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*

PRACTICAL ASPECTS OF ROCK ANCHORAGES FOR DAMS

by

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"The work of designing, fabricating, installing, grouting, stressing and monitoring ground anchors is of a highly specialist nature in which standards and methods are improving world wide at a rapid rate. Technical specifications and directions cannot replace professional experience and conscientiousness of a contractor's staff at all levels.

A valuable role of a specialist subcontractor, as compared with a main contractor, is as a specialist adviser to the main overall project designer during the pre-tender design process. Such specialist advice is rarely available from a main contractor. In general, it is my opinion that ground anchoring is best carried out by a specialist subcontractor rather than by a main contractor installing anchors made from material supplied by a post-tensioning firm." J. C. Rutledge, Chief Geotechnical Control Office, Hong Kong, and Member of Ground Anchor Working Committee, FIP (1982).

1. PREAMBLE

The use of prestressed rock anchorages in dam engineering is as old as the technique itself: the first recorded use of anchorages was to stabilize Cheurfas Dam, Algeria in 1934. Since then, anchoring has gained worldwide recognition and application not only in connection with dams but for a multitude of other purposes (Littlejohn, 1982).

The classic applications in dams are to provide resistance to overturning (Figure 1) and restraint to sliding (Figure 2). However, there are countless examples of anchorages being used as tie downs for spillway construction (Figure 3) and for straight forward rock slope stability around tunnel portals or abutment excavations. Again, another famous example was in the Service Spillway Plunge Pool of Tarbela Dam, Pakistan (Figure 4) where almost 2000 anchorages of 430 ton ultimate tendon load were installed to provide, in effect, a zone of "compressed rock" to further resist the tremendous dynamic forces exerted during operation of the Spillway/Flipbucket structure. A newer concept was described by Nuss (1988) in the remedial works foreseen for Stewart Mountain Dam, AZ: not only will the anchorages provide against a general overturning failure of the concrete structure as a unit, the will also act to compress contiguous blocks (separated by horizontal construction joints) to prevent their relative movements within the structure in the event of high seismic activity.

As noted below, there is a rich literature available to the engineer who wishe to design anchorage systems. Equally there is generally a high degree of experience and expertise to exploit from within the ranks of the specialist contractors, and drilling and post-tensioning companies.

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In line with the growth in the dam remedial market, the expansion in the use of prestressed rock anchorages continues. As it does, it touches a progressively wider field of dam engineers who perhaps have neither the time nor the inclination to read comprehensive treatises, and will not have the specialist background in a technique which involves intricacies of drilling, grouting and post-tensioning.

This paper has, therefore, been prepared as a practical guide to some major aspects of rock anchor construction, about which there are frequently sad misconceptions. The paper will be of limited use to the engineer seeking only to learn how to design anchorage systems: he is referred to the publications cited in the following section. In addition, the paper demands some basic knowledge of the theory and practice of anchor technology. The paper is not intended to be a "first principles" state of the art, it is intended to be a short guide aimed at helping to reduce unnecessary conflict and debate between Owner and Contractor. Reference is made to the best of the international standards to lend perspective to our national approach.

2. SOURCES OF DATA

The stabilization of dams by anchorages is always a process involving a high engineering content, and frequently provides unique or exceptional facets such as these arising from difficult access restrictions, extreme construction tolerances or tendons of great weight or length. For example, Thurnherr (1982) described anchors of 1400 ton ultimate tendon capacity up to 380' long installed from the 12' wide crest of Lalla Takerkoust Dam, Morocco.

For these reasons, good case histories abound in the engineering press, (e.g. Standig, 1984, and Henn & Yow, 1985) usually authored by the design engineer or specialist contractor involved. Frequently, however, such papers tend to have a strongly commercial motive, and although they contribute to the general pool of knowledge, often do not approach the subject in a generic manner. Global literature survey reviews such as by Littlejohn and Bruce (1977), Hanna (1982) and Hobst and Zajic (1983) have proved very useful to the anchor community in that they provide a perspective on all aspects of the technology from design and construction, through corrosion protection to stressing, testing and long-term performance.

With respect to national construction standards, there is, for example, currently no ASTM Code for rock anchorages, earlier attempts having been disrupted by conflicting proprietary viewpoints. However, the PTI (1986) Recommendations do constitute an acceptable and well regarded substitute, having been prepared by a representative cross section of anchor specialists.

Probably the most comprehensive and up-to-date work on the world scene is the new British Code of Practice BS 8081 (1988) which supersedes the Draft for Development DD81 (1982), for six years tried, tested, and modified by the industry. These documents compliment the excellent FIP+ documents on Design and Construction (1982) and Corrosion and Corrosion Protection (1986).

+FIP are the familiar initials of the Fédération Internationale de la Précontrainte.

These works are essential reading for ground anchor engineers.

In the following section, reference will be made to these publications in support of comments made.

3. REQUIREMENTS FROM SITE INVESTIGATION PROGRAMS

The site investigation is obviously the starting point in the serious evaluation of a rock anchorage project. Frequently, however, there is uncertainty about what parameters are most relevant, or should have priority, in an investigation, leading to omissions or irrelevances in the final report. As a guide, Table 1 is a matrix linking the various geotechnical parameters with the different aspects of anchoring. An assessment of this type should be made on a site specific basis by the planner. The following points are provided for clarification and explanation.

Overall Stability: conventionally the overall stability (or ultimate uplift capacity) is calculated on the assumption that a single anchorage engages a cone of rock, of a certain geometry and location (Figure 5). The uplift capacity is then estimated as the weight of rock in that cone (based on submerged weight, where appropriate). Investigations by Littlejohn and Bruce (1977) and Littlejohn, et al., (1977) show clearly that this is normally an extremely conservative approach as it does not take into consideration the contribution of "rock strength" acting over the surface of the theoretical failure volume. An appreciation of the rock mass structure will give a more accurate indication of a likely "failure volume," and also of the main rock strength parameters likely to contribute most strongly within and around it.

Analyses of this type will therefore permit anchorage embedment lengths to be minimized rationally, and so reduce overall costs.

Rock Grout Bond: in the absence of direct test data on ultimate bond values or shear strength parameters, one reliable rule of thumb, dating from Littlejohn (1972) is to estimate the ultimate bond to be 10% of the U.C.S. (unconfined compressive strength) of massive rock (to a maximum value of U.C.S. of 6000 psi). This assumes also that the grout has at least 6000 psi strength upon testing. The ultimate value is then factored to give an estimated safe working bond value. The safety factor should reflect the degree of weathering and the nature of the rock structure, as well as the quality of the investigation data.

Significance of Knowing E Value of Rock Mass: theoretical analyses (Coates and Yu, 1970) and field data (Berardi, 1967) confirm that the distribution of bond stress in and along the fixed anchor length is dependent on the ratio of the elastic moduli of the anchorage material (Ea) and the rock mass (Er). The smaller this ratio (i.e. the higher the rock modulus) the greater is the bond stress concentration at the top of the fixed anchorage (Figure 6). Only in very soft rocks is it reasonable to assume, therefore, that the bond is evenly distributed and that the design may be based accurately and directly on the shear strength of the weaker medium. This also affects the assessment of anchorage stressing information, especially during Performance Testing (see below) where careful analysis will indicate the amount of apparent debonding in the fixed anchor length.

Corrosion Protection: although this is dealt with below it is worth reiterating that this must be regarded as an integral and vital part of anchorage design. The degree of corrosion protection to be provided must be based not only on the chemical and dynamic properties of the groundwater, but also on the permeability of the rock mass - before and after any phase of pregrouting and redrilling.

Construction: throughout every phase, it is clear that the more detailed the site investigation data provided, the less uncertainty there is about the suitability of the techniques foreseen. This in turn will lead to optimized technical and program performance and reduced costs.

4. CONSTRUCTION

4.1. Drilling

There is no reasonable argument against the normal use of down-the-hole hammers for drilling unreinforced concrete and competent rock. It is a rotary percussive method (e.g. Deere 1980) routinely employing delivered compressed air pressures of up to 300 psi (actual air pressures acting on the borehole wall are, of course, much less). It provides holes of exceptional straightness as the percussion is applied at the bottom of the hole, just above the bit: the drill string is thus pulled as opposed to pushed. Typical tests show deviations of less than 1 in 100 to be routinely attainable when proper drilling practices are observed. For the same reason, rates of penetration are more or less constant with depth, as no percussive energy is lost between drill head and drill bit. Penetration rates are high (up to 60 ft./hr. is not exceptional). Mechanical practicalities restrict conventional hole sizes to 4-12" in diameter - a range which encompasses by for the larger portion of anchor requirements. Depths in excess of 300' can be reached. The down-the-hole hammer method, not relying on very high torques or down pressures for its efficiency, can be operated from relatively small drilling rigs provided they provide a stable, rigid frame. This is a considerable advantage for most operations on existing dams. In addition, the fact that the percussion is activated in the hole renders the method relatively quiet, and provided the rock debris which is blown out of the hole during drilling are properly dealt with, the whole system is very sympathetic to the environment. There have been no published instances of the method causing damage to the existing concrete structure, even when holes have been drilled as close as 2' to free faces e.g. at Pickwick Lock and Dam, Tennessee. In short, diamond drilling may be necessary for only the reinforced portion if substantial steel reinforcement of the concrete is foreseen, and rotary (tricone) drilling in the softer sedimentary rocks may be apposite, but overall, down-the-hole drilling is the most logical and economic choice in most instances. Examples of projects conducted by the author's company alone are summarized in Table 2.

