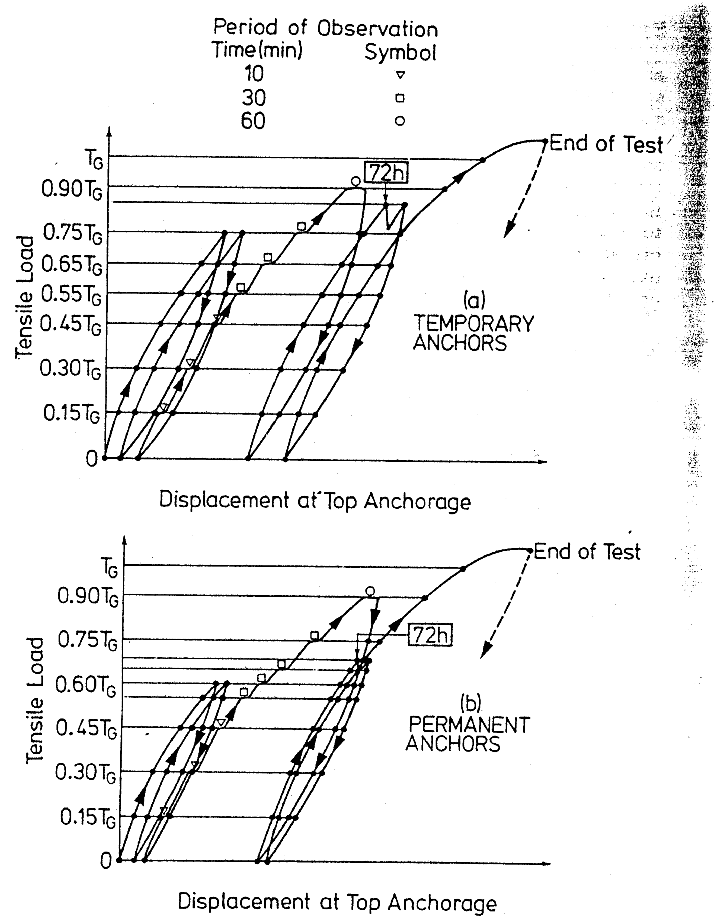


Fig. 21 (above). Stressing programmes in soils where anchor behaviour is known
(after Bureau Securitas, 1972)

mum observation periods for fine grained soils should be adopted.

In accordance with Fig. 19, the creep $K\Delta$ is calculated as follows

$$K\Delta = \frac{\Delta_2 - \Delta_1}{\log t_2/t_1} \qquad \ldots (1)$$

The values of $K\Delta$ are evaluated at different stages of loading and recorded as shown in Fig. 20, and by definition the limit force T_k corresponds to a creep $K\Delta$ of 2mm. After this stage of the test, the

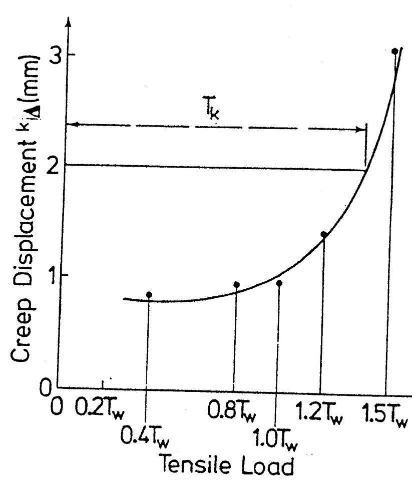


Fig. 20. Method for the determination of limit force T_k (after Draft DIN 4125-1974)

anchor is subjected to twenty load cycles (range—0.3 T_y to 0.6 T_y) and the extension at the maximum and minimum loads must be measured at least after every five cycles. Pauses for observation of extensions should not be included for intermediate cycles. Subsequently, the load is reduced to T_o, then increased to 0.6 T_y with an appropriate observation period.

A similar approach is applied to the suitability tests on the construction site, where it is specified that the tests should be carried out in the most unfavourable soil conditions. The loading stages are shown in Table VI with the basic observation periods. Subsequently, twenty load cycles (range—0.5 T_w to 1.0T_w) are carried out. Only when these rigorous tests have been completed satisfactorily, is the permanent service load locked-off.

In both the basic and suitability tests the maximum permissible load specified for the anchor is the smallest of the following values:

(i) $T_y/1.75$ (T_y = guaranteed yield strength of the tendon),
(ii) $T_f/1.75$ (T_f = failure of the bonded fixed anchor), and
(iii) $T_k/1.50$ (T_k = limit force for creep $\not>$ 2mm according to equation (1) above.

In France, basic test anchors as detailed by Bureau Securitas (1972) are categorised by geometry and ground type, and the minimum number of test anchors is related to the number of production anchors in one category, as shown in Table VII.

TABLE VI. LOAD STAGES AND OBSERVATION PERIODS FOR BASIC AND CONSTRUCTION SITE SUITABILITY TESTS (after Draft DIN 4125: 1974)

Stage of loading		Minimum period of observation	
Basic test $T_o \not> 0.1\,T_y$	Suitability tests* $T_o \not> 0.2\,T_w$	Coarse grained soils	Fine grained soils
$0.30\,T_y$	$0.40\,T_w$	15 min	30 min
$0.45\,T_y$	$0.80\,T_w$	15 min	30 min
$0.60\,T_y$	$1.00\,T_w$	1 hour	2 hours
$0.75\,T_y$	$1.20\,T_w$	1 hour	3 hours
$0.90\,T_y$	$1.50\,T_w$	2 hours	24 hours

*If the working load is not known at the time of the test or the upper limit load is uncertain, it is recommended that smaller load stages should be selected

TABLE VII. MINIMUM NUMBER OF TEST ANCHORS RELATED TO NUMBER OF PRODUCTION ANCHORS (*after Bureau Securitas, 1972*)

No. of test anchors	No. of production anchors
2	1— 200
3	201— 500
4	501—1 000
5	1 001—2 000
6	2 001—4 000
7	4 001—8 000

As an example, if a project involves 500 anchors, of which 300 are inclined and 200 are vertical, then two categories are present, based on geometry. If, in addition it is known that 200 are inclined into gravel, 100 are inclined into clay, and all the vertical anchors are installed in clay, then a total of three categories must be recognised as follows:

200 inclined/gravel—2 test anchors
100 inclined/clay —2 test anchors
200 vertical/clay —2 test anchors

Bureau Securitas states that the test anchors must be similar to the categories of the production anchors envisaged. This requirement concerns the method of construction and anchor geometry although it is accepted that the tendon can be of larger capacity to permit a high test load to verify a high safety factor or possibly induce failure of the grouted fixed anchor.

For ground *where previous anchoring knowledge is available and there is no risk of creep*, the Bureau states that it is possible with confidence to load the test anchor up to the anticipated working load of 0.75 T_g and 0.60 T_g for temporary and permanent anchors, respectively (Figs. 21a & b). T_g is the elastic limit of the tendon and equivalent to 83.5% fpu.

In order to eliminate from the start parasitical movements such as tendon slack and plate "bedding-in", two successive load cycles are recommended (Table VIII) with pauses only to record the extensions. On completion of the second loading cycle, stressing is carried out in stages, with observation periods under constant load at each stage to permit creep observations.

At each of the stages, displacement measurements are taken every 30 seconds during the first two minutes, every minute between the second and tenth minutes, and every two minutes thereafter. After the one hour observation period at 0.9 T_g, the load is removed completely in stages and then reapplied in stages up to the lock-off load with pauses only for displacement readings. Allowing for lock-off losses, the initial residual load must not be lower than 0.80 T_g and 0.65 T_g for temporary and permanent anchors, respectively, to accommodate tendon relaxation and ground creep. After 72 hours the load is reapplied and the increment of top anchorage displacement to regain the initial residual load is monitored. This displacement should be less than 4mm.

The anchor is then unloaded completely prior to a final stressing operation where the load is increased in load increments as before until failure occurs or the extension of the steel tendon is equal to 150% of the extension at the 0.1% proof stress (Fig. 22). The test is now complete and the load is reduced to zero before the anchor is abandoned.

Where the *ground conditions are not known, or prior experience of anchoring in the ground does not exist* failure may occur at a load below 0.9 T_g. In these circumstances the maximum test loads for the first three load cycles which are carried out without pauses are lower (see Figs. 23a & b). During these three cycles, displacement measurements are taken each time the load is changed by 0.05 T_g. With regard to creep or relaxation losses measured over 72 hours, the initial residual loads locked-off are 0.85 T_g and 0.7 T_g for temporary and permanent anchors, respectively. If the displacement required to regain the initial residual load is less than 4mm, then the test proceeds as already described. If however, the displacement is greater than 4mm indicating creep of the grouted fixed anchor, a second 72 hour check is carried out (Fig. 23c). If the displacement now required to regain the initial residual load is less than 1mm, the test may proceed as already described. If however the creep displacement exceeds 1mm, the Engineer may continue the present test or order a second test anchor and repeat the test but with a lock-off load at least 30% lower. It is important to note that the Bureau Securitas recognises that the figures of 4mm and 1mm are rather arbitrary and should be regarded as provisional values only.

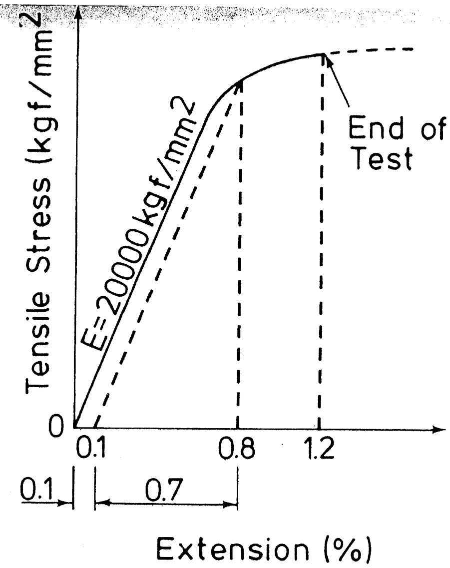


Fig. 22. Typical stress-strain curve for prestressing steel (after Bureau Securitas, 1972)

If failure of the first test anchor occurs at load T_f during one of the intermediate test stages, tensioning of the second or subsequent anchors should follow the principle illustrated in Fig. 24 for temporary anchors. The basic approach is identical to that already described in Figs. 21 & 23 but this time the load increments are related to T_f and not T_g.

With regard to the scatter of results, if all test anchors fail in the fixed anchor zone or the test is stopped due to excessive extension, the ultimate loads should not differ by more than 30%, with respect to the smallest ultimate load. Where the scatter is above this figure, a rigorous analysis of the reasons is necessary.

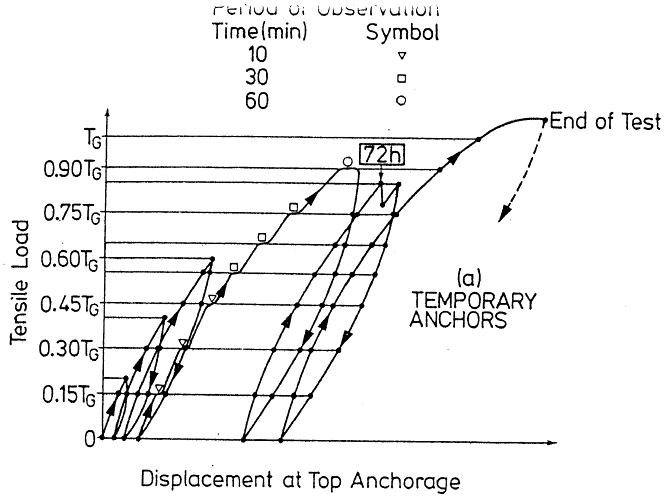
The maximum working load is specified equivalent to 0.67 T_{min} and 0.50 T_{min} for temporary and permanent anchors, respectively (T_{min} = minimum ultimate load for test anchors). If none of the test anchors fails, the maximum working load must not exceed 0.75 T_g and 0.60 T_g for temporary and permanent anchors, respectively. These working loads can only be applied of course to test anchor results where the creep displacement criteria already described have also been satisfied.

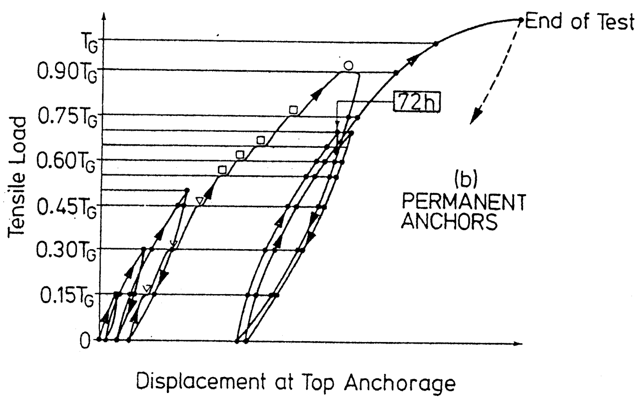
The Czech Draft Code (1974) relates to

TABLE VIII. RECOMMENDED LOAD INCREMENTS AND PERIODS OF OBSERVATION FOR BASIC TEST ANCHORS (after Bureau Securitas, 1972)