Regarding borehole deviation, specific project requirements should always be addressed. A good drilling contractor should routinely provide holes with an overall deviation not greater than 1 in 100, although there are many cases (Littlejohn and Bruce 1977) where 1 in 50 or less has been wholly acceptable. By the careful use of certain drilling accessories e.g. drill string centralizers, and using the best drilling practice, deviations of up to 1 in

200 can be provided, if not necessarily always guaranteed. To specify tolerances finer than this is practically unreasonable, and in any event may not be possible to verify given the in hole instrumentation currently available to measure them. As a general guide, PTI (1986) recommends that "drill holes normally can be started within an angle tolerance of 1 to 3 degrees from their planned orientation. A deviation of 1-2" in 10' can be maintained with normal drilling methods." Bearing in mind the length and proximity of most anchors on dams, these recommendations should be regarded as a minimum standard. It is interesting that FIP (1982) and BS 8081 (1988) both allow deviations of up to 1 in 30 for routine anchorage installations.

As a final word on drilling, it is worth reiterating that it remains good practice to over drill every hole by 2-3', thus providing a sump which will accommodate any drilling and other debris and so permit the subsequent introduction of the tendon to the full, designed depth.

4.2. Permeability Testing

Excluding the case of exceptionally porous sedimentary deposits, the permeability of rock masses is a reflection of the geometry and characteristics of the discontinuities principally their size and frequency. Therefore, it is fundamentally incorrect to express rock mass permeabilities in terms of soil units viz. cm/sec. When water testing rock anchor boreholes, we are more interested in what can <u>escape</u> from the hole, as opposed to what can the rock mass <u>accept</u> (which is the grouter's viewpoint).

A fluid cement grout is particulate - what defines its penetrability (and so potential escape from an anchor borehole) is the number and aperture of the fissures in the surrounding rock mass that hole intersects. It has intrinsically nothing to do with the diameter of the hole. Therefore, permeability test analyses - which dictate whether pregrouting is necessary, and also impacts the choice of tendon corrosion protection - should be conducted with understanding and care.

Recent experimental studies suggest that a fissure tighter than 160μ* will not accept Type I cement grout particles. A fissure of this width will permit a flow of about 0.4 gal/min at an excess head of 1 atmosphere (14 psi). Therefore, if the total water loss from a rock stage is less than this value, the possibility of appreciable anchor grout loss may be discounted, and no phase of pregrouting and redrilling will be required. It is noteworthy that with the smaller cement particles common in finely ground cements the limiting fissure width reduces to 100μ through which water flow would be 0.15 gal./min./ atmosphere.

It is clear that this loss of grout potential should not be related to borehole length: in a worst case, a single fissure over 160μ wide may exist in a 10' borehole or in a 100' borehole. Therefore, it is strongly suggested that the limiting value for determining the need for pregrouting should be specified simply in terms of gal./min./atmosphere without reference to hole diameter or length. This contradicts the current PTI (1986) Recommendations, but is in line within FIP (1982) and BSI (1988), which sets a realistic criterion of about 1 gal./min. at an excess pressure of 1 atmosphere. The position of the local

*1μ = 1 thousandth of 1mm.

water table must of course be measured in order to calculate the excess pump pressure. In addition, FIP (1982) confirms that any hole showing artesian water gain should be pregrouted irrespective of the magnitude of inflow.

4.3. Tendon Handling and Installation

Good practice is summarized by Littlejohn and Bruce (1977) and by the recent codes and recommendations. All confirm that the presence of rust on strands per se, is no detriment to bond development with grout, quite the contrary. Only if the rust is loose or flaky should remediation be demanded.

There is a trend towards longer and heavier anchor tendons as projects become more ambitious. For example, on a current contract at Shepaug Dam, CT, tendons up to 220' long and weighing over 4 tons are being installed. To avoid damage to the tendons and their corrosion protection - as well as for the sakes of safety and practicality - it is essential to specify (1) the necessity for some form of mechanical installation device, such as a hydraulically braked drum: this is much more controllable and economic than a helicopter or a crane, for example, and (2) proper, strong and frequent spacer/centralizer units at regular intervals along both free and fixed lengths. As a final word, research by Bruce (1976) has shown that within the fixed anchorage length, not more than 15% of the borehole volume should be occupied by steel (as opposed to grout). This limit should ensure that there will be sufficient interstitial space to permit the grout to penetrate uniformly between the strands thus allowing proper load transfer and efficient corrosion protection.

4.4. Grouting

Poor or inefficient grouting techniques represent one of the most common causes of anchorage failure when construction - as opposed to design or corrosion protection - is to blame.

High speed grout mixers must always be used (Gourlay and Carson, 1982) to ensure uniform and intimate mixing of the cement particles and the water. This high wetting efficiency permits lower water cement ratios (say w = 0.4 by weight) to be used, leading directly, therefore, to higher and earlier strengths, and greatly reduced bleed (Figure 7). This latter factor is essential to monitor, as bleed water in long ducts such as in the case of anchorages cannot necessary migrate to the top of the grout column. It typically forms long "ribbons" or lenses which may be in contact with the strands. Upon setting of the grout, therefore, the absence of uniform grout steel contact will reduce the efficiency of bond development and compromise the security of the corrosion protection. BS 8081 (1988) states that for relatively impermeable ground conditions, anchor borehole grout bleed should not exceed 2% after 3 hours, or 3% ultimately (at 68^{o}C). Where a w/c ratio lower than 0.40 is truly required, for example to reach high early strength, then the use of a plasticising additive will be necessary. Generally this is not common in America.

The measurement of bleed capacity is a simple, fast and cheap test which can be used as a routine quality control of the fluid grout.

Another simple monitor of grout composition during construction is the Baroid Mud balance for measuring specific gravity (and hence the w/c ratio - Table 3)

Most rock anchors do not require pressure grouting to develop adequate rock-grout bond resistances, and so grout pumps need not have very high pressure capacities. Nevertheless, pumping distances in dam anchor work are frequently long, and holes may be very deep. High line pressures may therefore be developed especially during the final stages of tremie grouting if the injection line has not been extracted during grouting. Pumps - either progressive cavity or piston - must therefore provide a certain guaranteed minimum rating - and 150 psi may be regarded as that minimum.

Regarding grouting in cold weather, BS 8081 (1988) states that special precautions be taken to prevent the temperature of grout falling to $32^{o}F$ during the early stages of hardening. Grouting in air temperatures below $36^{o}F$ should only be undertaken if the following precautions are taken:

a) grout temperature during injection is at least $41^{o}F$,
b) mix constituents are free from snow, ice or frost,
c) tendon and any surface which will be in contact with the fresh grout must be free of snow ice or frost, and preferably should be at a temperature which will not chill the grout.

Cement grout type should be chosen to reflect the aggressiveness of the ground FIP (1982) lists the following criteria indicative of water aggressive to concrete or hardened cement paste;

pH values < 6.5 (later - 1986 - revised to 5.5)
hardness < $3^{o}d$
CO_2 content > 15 mg/1
NH_{4+} content > 15 mg/1
Mg_{2+} content > 100 mg/1
$SO4^{2-}$ content >200 mg/1.

Note should also be taken of the presence of stray electric currents and recorded damage to buried steel. The soil resistivity (>2000 ohm-cm) is also a good guide.

In general, grouting should be conducted as soon as practical after tendon installation to prevent possible deterioration of the borehole wall, and to arrest the development of steel corrosion.

Corrosion protection of the fixed anchor length by applying an outer corrugated plastic sheath is becoming increasingly more common. Except in the case of very short monobar tendons, it is impractical to specify pregrouting of the tendon inside the corrugation, and letting it set, prior to later installation borehole. Such a system vastly complicates handling and insertion - especially on dam environments - and in any case there is a high probability that the grouted encapsulation will be internally cracked and damaged during installation.

It is much more practical, and safer, to grout the inside and the outside of the protected tendon after installation, even if this may dictate the use of two grouting tubes.

Assuming the free length is greased and sheathed, anchor hole grouting is most easily accomplished in one operation. Of course, where the newer "Stewart Mountain" type approach is to be used, then a Primary/Secondary sequence is necessary - the latter being done only after stressing has been completed. In general, however, two stage grouting (i) does create a "construction joint" (above the Primary grout) which may be a weak point in a corrosion protection system, and (ii) does introduce another separate operation into the overall construction program.

Attention is drawn to the recently published research of Houlsby (1988). He confirmed the need for selected plasticisers for Type I grouts with w/c ratios below 0.4 whilst demonstrating that gas producing expanding additives (e.g. aluminum powder) should be rejected as large, agglomerated gas bubbles tend to form in the hole. In addition, he noted that continued injection through a single fixed tremie in long anchors (say 300' in length) may significantly weaken the fixed anchor grout in its setting process by disturbing juvenile crystal growth and chemical bonding. This problem has been resolved in recent projects by grouting successively through pipes terminating at different levels along the tendon.

Just as gas producing additives should be prohibited, so gelling or thixotropic agents should also be strictly avoided. Their supposed role can be duplicated equally by conventional neat cement grouting practices, whereas their presence compromises severely effective bond development in the fixed anchorage zone.