Temporary anchors			Permanent anchors		
Load increment		Period of observation (minutes)	Load increment		Period of observation (minutes)
Initial two load cycles*	Third load cycle		Initial two load cycles*	Third load cycle	
$0.15\,T_g$	$0.15\,T_g$	10	$0.15\,T_g$	$0.15\,T_g$	10
$0.30\,T_g$	$0.30\,T_g$	10	$0.30\,T_g$	$0.30\,T_g$	10
$0.45\,T_g$	$0.45\,T_g$	10	$0.45\,T_g$	$0.45\,T_g$	10
$0.55\,T_g$	$0.55\,T_g$	30	$0.55\,T_g$	$0.55\,T_g$	30
$0.65\,T_g$	$0.65\,T_g$	30	$0.60\,T_g$	$0.60\,T_g$	30
$0.75\,T_g$	$0.75\,T_g$	30		$0.65\,T_g$	30
	$0.90\,T_g$	60		$0.75\,T_g$	30
				$0.90\,T_g$	60

*For these load cycles, there is no pause other than that necessary for the recording of extension data





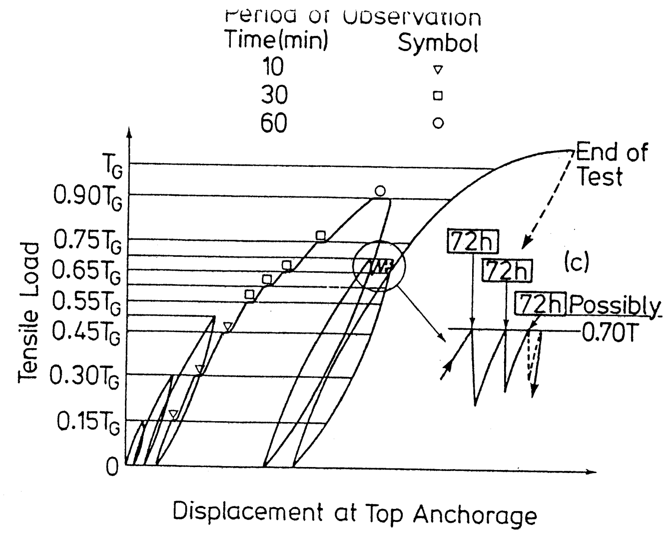


Fig. 23 (a & b), (above). Stressing programmes in soils where anchor behaviour is not known. (c), (above right), Stressing programme where creep replacement is excessive
(after Bureau Securitas, 1972)

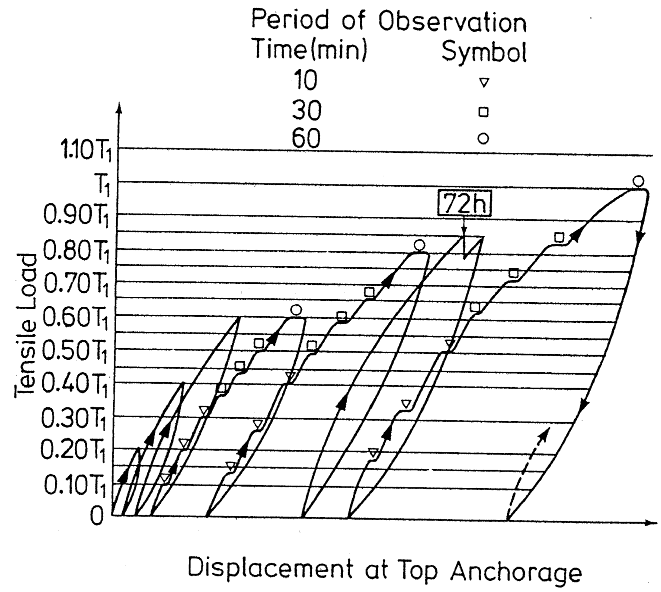


Fig. 24 (right). Temporary anchors: stressing programme for the second test anchor after failure of the first at load T_1
(after Bureau Securitas, 1972)

both DIN 4125 (1972) and Bureau Securitas (1972). A basic anchor test is recommended for each type of anchor which includes subsequent excavation. No details are provided however on acceptance criteria related to test load or creep displacement. It is noteworthy however that a prime objective of the basic tests is to confirm design safety factors of 1.5 and 1.6 for temporary and permanent anchors respectively.

In the case of ground where anchor behaviour is unknown, the FIP Draft Recommendations (1973) suggest special long-term tests using restressable top anchorage heads. Where it is necessary to observe the variation of load over a period of time, lift-off checks or the use of load cells is an acceptable practice but monitoring the displacements of the fixed anchor and the top anchorage is also recommended to facilitate analysis of anchor behaviour. No specific guidance is provided by FIP on acceptance criteria in relation to these long-term tests.

In order to optimise the design and construction of anchors in a particular type of ground, a minimum of three test anchors has been recommended in Britain (Littlejohn 1970). The fixed anchor length is varied, and for a particular ground condition and anchor position an estimate of the magnitude of the side shear and end-bearing component of the ultimate load is ascertained, if failure is achieved at the ground/grout interface, by plotting the failure load against fixed anchor length. In addition to establishing actual factors of safety, the validity of empirical design rules can be checked.

When assessing the suitability of a proposed anchor system for a contract the minimum data required from test anchors on the construction site are shown in Fig. 25. In current practice the number of test anchors usually ranges from one to three.

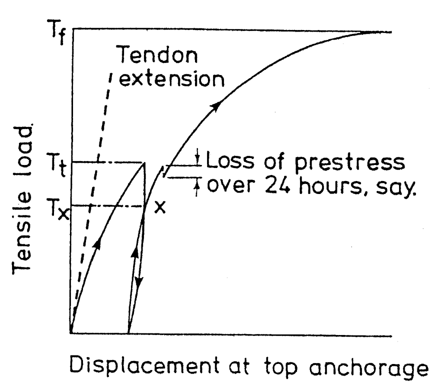


Fig. 25. Minimum stressing programme for test anchors
(after Littlejohn, 1970)

Assuming that the basis of the production anchor design is to be checked before the contract, then the tendon strength at 80% fpu should be sufficient to test the anchor to give a measured safety factor of 2, or in the case of ground susceptible to creep, the safety factor may be in the range 2.5 to 4.0 depending on duration of service.

The anchor is first loaded incrementally up to 1.25 T_w or 1.5 T_w, depending on whether the production anchors are temporary or permanent, respectively, since this will represent the normal load test (T_t) in practice. After an observation period of five minutes the anchor is destressed, the load-extension graph being plotted for the full cycle. On restressing to T_t, the load at the cross-over point T_x is noted if additional fixed anchor displacement is required to mobilise T_t. In this situation it is considered that for the value T_t shown, T_w should have a value less than T_x in order to minimise loss of prestress particularly if the production anchors are subjected to cyclic loading.