4.5. Stressing and Testing

The PTI Recommendations (1986) form a good basis for conducting and analyzing stressing programs. Guidance is provided on both short term testing and long term performance. The following points are made, with special reference to dam anchoring

Setting AL. Tendons are usually long and may comprise many tens of strands. Stressing is therefore conducted with a multijack. However, individual strand breakages, and anomalous extensions have frequently been recorded at stress levels from 70-80% of the overall tendon guaranteed ultimate tensile stress. In the great majority of cases, this has resulted from unequal setting of AL, the Alignment Load, typically 2-10% of the design load. This can be done accurately and uniformly if the time and care are taken to apply AL to individual strands with a monojack, prior to routine stressing with the multijack. As an example, tendons over 200' long and comprising 54 strands 0.6" in diameter are being installed by National Foundation Company at Shepaug Dam, CT. A monojack is used to apply AL, and the apparent extensions of individual strands recorded to reach this load has varied by about 6". This merely reflects the fact that different strands lie in different paths in the hole and need, therefore, to be straightened out by different amounts. Multijack stressing has been conducted thereafter to 80% theoretical tendon GUTS, with no records of individual strand rupture.

Phenomenon of Wedge Pull-In. During stressing, the gripping wedges, holding each individual strand in the head plate are progressively "sucked in" as the load is increased. For smooth profile 0.6" diameter (Dyform) strand (GUTS 33 tons), Bruce (1976) measured the amount of pull-in shown in Figure 8. The magnitudes shown are in close comparison with data recorded for normal 0.6" strand at Shepaug Dam, CT. As a preliminary step, therefore, in analyzing stressing data - especially for the important Performance Tests - this amount of relative movement must be subtracted from gross anchor head (or jack ram) extension. In addition, any change of elevation of the stressing head during stressing - i.e. through bedding in, or by structural movement, must also be measured and subtracted from gross extensions prior to detailed analysis.

Performance Tests. Such tests (also known as "on-site suitability tests" FIP 1982) are conducted on the first group of anchorages installed. As defined by PTI (1986) they determine whether the anchorage has sufficient load carrying capacity, that the free length has actually been established, and the magnitude of the residual (or permanent) anchorage movement.

Typically testing is conducted to 1.33, or better, 1.5 times Design Working Load, whilst ensuring the tendon stress at Test Load does not closely approach 80% GUTS. The cyclic nature of the stressing allows the elastic and permanent components of total extension to be separated out for each successive cycle maximum (Figure 9). Examination of the former permits conclusions to be drawn about the efficiency of load transfer down the free length, and the apparent extent of effective debonding in the fixed anchorage. Reflecting the principles of world practice, PTI (1986) recommends acceptance if the elastic movement at Test Load exceeds that movement corresponding to 80% of the designed free length (say, Line A) but is less than that movement corresponding to 100% design free length plus 50% of designed fixed length (say, Line B). These criteria may be regarded as being generous in dam anchoring, especially if the data corrections listed above are applied in advance. For example, an anchor performing near the 80% line implies that load is being dissipated above the fixed anchor length. On long tendons, this could possibly be within the dam, affecting therefore overall stability concepts. Equally, a tendon apparently debonding 24.9' in a 50' fixed anchor zone would theoretically be acceptable although it is well known that in hard rock conditions, (Littlejohn and Bruce 1977) little stress is transmitted beyond the upper few feet of bond zone.

BS 8081 (1988) sets the boundary lines as being 90% of free length for Line A, and either free plus 50% fixed or 110% free, for Line B. Where the inferred free tendon length falls outside these boundary Lines A and B, a further two cycles should be carried out to Test Load to gauge reproducibility of the load-extension data. If the anchorage behaves consistently in an elastic manner, it need not therefore be condemned.

PTI (1986) provides no guidance as to limits of acceptability for residual movements, presumably leaving the choice to engineering judgement, backed by reference to the elastic and creep performances. FIP (1982) confirm the matter should be "agreed by consultant and contractor jointly," as a basis for analyzing the subsequent Proof Tests. Regarding creep performance, PTI (1986) does outline the procedure and the analysis but notes that "in all but the most decomposed rock formations (it) seldom yields any useful information."

Proof Tests. This is a fast economical test, (known by FIP as "Routine Acceptance Test") which used in conjunction with Performance Test data, verifies the acceptability of the installation. It does not feature cyclic loading, and so examination of performance in terms of elastic and residual contributions is not directly possible. However, if the Performance Tests can provide a reliable and consistent data base of the scale and range of residual movements to be expected at different tendon stress levels, then, with care and understanding, a more detailed analysis of the Proof Test data can be undertaken for investigatory reasons, in appropriate conditions.

Long Term Performance. Assuming that there is no structural movement and that the corrosion protection is adequate to prevent attack to the tendon, the significant sources of long term load loss after lock off for anchors in existing dams* are creep in the rock or grout or relaxation of the strand. The former is not considered a major potential problem in anchorages in most rock conditions, and may be foreseen from the creep testing. Conversely, all steel tendons will relax, the amount and rate of which reflecting many factors including the type of the strand, (Table 4) the temperature, its lock off stress level, and its loading history (Littlejohn and Bruce 1977). However, these amounts are quantifiable and predictable, and long term performance should be judged extraordinary only if the characteristics contract with these tendon performance parameters. Typically 5-10% of anchorages are equipped for long term monitoring in major dams worldwide.

5. CORROSION PROTECTION

Anchorages for dams can nearly always be regarded as permanent. By all international standards, such anchors must be properly protected against corrosion. As FIP (1982) states, "the protection system against corrosion should be designed in such a way that the presence of aggressiveness is always assumed." Whereas uncracked cement grout generally provides an effective protection, cracks are almost inevitable, especially in the fixed anchor length. For example, Graber (1981) reported on the exhumation of a test anchor at the Tarbela project: radial and longitudinal fissures up to 0.1" wide were found as far as halfway down the fixed anchor length. In addition, there was crushed grout at the top of that length.

Plastic sheathing (polyethylene or polypropylene) has proven effectiveness. Special attention must be paid to the "transition points" - for example, between stressing head and free length, between free and fixed anchor lengths, and at the distal end of the fixed anchor (Figure 10). Of these, the protection of the stressing head and just below it is absolutely critical, bearing in mind its exposure to atmospheric conditions, construction damage and so on. This problem has been addressed competently in recent years by reputable tendon supply companies, but the proper installation still requires great care and attention to detail during construction (Figure 11).

The definitive work currently available is the FIP State of the Art Report "Corrosion and Corrosion Protection of Prestressed Ground Anchorages" (1986). It reviews and summarizes mechanisms and types of corrosion, ground aggressivity, case histories of failures and various types of systems and intensities of protection - which are "the responsibility of the designer." The question of protection specification philosophy is concisely expressed: "the problem is to balance the safety of people and property in the event of anchorage failure against the cost of providing protection. Since unprotected

*Eberhardt and Veltrop (1965) concluded that for new dams concrete shrinkage and creep were significant sources of possible load loss, contributing up to a 6% reduction in lock off load.

ground anchorages of steel will probably corrode in time, it is also necessary to decide whether the rate of corrosion merits the expense of protection. Corrosion rates vary enormously according to anchorage environment and working mode. Further, there is no certain way of identifying corrosive circumstances with sufficient precision to predict corrosion rates. Consequently, as a general rule, permanent anchorages should be protected, but the design solution may range from double protection, implying two physical barriers to corrosion, in aggressive permeable soils, to simple grout cover in the specific case of low capacity rock bolts used solely as secondary reinforcement" (Author's emphasis). It will be noted that anchorages for dams most definitely do not fall into that last category.

6. CONCLUDING REMARKS

As an indication of the likely expansion in the dam remedial market, Greenhut (1988) reported that in 1980 only 5% of dams were over 50 years old. By 2000, that figure will be over 33%. He noted that the bulk of unsafe dams fall under private, state or local control. The Corps of Engineers inventory identified 64,000 nonfederal dams, about 95% of the nation's total. Of these, 10,000 are considered "high hazard," with almost 30% classified as "unsafe."

Where the lack of safety in concrete or masonry structures is related to overturning, sliding, or certain other forms of structural distress, then anchoring is a tried, proven and reliable technique. Anchorages are also used in new dam construction, but clearly this market is more limited today. It is hoped that this paper will prove of value to all engineers involved in dam anchoring works – even if only to focus attention on major points of contemporary uncertainty or controversy.

REFERENCES

Berardi, G. (1967) Sul Comportamento Degli Ancoraggi Immersi in Terreni Diversi. Univ. of Genoa, Inst. Constr. Science, Series 111, No. 60, 18 pp.

British Standards Institution (1982) Recommendations for Ground Anchorages. Draft for Development DD81, BSI, London, 123 pp.

British Standards Institution (1988) Draft BS 8081: Ground Anchorages. Prepared for Issue in 1988 by Technical Committee CSB/22. BSI, London

Bruce, D. A. (1976) The Design and Performance of Prestressed Rock Anchors with Particular Reference to Load Transfer Mechanisms. PhD Thesis, Dept. of Engineering, Aberdeen Univ., Scotland. 2 Volumes. 670 pp.

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, pp. 175-182.

Deere, D. U. (1980) Some Recent Experiences in curtain grouting for high dams. Joint Meeting ASCE and AEG, Denver Section, September 8, 29 pp.

DIN (1972) Verpressanker für vorübergehende Zwecke in Lockergestein, Bemessung, Ausführung und Prufung. Vol I June.

Eberhardt, A. and Veltrop, J. A. (1965) Prestressed anchorage for large trainter gate. ASCE Jour. Struct. Div. 90 (6), June, pp. 123-148.

FIP (1982) Recommendations for the Design and Construction of Prestressed Concrete Ground Anchors. FIP, Wexham Springs, Slough, England, 31 pp.

FIP (1986) Corrosion and Corrosion Protection of Prestressed Ground Anchorages, Thomas Telford Ltd., London, 28 pp.

Gosschalk, E. M. and Taylor, R. W. (1970) Strengthening of Muda Dam foundations using cable anchors. Proc. 2 Cong. Int. Soc. Rock Mech., Belgrade, 3 pp. 205-210.

Gourlay, A. W. and Carson, C. S. (1982). Grouting Plant and Equipment. Proc. Conf. Grouting in Geotechnical Engineering, New Orleans, February 10-12, pp. 121-135.

Graber, F. (1981) Excavation of a VSL rock anchor at Tarbela. VSL Silver Jubilee Symposium, Losinger Ltd., Berne, Switzerland.

Greenhut, S. (1988) Dangerous Dams. Constructor, March, pp. 28-31.

Hanna, T. H. (1982) Foundations in Tension-Ground Anchors. Trans. Tech. Publications. McGraw Hill Book Co., New York, 573 pp.

Henn, R. W. and Yow, M. G. (1985) Field installation: large post-tensioned anchors. Hydro Review, Winter, pp. 81-85.

Hobst, L. and Zajic, J. (1983) Anchoring in Rock and Soil. Elsevier Scientific Publishing Co., Amsterdam, Holland.

Houlsby, A. C. (1988) Improvements in Grouting of Large Ground Anchors. ASCE Jour. Geot. Eng. Divn., 114, (4), April, pp. 448-468.

Littlejohn, G. S. (1972) Some empirical design methods employed in Britain. In: Questionnaire on Rock Anchor Design, Geotechnics Research Group, Dept. of Engineering, Aberdeen University, Scotland (Unpublished).

Littlejohn, G. S. (1982) The Practical Application of Ground Ancorages. Proc. 9th FIP Congress, Stockholm, Sweden, June 6-10, 6 pp.

Littlejohn, G. S. and Bruce, D. A. (1977) Rock Ancors: State-of-the Art. Foundation Publications Ltd., Brentwood, Essex, England. 50 pp.

Littlejohn, G. S., Bruce, D. A., and Deppner, W. (1977) Anchor field tests in Carboniferous Strata. Revue Fransaise de Geotechnique, 3, pp. 82-86.

Mohamed, K., Montel, B., Civard, A., and Luga, R. (1969) Cheurfas Dam Anchorages: 30 years of control and recent reinforcement. Proc. 7 Intl. Conf Soil Mech. Found. Eng., Mexico City, Specialty Session 15, pp. 167-171.

Nuss, L. K. (1988) Strengthening of a Thin Arch Dam with Post Tensioned Anchors. Stewart Mountain Dam, Arizona. Proc. 8th Annual USCOLD Lecture Series. Paper 8, 28 pp.

PTI (1986) Recommendations for Prestressed Rock and Soil Anchors. Post Tensioning Manual, Fourth Edition, pp. 236-276.

Standig, K. F. (1984) Rehabilitation of the Delta Dam. Water Power and Dam Construction, December, pp. 21-24.

Thurnherr, F. (1982) Permanent Ground Anchors - Recent Developments. Proc. Int. Symp. Soil and Rock Improvement Techniques, AIT, Bangkok, Thailand, November - December, 17 pp.

Water Power (1978) Use of rock anchors at Tarbela. February, pp. 44-47.

TABLES 1

	ROCK PARAMETERS									HYDROGEOLOGIC PARAMETERS		
	Structure/ Bedding/ Fractures	UCS	Shear Str.	Abrasiveness	Tensile Strength	Weathering	Permeability	Unit Wght.	E Value	GWL	Flow/ Movement/ Pressure	Agressivity
DESIGN												
Overall Stability	4	3	3	0	3	3	2	4	2	4	4	0
Rock Grout Bond	2	4	4	0	3	4	3	1	3	1	1	1
Grout-Steel Bond	1	3	1	0	1	1	2	1	3	0	0	0
CORROSION PROTECTION												
Design of Tendon	2	0	0	0	1	2	4	0	1	4	4	4
CONSTRUCTION												
Drilling	3	3	2	4	2	3	1	2	3	3	1	1
Installation	3	0	0	0	0	2	0	0	0	1	4	0
Water Test/Pre-grout/Redrilling	4	1	1	0	0	2	4	0	0	4	4	2
Grouting	3	0	0	0	0	2	4	0	0	3	3	4
PERFORMANCE												
Short Term	1	2	2	0	1	3.	1	0	3	0	0	0
Long Term	3	2	2	0	1	3	3	0	1	3	3	4

Table 1 Suggested significance of various site investigation parameters for rock anchorages. (Excluding qualitative descriptions of lithology and petrography, although Core Recovery is an "absolute necessity" - PTI 1986)

Key: No significance - 0
V. Little - 1
Some - 2
A Lot - 3
Essential - 4

CONTRACT	LOCATION	YEAR OF COMPLETION	CONTENT	BEDROCK	DRILLING METHOD	TOLERANCE (WORST)	OWNER
Cannelton Lock & Dam	Tell City, ID	1972	27 inclined anchors, 9" dia. av. 165' deep	Sandstone, sandy shale	Rotary	1 in 120	U.S. Army Corps, Louisville District
Newburg Dam	Newburg, ID	1973	18 inclined anchors, 9" dia. av. 135' deep	Soft shale	Rotary	1 in 120	U.S. Army Corps, Louisville District
Hilde-brand Lock & Dam	Morgantown, WV	1974	70 inclined anchors, 6" dia. av. 110' deep	Siltstone & sand-stone w/ shale seams	Rotary & DTH	None: no inter-section of holes	U.S. Army Corps, Pittsburgh District
Lake Lynn Dam	Lake Lynn, PA	1974		Shale, sandy shale	DTH	About 1 in 100	Allegheny Power Services Corp.
Lock & Dam 3	Monongahela River, PA	1977		Med. hard silty shale	DTH	None	U.S. Army Corps, Pittsburgh District
Pickwich Lock & Dam	Savannah, TN	1978 & 1981	129 vertical & inclined anchors, 9" dia. to 145' deep	Shale, limestone	DTH	Holes within 2' of 80' high conc. face	TVA Chattanooga
Edgar M. Hoopes	Wilmington, DE	1979	10 vertical anchors, 9" dia. 140 - 220' deep	Mica schist	DTH	1 in 120	City of Wilmington
Wallen-paupack Dam	Wallen-paupack, PA	1979	12 inclined anchors, 6" dia. 100' deep	Sandstone and quartzite	Rotary (DTH not allowed)	18" radius at 100' i.e. lin. 67	PA Power & Light Allentown
Lock & Dam 1	Minnea-polis, MN	1979 & 1981	57 inclined anchors, 6" dia., 70' long plus 57 an-chors 5" dia. 50' deep	Limestone, shale	DTH in conc. Rotary in rock	1 in 60	U.S. Army Corps, St. Paul District

CONTINUED

Bagnell Dam	Lake of Ozarks, MO	1982	277 vertical anchors, 9" dia., max. 300' deep	Limestone	DTH	1 in 200, verified on each	Union Electric Co., St. Louis
Bath Co. P.S.	Warm Springs, VA	1982	24 inclined anchors, 8" dia. max. 75' deep	Vertically bedded shale (Drilling strike)	DTH	1 in 576	Virginia E & P Co. Warm Springs Virginia
Wilson Shoals,	Muscle AL	1982	8 vertical anchors, 9" dia. max. 188' deep	Limestone	DTH	1 in 120	TVA Dam Chattanooga
Barker Dam	Denver, CO	1984	59 vertical plus 35 anchors, 9" dia. 70 - 225' deep	Grantic Gneiss	DTH	1 in 100	Public Service Co. Colorado
Mont-gomery Lock & Dam	Beaver, PA	1985 to 1986	379 anchors, 4-1/2" - 6" dia. 110' deep	Shale	DTH in conc. & rock	1 in 120	U.S. Army Corps Huntingdon District
Elkhart Dam	Elkhart, IN	1986	31 inclined anchors av. 210' long (9 - 6")	Overburden & shale	DTH in conc. rotary in rock	1 in 60 overall, 1 in 240 for stra-ightness	Indiana & Michigan Electric Co
Wallen-paupack Dam 2	Wallen-paupack, PA	1986	33 vertical anchors 8" dia. av. depth 100'	Sandstone and Quartzite	Rotary (DTH not allowed)	1 in 120 to 1 in 240 in places	PA Power Light Co. Allentown
Lock & Dam 8	Point Marion, PA	1988	63 inclined anchors av. 80', 6" dia.	Sandy shale	DTH	1 in 100	U.S. Army Corps, Pittsburgh District
Shepaug Dam	Near Danbury, CT	1988	97 inclined anchors from 75-210' long 10" dia.	Schist	DTH	Max. deviation of $1/2^{o}$	N.E. Utilities Connecticut

Table 2 Summary details of dam anchoring projects undertaken by companies of the NiCon Corporation

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

Table 3 Calculated Specific Gravities of water/cement grouts (Littlejohn and Bruce, 1977).