The test anchor is then locked-off at T_t, and left for at least 24 hours to measure loss of prestress. Thereafter, the duration of the test should be as long as possible since it serves to indicate whether creep of the anchor is likely to be serious during service. The test anchor is finally stressed to failure, or 80% fpu, in an attempt to establish the actual factor of safety of the anchor. In addition, the ultimate bond values attained at the ground/grout and grout/tendon interfaces, respectively, are compared with the values assumed in design.

During the second loading cycle up to T_t the load-extension curve should compare closely with the theoretical extension due to the free length of the tendon. Bearing in mind the known sources of error in materials and measurements (see Part 3—Stressing), British engineers normally accept a discrepancy of $\pm$ 5% between observed and calculated results. Where discrepancies approach $\pm$ 10% a detailed examination of the results is undertaken to more fully interpret and explain the observed behaviour.

Remarks

The major advantages of test anchors may not be fully appreciated at present, but it is important to note that these tests can provide:

(*i*) confirmation of specified safety factors (in the case of test anchors taken to failure, the validity of empirical design rules can be verified since ultimate values are determined),

(*ii*) a check on the suitability of the proposed anchor system for the construction site,

(*iii*) advance warning of construction difficulties, and

(*iv*) a predictive capacity concerning time-dependent phenomena, where the test loading is observed over a significant period of time.

A survey of the most influential recommendations reveals no general agreement on the number of anchors to be tested, but it would appear that a minimum of three precontract anchors should be tested for each geotechnically distinct rock type likely to be encountered on site. One test anchor in each group should have sufficient strength of tendon to fail, or at least test severely, the bond at the rock/grout interface.

The time and expense involved in test anchor programmes warrants careful planning, execution and analysis, otherwise the potential advantages above will not be fully realised. In this respect the value of practical guidelines, agreed nationally or internationally, cannot be over-emphasised.

Monitoring of the overall anchor rock structure system

Monitoring the complete anchor/rock/structure system can improve basic understanding of anchor behaviour and act as a quality control by checking that the overall engineering solution adopted is satisfactory during service. This form of monitoring covers the behaviour of the structure, rock mass and anchors, whether individually or in groups, and facilitates study of the short and long-term interaction between different components of the complete system.

This type of monitoring is particularly important in excavation engineering e.g. stabilisation of opencast pit slopes, where it is advantageous to observe overall behaviour of the anchored slope as excavation proceeds.

Clearly, monitoring of overall behaviour is expensive and time consuming and in practice may be restricted to major mining operations or prestigious civil engineering projects. Nevertheless, only by such studies in the field can important concepts relating to overall stability and group effects be verified.

SERVICE BEHAVIOUR OF PRODUCTION ANCHORS

Introduction

This final section deals with the long-term behaviour of rock anchors in service, with particular reference to the load-retaining characteristics of anchors for periods in excess of 24 hours after final stressing. Disproportionately little field research has been conducted into this aspect of rock anchors, despite its important bearing on various fundamental aspects of design, stressing and testing. This dearth of data—including attempts to correlate anchor performance up to, and after, the first 24 hours of service— is due partly to the fact that the potential yield of such results is not fully and widely appreciated, and partly to the time and expense required to set up and pursue a programme of long-term monitoring.

This lack of knowledge exists despite the fact that all engineers associated with anchor contracts have a responsibility to be concerned with long-term behaviour and would benefit from such information. For example, the designer would be able to "feed back" performance data collected during service into future designs and thereby optimise such parameters as overload allowances and safety factors. Likewise a prospective client could be accurately and confidently informed by the consulting engineer of how the anchors installed at his expense would perform after installation. Furthermore the presence of a comprehensive "data bank" would permit engineers to judge at an early stage whether anchors being monitored were, in fact, acting satisfactorily or in a potentially dangerous manner. Long-term monitoring also permits correlation of anchor load fluctuation and structural movement e.g. the performance of a diaphragm wall tied at several levels (Saxena, 1974; Littlejohn and MacFarlane, 1974; and Ostermayer, 1974).

In the following review the authors firstly discuss information relevant to the relaxation and creep properties of steel tendons, since tendon characteristics alone can be assessed accurately under controlled test conditions in the laboratory. In analysing subsequent field observations, this knowledge can be used to isolate and recognise other time-dependent variables influencing the service behaviour of full-scale anchors. Finally, a limited number of case records is presented to illustrate different aspects of field anchor performance.

Time-dependent behaviour of steel tendons

Assuming that no structural movement occurs, relaxation or creep of the tendon will result in loss of prestress during service. Relaxation is regarded as the decrease of stress with time while the tendon is held under constant strain, whereas creep is the change in strain of the tendon with time under constant stress.

Relaxation

According to Antill (1965), both relaxation and creep lead to approximately the same loss of prestress in practice for a given tendon under constant temperature, but the computation of such loss from relaxation characteristics of the steel is preferred by steel manufacturers because of its closer simulation of actual working conditions in the field of prestressed concrete construction. In this connection, prestressed rock anchors may be regarded as a similar application and long-term relaxation properties for the tendon permit prestress losses and therefore residual loads to be determined in practice.

Details of tendon relaxation have already been presented in Part 1—Design, of this review. However, it is relevant at this point to consider the major conclusions reached by Antill (1965), Bannister (1959), and Mihajlov (1968):

(*i*) Early conceptions that relaxation values at 1 000 hours are equivalent to ultimate values are completely erroneous. Currently, long-term relaxation is understood to mean the stress loss after 100 000 hours, and Antill (1965) suggests that the ultimate loss of stress is about twice the loss at 1 000 hours at 20°C, for all common values of initial stress. In fact, the loss at 100 hours is twice that at 1 hour, 80% of that at 1 000 hours and 40% of the loss at 30 years, according to long-term tests on various types of steel.

(*ii*) The introduction of "stabilised" wire and strand has reduced load losses from 5-10% in ordinary stress relieved steel; to 1.5% at 75% GUTS (= guaranteed ultimate tensile strength) and 20°C.

(*iii*) The rate of load relaxation increases rapidly with temperatures above 20°C.

(*iv*) The rate of relaxation varies with the initial stress, the actual rate being a function of the type of steel. Relaxation from initial stresses up to 50% GUTS may be considered negligible in practice.

In fact for initial stresses greater than 0.55 f_y the relationship is

$$\frac{f_t}{f_i} = 1 - \frac{\log t}{10}\left(\frac{f_i}{f_y} - 0.55\right)$$

where f_t = residual stress after time t
f_i = initial stress.
f_y = 0.1% proof stress at working conditions and temperatures, and,
t = time in hours after application of initial stress

(*v*) With initial stresses of 70% GUTS, restressing at 1 000 hours reduces the amount of ultimate relaxation to almost one-quarter of its normal value and for initial stresses of 80% GUTS the reduction is about one half. Insufficient information is available at present to permit firm conclusions with respect to the effect of restressing at 100 hours.