Tendon	Class of relaxation	Typical relaxation of stress					
		Elapsed time					
		0.1 h	1.0 h	10 h	100 h	500 h	1000 h
		%	%	%	%	%	%
Prestraightened wire	Normal	0.25	0.80	1.90	2.90	4.00	4.50
	Low	0.09	0.20	0.40	0.65	0.95	1.10
7-wire strand	Normal	0.35	1.15	2.10	3.40	4.70	5.50
	Low	0.07	0.33	0.60	0.84	0.98	1.10
Low alloy bar	Normal	0.90	1.80	2.45	2.75	2.90	3.05
Stainless steel wire and bar*		1.0	3.0	4.5	5.1	5.9	6.3

* Relaxation figures for are wire only and are based on very limited information.

Table 4 Typical relaxation of stress (as percentage of initial stress) from an initial stress of 70% GUTS at a temperature of 68° F (BS 8081, 1988).

FIGURES 1-11

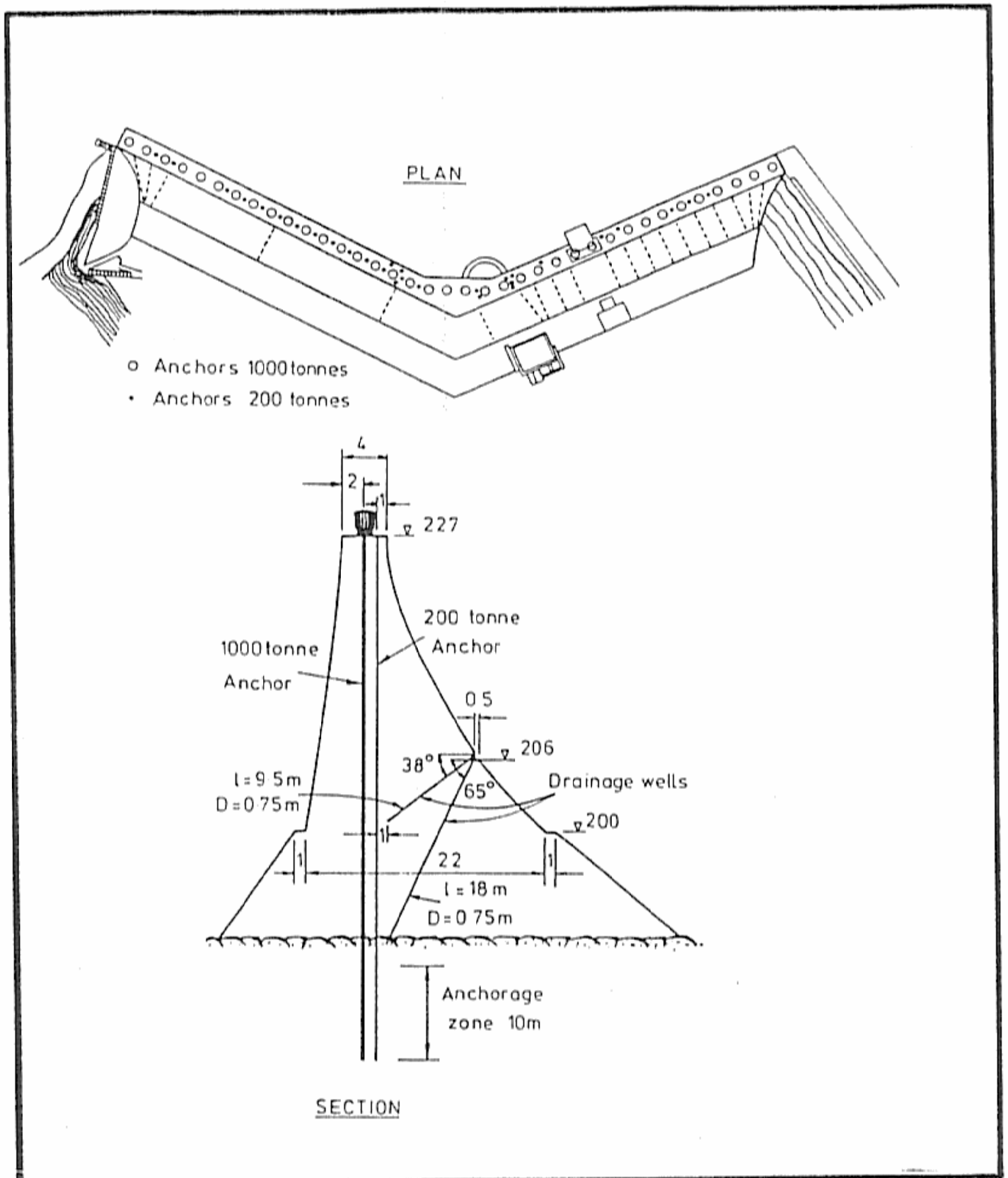


Figure 1. Layout of anchors used to strengthen Cheurfas Dam, Algeria (After Mohamed et al, 1969).

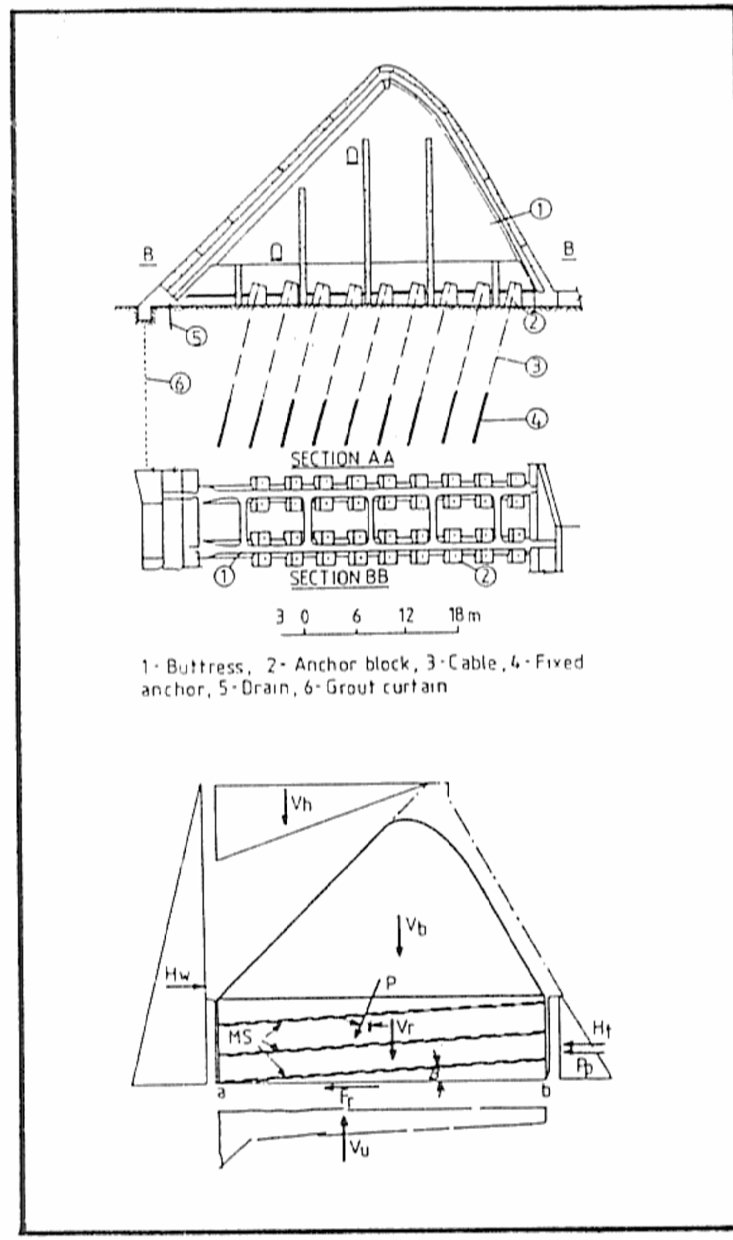


Figure 2. Details of buttress anchoring at Muda Dam, Malaysia (Afte Gosschalk and Taylor, 1970).

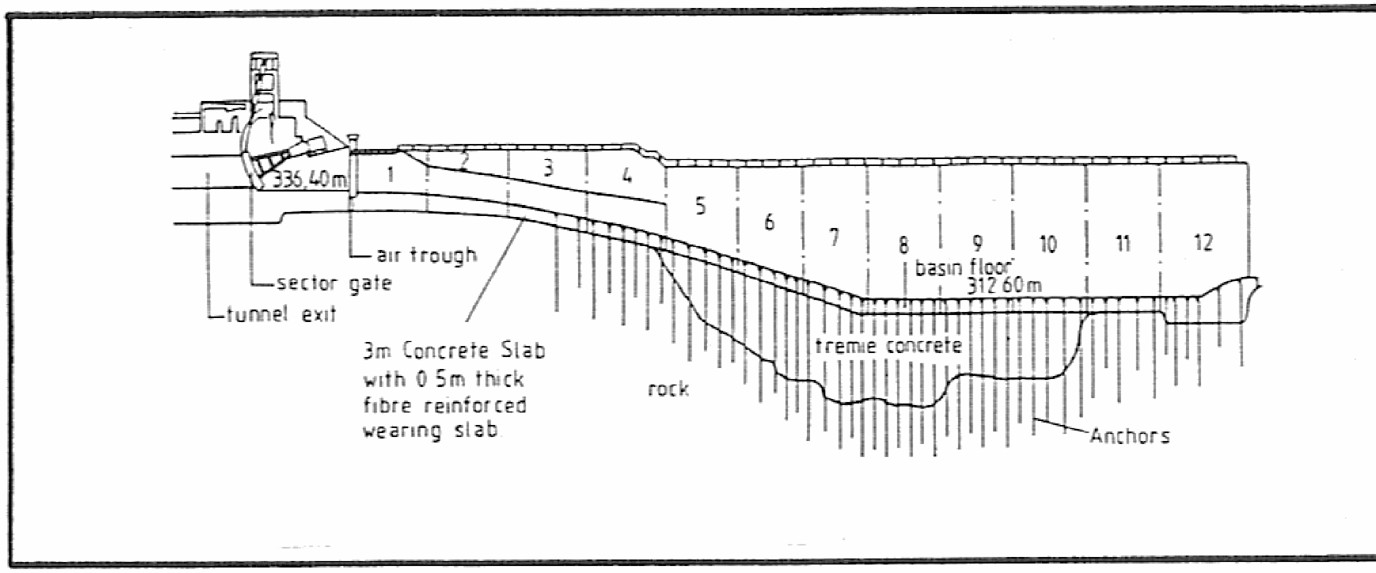


Figure 3. Layout of anchors for repair of Stilling Basin 3, Tarbela Dam, Pakistan (Water Power, 1978).

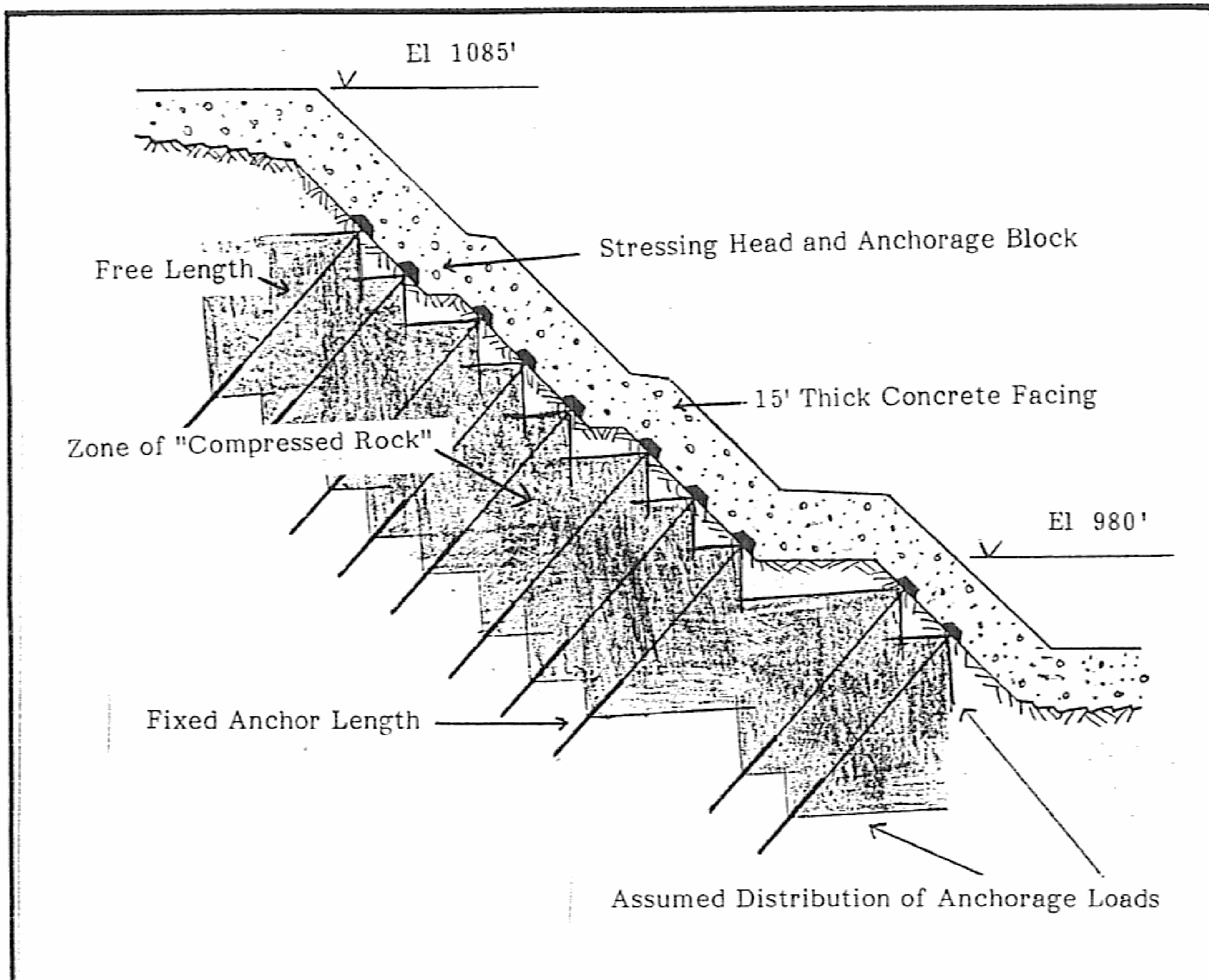
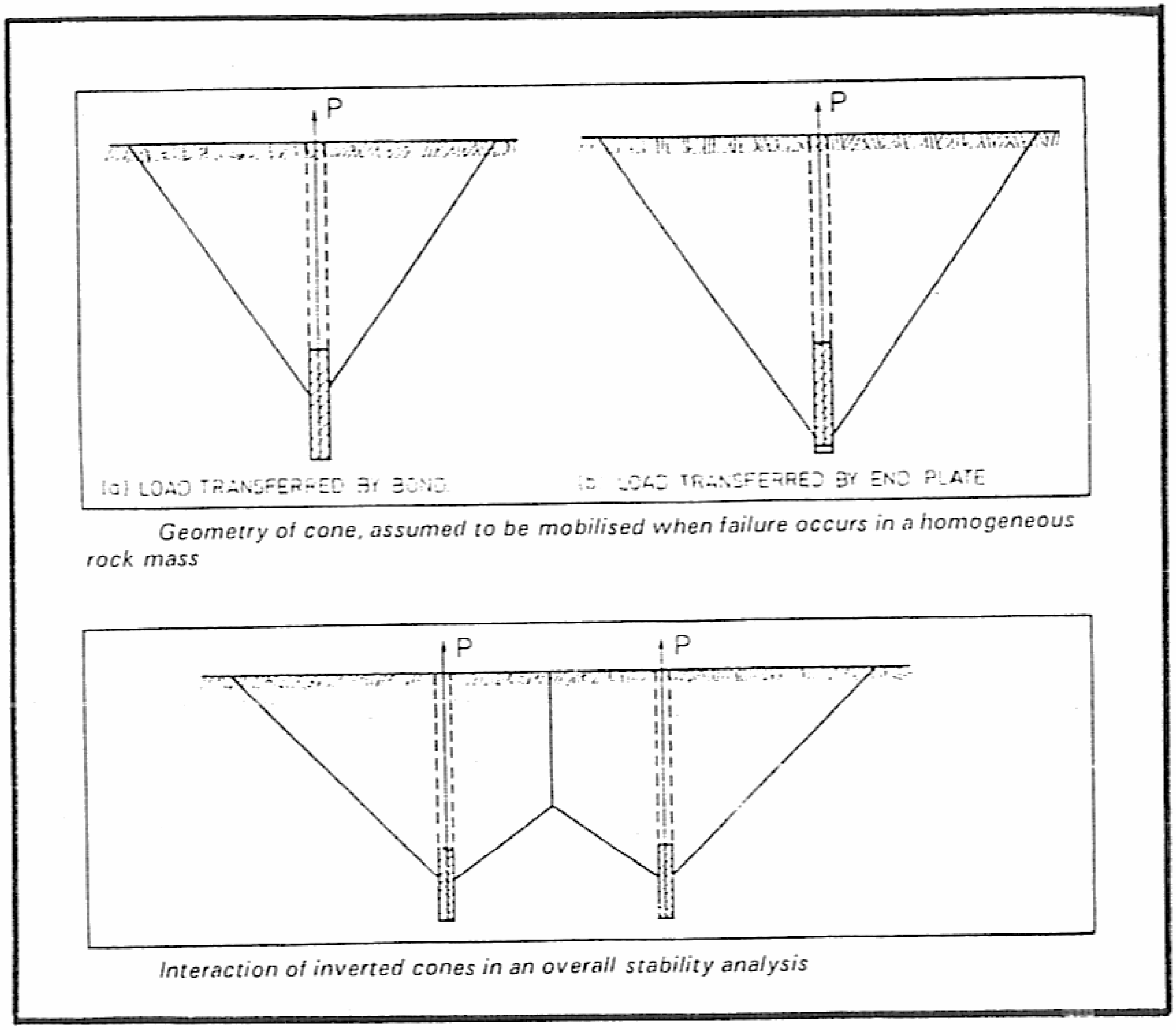
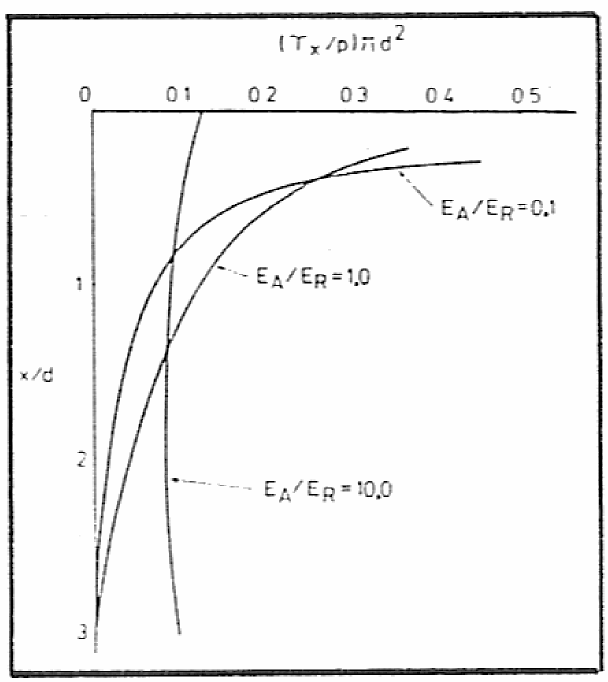


Figure 4. Use of anchorages for slope stability and rock reinforcement. Service Spillway Plunge Pool, Tarbela Dam, Pakistan. (Idealized section)



<u>Figure 5</u>. Conventional basis of designing for ultimate resistance to pull out in homogeneous rock masses (Littlejohn and Bruce, 1977).