(*vi*) An unduly high order of accuracy in determining relaxation losses is often not warranted since the significant parameter in practice is the residual stress in the tendon.

(*vii*) Deliberate temporary overloading of the tendons (for a short period of time e.g. 2-10 min.) at the time of initial stressing, in order to reduce future relaxation losses by disposing of the rapid initial relaxation, is thought to be generally beneficial and a particular advantage in the case of strand. However, the reduction is of little consequence in stabilised strand where the long term relaxation loss is not appreciable in any case.

(*viii*) A feature of importance in the field is the effect of the design of strand jacks upon the relaxation behaviour of the prestressed strand. The tendency of strand to "unwind" under load has been discussed by Bannister (1959): it arises from the presence of a torsional component approximating to 10% of the load applied to the tendon. The presence of this component would appear to have a marked effect upon relaxation losses and in tests on 12.7mm strand (Duckfield, 1964), the relaxation at 1 000 hours was found to be of the order of about 5% and 8% with and without torsional restraint, respectively. Hence, for practical purposes, those jacks designed with a key way or other device to prevent rotation during stressing may be preferred.

Creep

Creep is intrinsically more difficult to theorise upon, or measure experimentally in the field. The phenomenon of creep (fluage) in steel is, however, discussed by Fenoux and Portier (1972).

As a result of precise experiments augmented by the findings of other authors, they conclude that

(*a*) The creep rate $\alpha(F)$ increases over the range 0-30% GUTS, is constant to the limit of proportionality (68% GUTS in the case studied), and then increases rapidly at higher loads.

(*b*) The amount of creep can be represented by an equation of the form: creep at time t after lock-off $= \alpha(F) \times \log t$.

Fenoux & Portier point out that creep does not terminate with time, but no indication of a practical time limit for stabilisation or negligible creep is provided.

(*c*) Values of $\alpha(F)$ appear independent of steel type for stresses less than the limit of proportionality.

To illustrate the importance of creep for a test stress near the limit of proportionality, Fenoux & Portier have stated that the creep in 2 minutes is 0.2mm/m of free length.

It is further shown that the relation between creep and relaxation rates, under identical conditions, is of the form

$\beta(F) = E \times \alpha(F)$

where $\beta(F)$ is the rate of relaxation, and E is the elastic modulus of the tendon.

Field observations

To illustrate the importance of the phenomena causing load loss, the authors have assembled some of the better documented case histories. Generally, however, the type and quality of the scanty data published to date relating to long-term behaviour are disappointing. For instance, it is intuitive to suppose that rock type is a major influence on anchor performance, yet little information on relevant rock properties, other than the geological name, is commonly supplied in case histories. For example, Schwarz (1972) monitored the behaviour of many anchors at frequent intervals over seven months in Stuttgart but although he presented comparisons of anchor performance in lithologically distinct horizons, no relevant rock properties were detailed.

It would appear that little guidance is available at Code level. PCI (1974) affirms that for most rock anchor applications, the primary time-dependent loss is steel relaxation—up to 3% in seven days dependent on the type of steel, and the South African Code (1972) recommends locking-off an overload of 10% as "an allowance for relaxation and creep" similar to British practice.

In the following examples, the relevance of such allowances may be readily judged.

Much of the early published data relates to the prestressing of dams and in the particular case of raising existing dams founded on good quality rock, where the structure is "old and worked", Parker (1958) advises that no allowance is necessary for creep and shrinkage in the concrete. Loss of prestress with time, therefore, is only due to tendon relaxation. In this connection, Walther (1959) describes the performance of VSL anchors at the Luzzone Dam. In particular, for a 1 000kN test anchor (fixed anchor length = 3.20m, diameter = 90mm), the loss in prestress over 3 500 hours was 4%—"virtually exactly that which had been anticipated from relaxation losses".

For new dams, Zienkiewicz and Gerstner (1961) have estimated that load loss is primarily due to creep in the concrete of the dam and only secondarily to tendon losses. They computed that an ultimate prestress loss of 9% was possible—compared to an allowance of 10% at the Allt-na-Lairige Dam, where the anchors were installed in fissured granite.

Eberhardt & Veltrop (1965) conclude the the 24 hour load check is much too soon to check "one significant possible source of stress loss; namely shrinkage and creep of the concrete". They estimate ultimate load losses to be of the order

Concrete-creep	2.0%
Concrete shrinkage	3.6%
Steel creep	1.0%

but overload by 10% to cover the worst possible case.

Thompson (1969) describes six test BBRV anchors (fixed anchor length = 9m, diameter = 152mm) as detailed in Table IX, at the John Hollis Bankhead Dam, Alabama.

The relatively high load loss in anchor 6 is ascribed to its shorter length causing the fixed anchor to intersect the lower of two 0.6m thick coal seams in the sandstone—shale sequence. Thompson claims that some crushing in one or both of the coal seams could account for the higher loss.

The longest record of prestress loss available is that from Cheurfas Dam, the salient points of which are summarised in Table X. The fixed anchor zone, consisting of a grouted borehole (250mm dia.) with two under-reams (370mm dia.) was formed in 10m of yellow sandstone, overlain by about 4m of fossiliferous limestone and underlain by marl.

A claim by Khaova *et al* (1969) that the long-term load loss was due principally to corrosion of the tendons has proved unfounded (Portier, 1974).

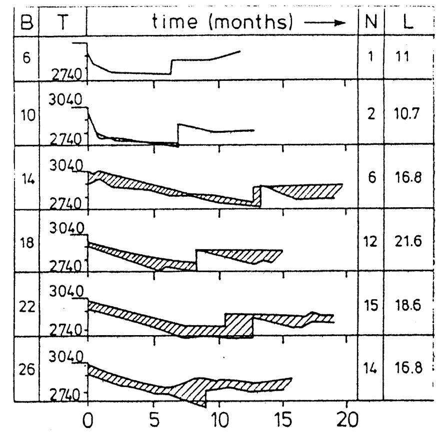
Gosschalk & Taylor (1970) describe various aspects of 2 740kN anchors (fixed anchor length 5-6.5m, diameter = 140mm) installed in quartzite at Muda Dam, Malaysia. The stressing procedure involved stressing to 3 030kN, followed by two complete load—unload cycles. The residual load was measured at seven days, and was found to have dropped by up to 450kN. Restressing to 3 030kN resulted in all loads being above 2 887kN three days later. Subsequently 25% of the anchors were monitored, and were found to have "remained fairly steady" as shown in Fig. 26. Measured settlements of the anchorage blocks at service were considered negligible.