<u>Figure 6</u>. Variation of shear stress with depth along the rock/grout interface of an anchorage. (After Coates and Yu, 1970).

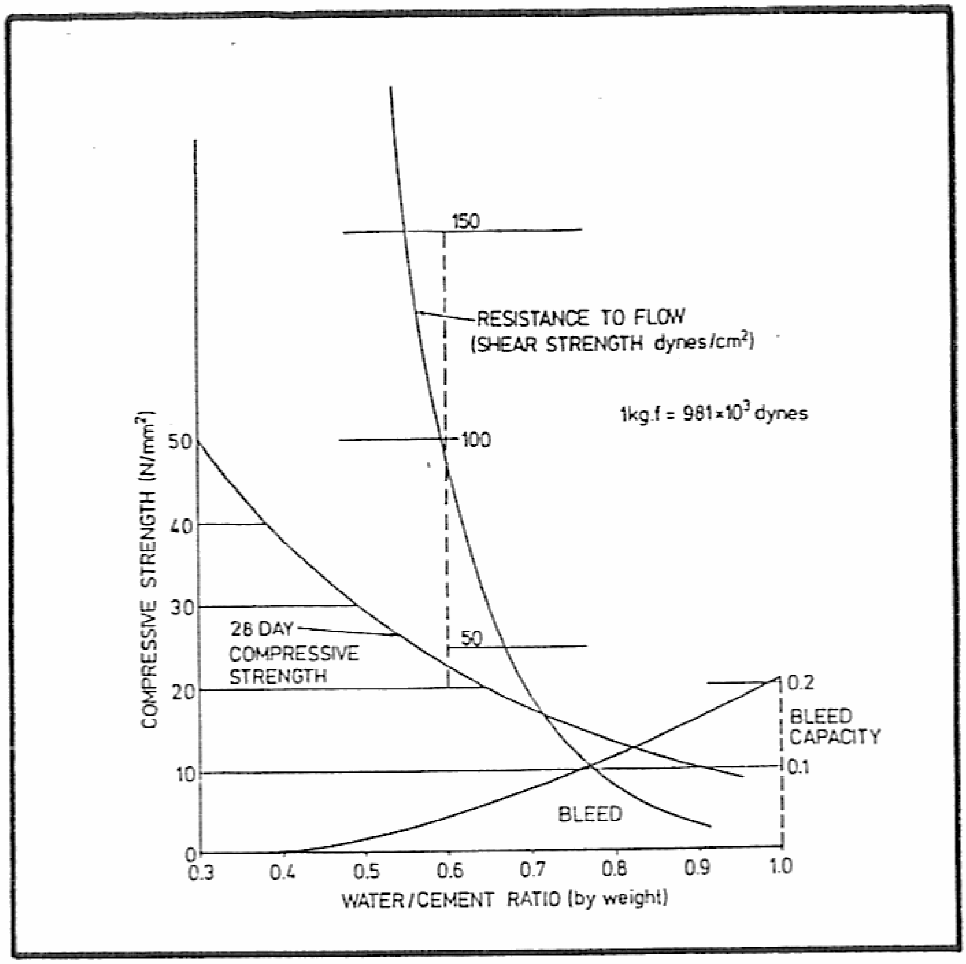


Figure 7. Influence of water content on cement grout strength, bleed and fluidity (Littlejohn and Bruce 1977).

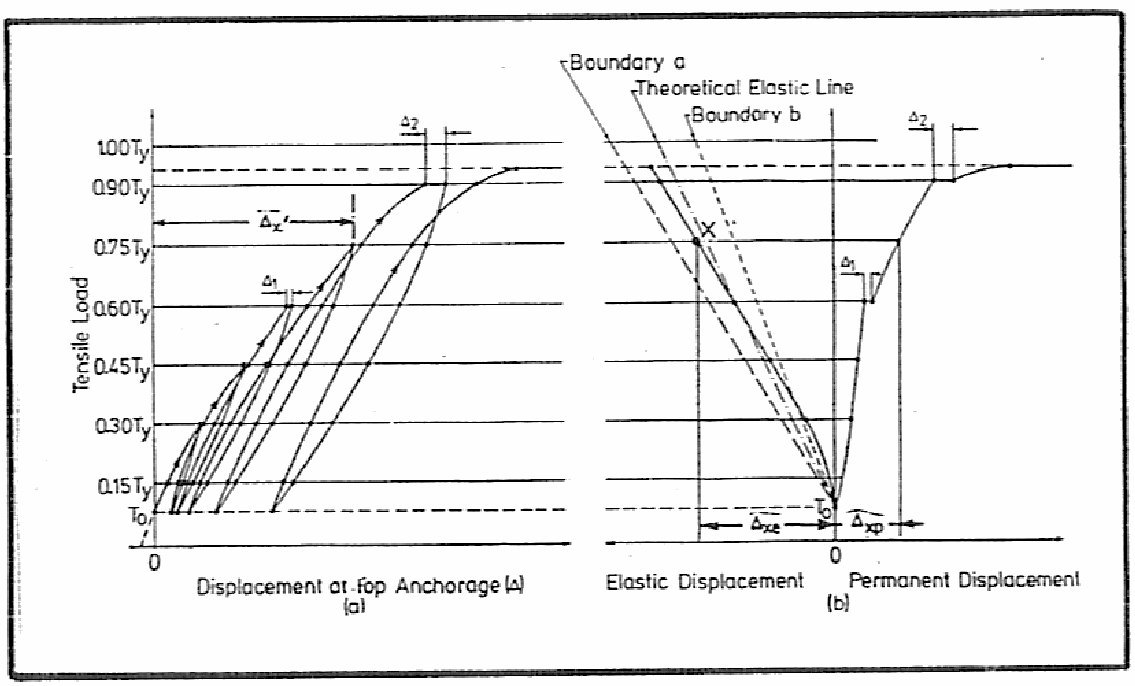
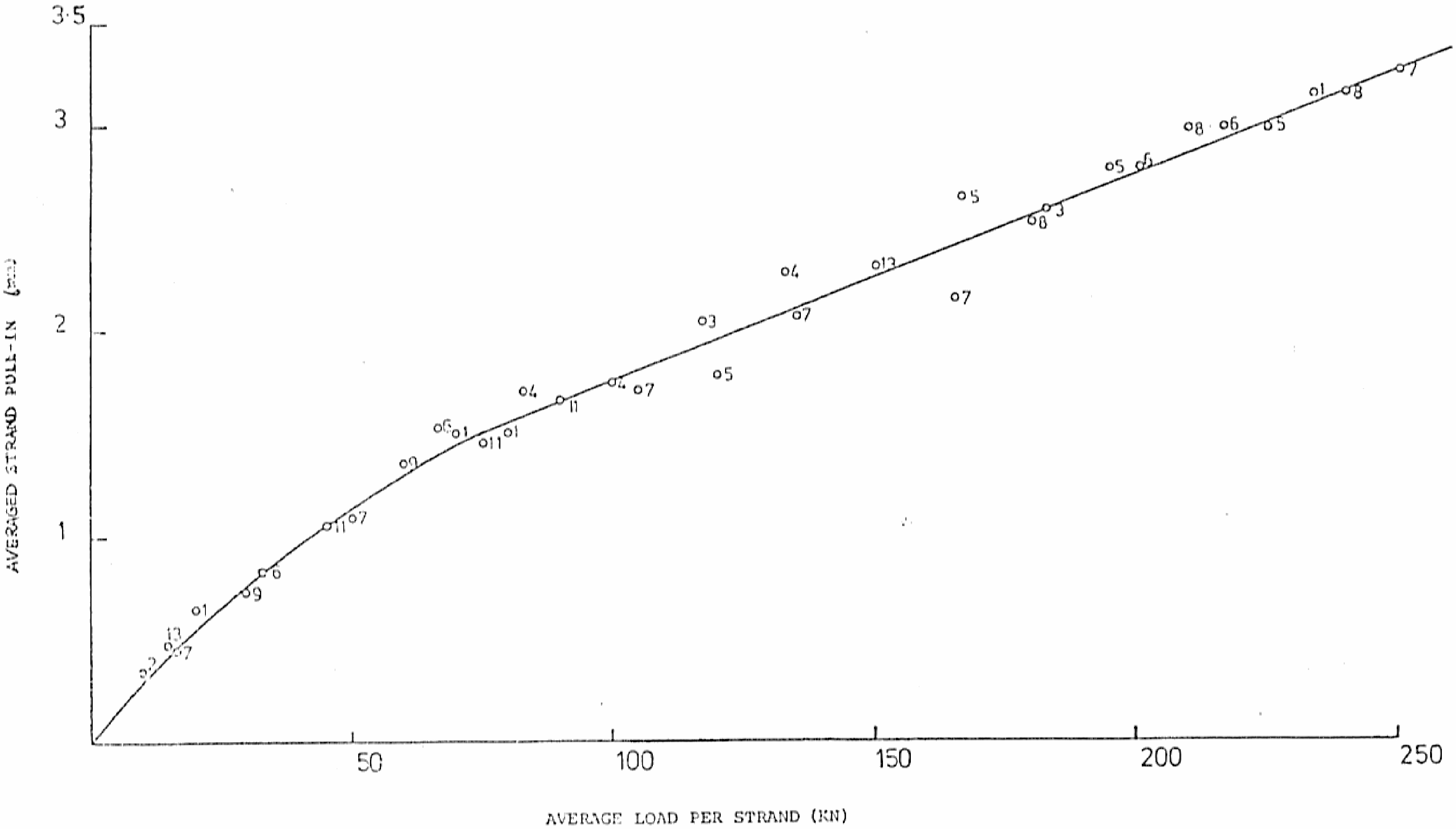


Figure 9. Resolution of total tendon extension (a) into Elastic and Permanent (Residual) Components (b), for analysis of Performance Tests. (Originally in DIN 4125, 1972).