The long-term performance of anchors designed for service in other applications has also been briefly recorded.

Comte (1965) describes 1 250kN BBRV anchors in very variable fissured argillaceous schist in the Nendaz Cavern and recorded losses of 4-8%—notably less than the 10% margin allowed. The greater part of this loss was found to occur in the very early stages of a five year period of observation.

In the course of stressing two test anchors (fixed anchor length 6m, diameter = 99mm), Barron *et al* (1971) subjected one to three loading cycles prior to lock-off, whereas the other was loaded directly to the lock-off load. Both were installed in jointed granite, the elastic modulus of which was 40-50 times less for the mass ($0.15 \pm 0.04 \times 10^{+4}$ N/mm^2) than for the material $6.3 \times 10^{+4}$ N/mm^2).

The load on the first anchor remained stable throughout the observation period, whereas this stable state was only achieved in the second anchor after marked loss in the first week (Fig. 27). This difference in behaviour was ascribed to "time-dependent behaviour of the rock under load, causing closing of fissures etc". They concluded that it is advisable to precycle the load up



B – Buttress No.
T – Tendon load (kN)
N – No. of tendons in buttress and included in envelope.
L – Length of tendon from top to centre of fixed anchor (m)

Fig. 26. Envelopes of tendon load variations (after Gosschalk and Taylor, 1970)

TABLE X. RECORD OF PRESTRESS LOSS FOR CHEURFAS DAM

Years after stressing	Amount of loss (kN)	% Loss
3	408	4
6	449	4.4
9	459	4.5
18	561	5.5

TABLE IX. LOSS OF ANCHOR LOAD WITH TIME FOR SELECTED ANCHORS AT THE JOHN HOLLIS BANKHEAD DAM, ALABAMA (after Thompson, 1969)

Anchor No.	Free Length (m)	Initial load (kN)	Total extension	Residual load (kN)	Time elapsed	Load loss (kN)	% Load loss
1	35	3336	206mm	3336	16 hrs	0	0
2	35	3363	205mm	3278	18 hrs	85	2.5
3	35	3278	206mm	3220	19 hrs	58	1.8
4	35	3336	214mm	3278	31 hrs	58	1.7
5	35	3363	210mm	3336	5 & 10 days	27	0.8
6	29	3363	217mm	3163	5 & 10 days	200	6.0

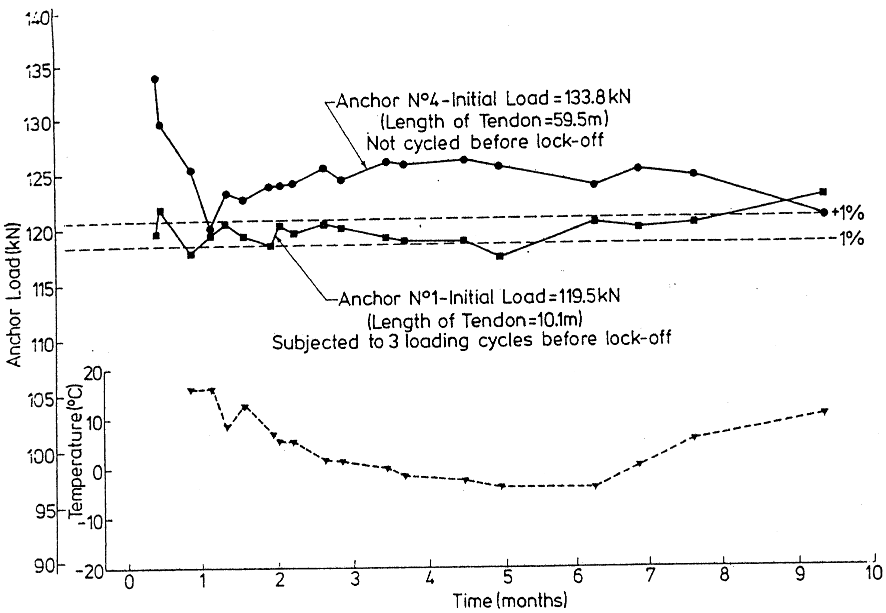
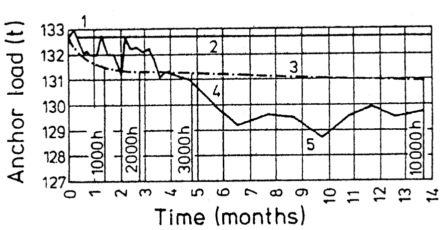


Fig. 27. Comparison of anchor performance with time (after Barron et al, 1971)



1. Initial reading.
2. Designed load. (132.7 t)
3. Theoretical tendon relaxation curve.
4. Actual anchor performance.
5. Lowest load recorded. [Loss of 4t≈3%]

Fig. 28. Performance of one monitored anchor (after Moschler & Matt, 1972)

to its maximum level for several cycles, in order to minimise load loss after lock-off. There would appear to be a temperature effect on the apparent load—but this could be due to the susceptibility of the load cells to temperature variation.

Möschler and Matt (1972) presented data on the performance of a 1 330kN VSL anchor (fixed anchor length 4.50m) after test loading to 1 725kN in fractured calcareous schist in the Waldeck Cavern. This is shown in Fig. 28, in which the theoretical steel relaxation curve is also plotted.

As noted previously, one of the largest scale anchor performance programmes described (McLeod & Hoadley, 1974) involved the placement of load cells under 100 anchors (diameter = 76mm) installed in Silurian mudstone in Melbourne. The maximum working load was about 900kN with most locked-off at 250-300kN, following a test load of 1.4T_w.

Of the results considered satisfactory, the average load loss after 3-6 months was 9%, but 80% of the anchors had an average loss of only 5%. The rather higher apparent losses in the other anchors may have been due to instrument malfunction. On a second site where more care was taken with the load cells, the average loss after one month was only 1%, with no large losses recorded in that time. The authors concluded that in general load loss can be expected, normally 5-10%, but occasionally up to 20%.

One of the most informative case histories has been published by Hutchinson (1970). Six rows of anchors were installed into Upper Chalk on the Isle of Thanet to stabilise a cliff face (Fig. 29a). With a factor of safety on the ultimate chalk-grout bond (0.5N/mm²) of 3.75, the fixed anchor lengths ranged from 5-8m (hole diameter = 102mm) to provide working loads from 167 to 265kN.

The anchors were initially locked-off at 1.25 T_w and checked ten days later when they were restored to the designed initial values. The maximum recorded loss in this time was 14% in one of the upper rows of anchors (in the poorest quality chalk). Load restoration was repeated three times on all anchors, after which all but one in each row were finally grouted up and locked off.

The remaining six anchors were monitored over 1.1 years, and the results after that time are shown in Fig. 29b. A maximum loss of 16% was recorded in the uppermost anchor.

Hutchinson considers his data provide a good correlation between chalk quality and load loss, and it is noteworthy that in the good quality chalk, the interfacial safety factor employed in design was associated with insignificant creep loss.

Remarks

The quality and accuracy of information published on the time-dependent behaviour of steel tendons would appear to be wholly suitable for application to rock anchor systems.

On the other hand, the authors find that too few long-term records of actual field behaviour provide sufficient data about anchor load and geometry, and rock classification. One important consequence is that optimum overload allowances cannot be determined to accommodate long-term losses.

However, it is evident that cyclic preloading may eliminate creep during service, choice of a large interfacial safety factor may inhibit creep, and restressable anchor blocks can be used to compensate for creep.

GENERAL CONCLUSIONS

In the field of rock anchors the quality of workmanship during construction greatly influences subsequent performance of the anchor. In addition, rock anchors are often spaced at close centres, and the normal site investigation programme cannot highlight, on such a small scale, subtle variations in rock quality which will affect the behaviour of individual anchors.

As a consequence, it is strongly recommended that each anchor should be subjected to an initial proof loading stage. Whilst it is fully appreciated that stressing is a skilled operation, and that considerable judgement must be exercised when analysing the results of the operation, only in this way can the safety of each anchor be ensured.

Bearing in mind the rapid growth of ground anchor technology, specialists should be aware of possible conflicts between new design concepts and existing code recommendations. For example, BS 4447 stipulates a 92% efficiency for the head relative to the tendon GUTS, although the minimum load rating factor in current design is related directly to tendon f.p.u. As a result, BS 4447 may well be stipulating a lower rating factor than those actually specified (see Table XV, Part I).

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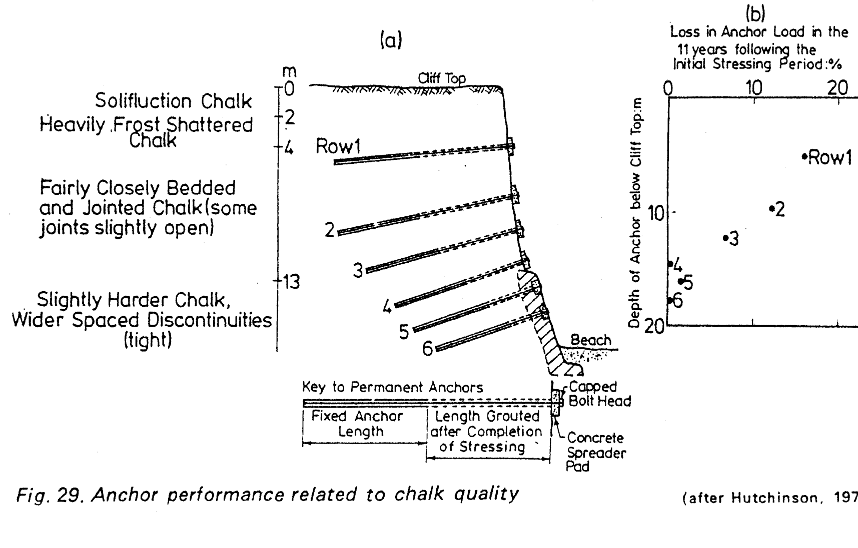

Fig. 29. Anchor performance related to chalk quality (after Hutchinson, 1970)

Mascardi (Italy); P. Matt (Switzerland); L. Hobst, K. Klein (Czechoslovakia); J. I. Adams, D. F. Coates, J. A. Franklin, G. C. Meyerhof, R. J. Van Ryswyk (Canada); J. W. Hilf, R. Miller, R. L. Schmidt, R. E. White, P. T. Wycliffe-Jones (USA); A. J. da Costa Nunes (Brazil); P. I. Mahaffey (Australia); M. Matsuo (Japan); R. Parry-Davies (South Africa); A. D. Barley, J. Harvey, K. W. Longbottom, J. Mitchell, D. Moy, D. G. Price, W. Deppner, and C. Truman-Davis (UK).

References

Arcangeli, E. & Tomiolo, A. (1975): "Modalita' di controllo delle caratteristiche dei tiranti ed elaborazione statistica dei risultati". XII Convegno Nazionale Di Geotecnica, Associazione Geotecnica Italiano—Consenza, 18-21 Sept. 1975, pp 3-13.

Antill, J. M. (1965): "Relaxation characteristics of prestressing tendons", Civ. Eng. Trans. Inst. Eng. Aust., 7 (2) pp. 151-159.

Bannister, J. L. (1959): "Characteristics of strand prestressing tendons" Structural Engineer, 37 (March), pp 79-86.

Barley, A. D. (1974): Private communication.

Barron, K., Coates, D. F. & *Gyenge, M. (1971):* "Artificial support of rock slopes". Dept. of Energy, Mines and Resources, Mines Branch, Ottawa, Research Report R 228 (Revised), 145 pp.

Baus, R. & *Brenneisen, A. (1968):* "The fatigue strength of prestressing steel". Proc. FIP Symp. on Steel for Prestressing, Madrid (6-7 June) pp. 95-103.

British Standards Institution: 2 Park Street, London, W.1.

- BS 18 (1962) Methods for tensile testing of metals.
- BS 3617 (1963) Stress relieved seven wire steel strand for prestressed concrete.
- BS 2691 (1969) Steel wire for prestressed concrete.
- BS 4545 (1970) Methods for mechanical testing of steel wire.
- BS 4447 (1973) The performance of prestressing anchorages for post tensioned construction.
- CP 115 (1969) The structural use of prestressed concrete in buildings. (Part 2, metric units).
- CP 110 (1972) The structural use of concrete (part 1).

Bureau Securitas (1972): "Recommendations regarding the design, calculation, installation and inspection of ground anchors". Editions Eyrolles 61 Boulevard Saint-Germain, Paris-Ve (Ref. TA 72).

Buro, M. (1972): "Rock anchoring at Libby Dam". Western Construction (March), pp, 42, 48 & 66.

Chen, S. C. & *McMullan, J. G. (1974):* "Similkameen Pipeline Suspension Bridge" ASCE, Transportation Eng. Jour., 100 T.E.I. (Feb.) pp. 207-219.

Comte C. (1965): "L'utilisation des ancrages en Rocher et en Terrain Meuble", Bull. Tech. de la Suisse Romande, 22 (Oct), pp. 325-338.

Da Costa Nunes, A. J. (1966): "Slope stabilisation—improvements in the techniques of prestressed anchorages in rocks and soils". Proc. 1st Cong. Int. Soc. Rock Mech., Lisbon, 2, pp. 141-146.