<u>Figure 8</u>. Amount of wedge pull-in measured on individual strands during testing of 23 full scale 10 strand anchors. Numbers refer to number of readings at each point. (Bruce 1976) <u>NOTE</u>: 8.9 KN is equivalent to 1 ton.

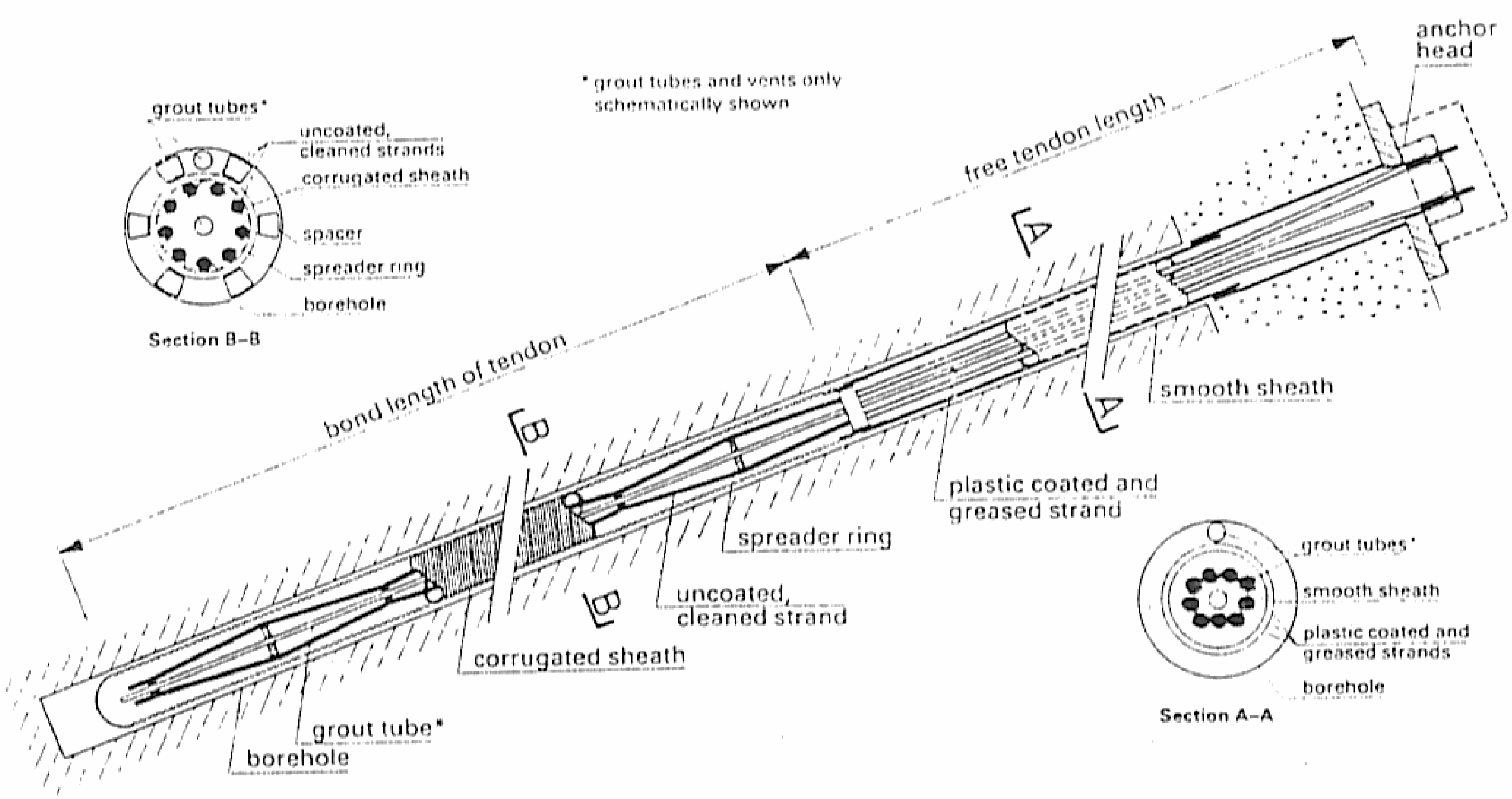


Figure 10. Example of a typical protected permanent tendon (Thurnherr, 1982).

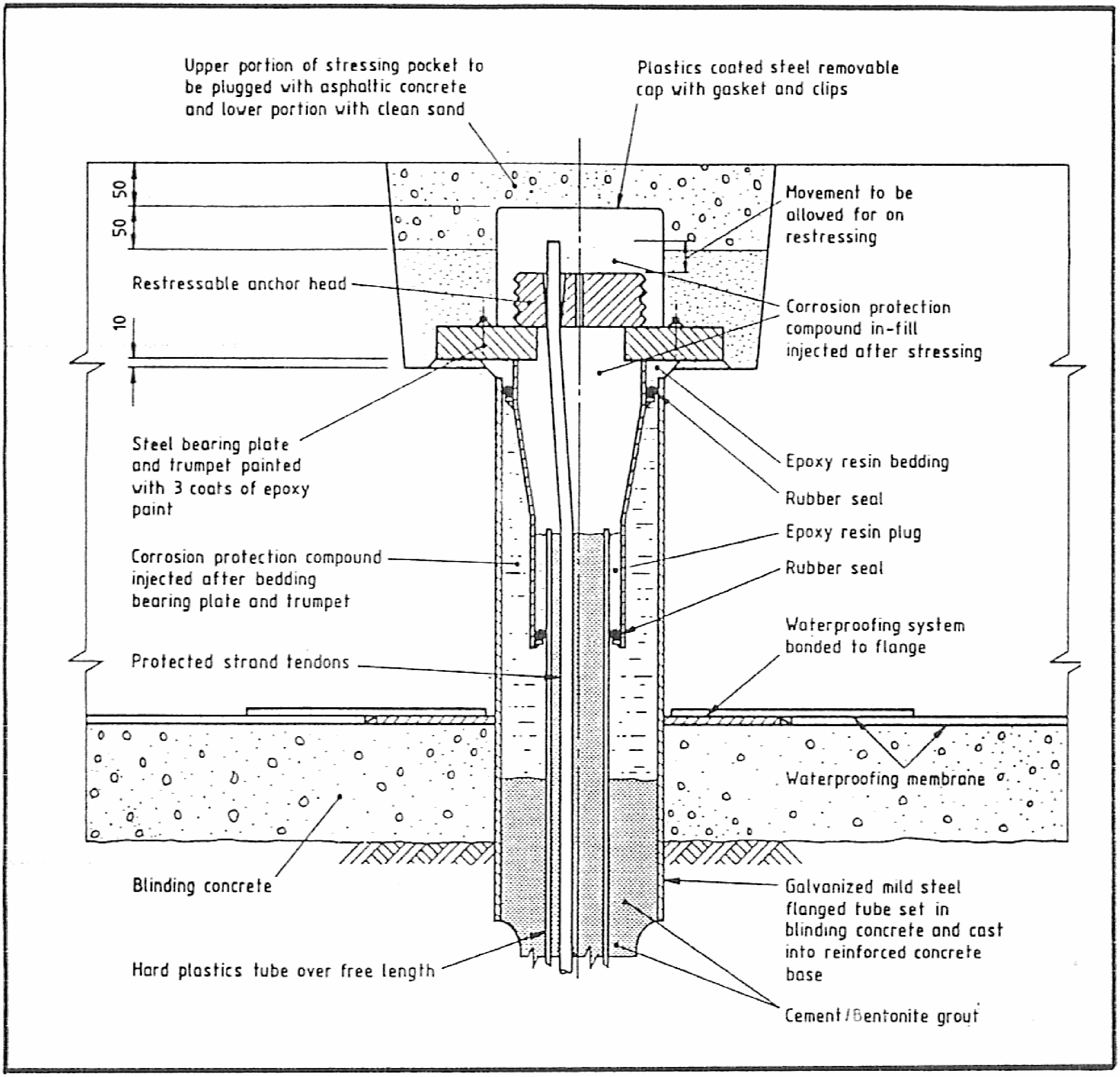


Figure 11. Typical anchor head detail for double corrosion protection of a restressable strand tendon in a water retaining structure (BS 8081, 1988).