Deutsche Industrie Norm (1972): "Soil and rock anchors; bonded anchors for temporary uses in loose stone; dimensioning, structural design and testing". DIN 4125, Sheet 1, June 1972.

Deutsche Industrie Norm (1974): "Soil and rock anchors; bonded anchors for permanent anchorages in loose stone; dimensioning, structural design and testing". DIN 4125, Sheet 2, 1974 (Draft).

Duckfield, B. J. (1964): Private communications. United Power Co Ltd., England (see Antill, J. M. (1965, op. cit.)

Eastwood, W. (1957): "Fatigue tests on prestressed concrete beams". Civ. Eng. Publ. Wks. Review, 52, (July), pp. 786-787.

Eberhardt, A. & Veltrop, J. A. (1965): "Prestressed anchorage for Large Trainter Gate". Proc. ASCE, J. Struct. Div. 90(ST6) pp. 123-148.

Edwards, A. D. & *Picard A. (1972):* "Fatique characteristics of prestressing strand". Proc. ICE, 53, Part 2, (7534), pp. 323-336.

Fenoux, G. Y. & *Portier, J. L. (1972):* "La mise en Precontrainte des Tirants". Travaux, 54 (449-450), pp. 33-43

FIP (1973): "Final draft of recommendations". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1974): "Recommendations for approval, supply and acceptance of steels for prestressing tendons." Cement and Concrete Assoc., 52 Grosvenor Gardens, London. (Ref. 15.321).

Geffriaud, J. P. & *Rouget, M. (1972):* "Contrôle de la tension des tirants précontraints. Tirants définitifs de la paroi moulée du Musée Archaeologique de Fourvière à Lyon. Travaux, 54, pp. 44-54.

Gosschalk, E. M. & Taylor, R. W. (1970): "Strengthening of Muda Dam foundations using cable anchors". Proc. 2nd Cong. Int. Soc. Rock Mech., Belgrade, 3, pp. 205-210.

Hanna, T. H. (1969): "Tunnel approaches, subways and underpasses excavation support by anchorage systems". Tunnels & Tunnelling, 1 (1), pp. 27-33.

Hanna, T. H. (1974): Foundation instrumentation. 1st Ed., Trans. Tech. Publications, 372 pp.

Hennequin, M. & *Camberort, H. (1966):* "Consolidation du remblai de Malherbe". Revue Generale des Chemins de Fer (Feb.).

Hutchinson, J. N. (1970): Contribution to Discussion on Soil Anchors. In: Proc. Conf. Ground Engineering, Institution of Civil Engineers, London (16 June), pp. 85-86.

Khavoa, M., Montel, B., Civard, A. & Lauga, R. (1969): "Cheurfas Dam Anchorage; 30 years of controls and recent reinforcement". Proc. 7th Int. Conf. Soil Mech. and Found. Eng., Paper 15-12.

Littlejohn, G. S. (1970): Discussion on Paper "Soil anchors". In proc. Conf. Ground Engineering. Inst. of Civ. Engrs., London, June 16, pp. 33-44, discussion pp. 115-120.

Littlejohn, G. S. & Macfarlane, I. M.; "A case history study of multi-tied diaphragm walls". ICE conf. on Diaphragm Walls & Anchors, London, Sept., pp 113-121.

Longbottom K. W. & *Mallett, G. P. (1973):* "Prestressing steels". The Structural Engineer, 51 (12), pp. 455-471.

Longbottom, K. W. (1974): Discussion on paper "Prestressing steels" by Longbottom & Mallett (op. cit). The Structural Engineer, 52, (9), pp. 357-362.

MacLeod, J. & Hoadley P. J. (1974): "Experience with the use of ground anchors". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 83-85, 7th FIP Congress, New York.

Mihajlov, K. V. (1968): "Stress relaxation of high tensile steel". Proc. FIP Symp. on Steel for Prestressing, Madrid, (6-7 June) pp. 57-78.

Mitchell, J. M. (1974): "Some experiences with ground anchors in London". ICE Conference on Diaphragm Walls and Anchors, London (Sept.) Paper No. 17.

Morris S. S. & *Garrett, W. S. (1956):* "The raising and strengthening of the Steenbras Dam", (and discussion). Proc. ICE Pt. 1, Vol. 5, No. 1 pp. 23-55.

Möschler, E. & Matt, P. (1972): "Felsanker und Kraftnessanlage in der Kaverne Waldeck II". Schweizerische Bauzeitung, 90 (31), pp. 737-740.

Nicholson Anchorage Co Ltd. (1973): "Rock anchor load tests; for sheet pile bulkhead for the General Reinsurance Corp. Project, Steamboat Road, Greenwich, Conn., USA. Unpublished Report (8 pp.).

Ostermayer, H. (1974): "Construction, carrying behaviour and creep characteristics of ground anchors". ICE Conference on Diaphragm Walls and Anchorages, London (Sept.) pp. 141-151.

Parry-Davies D. (1968): "The use of rock anchors in deep basements". Ground Engineering Ltd., Johannesburgh, South Africa, unpublished paper.

PCI Post-Tensioning Committee (1974): "Tentative recommendations for prestressed rock and soil anchors". PCI Chicago, USA, 33 pp.

Portier, J. L. (1974): "Protection of tiebacks against corrosion". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 39-53 7th FIP Congress, New York.

Saxena, S. K. (1974): "Measured performance of a rigid concrete wall at the World Trade Centre". ICE Conference on Diaphragm Walls and Anchors, London (Sept.), Paper No. 14.

Schwarz, H. (1973). "Permanent Verankerung Einer 30m Hohen Stutzwand in Stuttgarter Tonmergel durch Korrosions—Geschützte Injektionsanker System Duplex". Die Bautechnic, 9 pp. 305-312.

Short, A. (1975): "DoE studies creep and stress behaviour at Navy Complex". Construction News, Sept. 4, pp. 28-29.

South African Code of Practice (1972): "Lateral support in surface excavations". The South African Institution of Civil Engineers, Johannesburg.

Standards Association of Australia (1973): "Prestressed concrete Code CA35—1973, Section 5—Ground anchorages" pp. 50-53.

Stefanko, R. & de la Cruz (1964): "Mechanism of load loss in roof bolts". Proc. 6 Symp. on Rock Mechanics, Missouri, (Oct.), pp. 293-309.

Thompson, F. (1970): "The strengthening of John Hollis Bankhead Dam". Civil Engineering (NY), 39 (12) pp. 75-78.

Walther, R. (1959): "Vorgespannte Felsanker". Schweizerische Bauzeitung, 77 (47) pp. 773-7.

Zienkiewicz, O. C. & Gerstner, R. W. (1961): "Stress analysis of prestressed dams". Proc. ASCE Journal of the Power Division, 87 (POL), Pt. 1, pp. 7-43.