THE STABILIZATION OF CONCRETE DAMS BY POST-TENSIONED ROCK ANCHORAGES: THE STATE OF AMERICAN PRACTICE

Donald A. Bruce, M.ASCE*

ABSTRACT

Permanent post-tensioned rock anchorages have been used for over twenty years in America to stabilize existing concrete dams and their appurtenant structures. This paper provides a state of practice review focusing particularly on construction, corrosion protection and performance. Aspects of design are also addressed. Two areas requiring national attention, namely attitudes towards corrosion protection, and long term performance monitoring, are highlighted.

INTRODUCTION

Permanent post-tensioned rock anchorages have been used in America for over twenty years to help existing concrete dams meet contemporary safety standards. Anchorages have been used in dam raising operations where they have proved more economical in resisting the increased overturning movements than the placement of additional concrete mass. However, their most common usage has resulted from dam safety re-analyses, based on the new criteria relating to P.M.F. (Probable Maximum Flood) and M.C.E. (Maximum Credible Earthquake): designs of dams constructed in the first half of this century are often found to be deficient and owners are obliged by law to take appropriate remedial action.

Common applications of anchorages therefore include providing

- resistance to overturning
- resistance to sliding, and
- resistance to seismic effects.

However, in the United States alone, one can also cite their use in a range of ancillary applications, including:

- stabilization of rock abutments
- combatting the effects of alkali/aggregate reaction
- security of tunnel portals and open cuts
- stabilization of excavations for plunge pools, and spillways
- stabilization of lock structures against lateral and vertical forces.

Such dam repairs are conducted throughout the country and extend from private utility owned dams in the northern boundaries,

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* Nicholson Construction of America, P.O. Box 308, Bridgeville, PA 15017

southward through those owned by bodies such as the Tennessee Valley Authority, to the great federally owned structures of the west. As the average age of these dams continues to increase, and our ability to monitor and analyze them improves, so we may expect the use of permanent post-tensioned anchorages to continue to rise.

At this juncture in the United States, we have attained an admirable level of general competency in anchorage technology, although there remain a certain number of details of a practical and philosophical nature where we differ from practice in other countries. Indeed, one of the most fundamental differences is that we have no national standard or code for rock and soil anchorages. The recommendations of the Post-Tensioning Institute (PTI) (1986) come closest, but these are often altered and "improved" upon by individual specification writers, or are, unfortunately, ignored completely and highly "original" project specifications substituted. As a consequence, certain key issues are simply not addressed in a uniform manner, being viewed in a very parochial way, depending on the personal experience of the engineer involved.

The PTI Recommendations are currently under review with the goal to create a national document which will, where judged appropriate, reflect the best of international experience. As background to some of the issues which will be addressed in the field of rock anchorages, this paper provides a brief state of practice review. It focuses especially on the more contentious or less well understood issues in construction and performance.

COMMENTS ON BASIC DESIGN PROCEDURES

The basic design methods for rock anchorages remain largely as reviewed initially by Littlejohn and Bruce (1977), and summarized more recently by Hanna (1982) and Xanthakos (1991). For example, the overall resistance to pullout, by general rock mass failure, is calculated using simple assumptions on the geometry of the rock mass conceptually engaged, and the weight of rock in that mass. Certain designers, armed with reliable data on rock mass structure and strength parameters have optimized designs, and safely shortened the fixed anchorage embedment length accordingly. Most still acknowledge the real or potential presence of less competent rock for the uppermost 10 feet and so permit bond zones to be commenced only below such elevations in supposedly fresher, better quality material.

Regarding the design of the bond zone itself, rock anchorages for dams invariably fall into Littlejohn's (1990) Type 'A': straight shafted with gravity pressure grouting (Figure 1). This figure is also a reminder that pull-out capacity in any given ground is not only dependent on the anchorage geometry, but also the construction technique. The choice of assumed rock-grout bond values is traditionally based empirically on the unconfined compressive strength of the rock, or on the results of past successful applications which is valid as long as any variations due to construction method are accommodated. More engineers are becoming aware that the actual bond is not evenly distributed over the whole rock-grout interface but most do not appear to take this into account at the design stage. The most enlightened designers are, however, requiring special pre-production test programs to verify bond values, and time related performance (Bragg et al., 1990; Bruce, et al., 1991; Scott and Bruce, 1992). Such programs have confirmed the mathematical and laboratory theories of load transfer mechanisms, and the relation of bond stress distribution to the

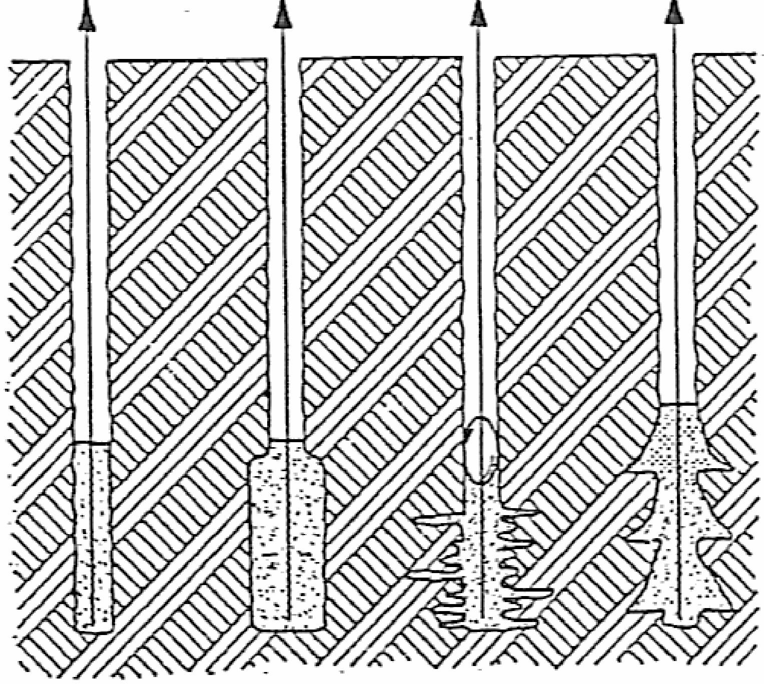


Figure 1. Main types of cement grouted anchors (Littlejohn, 1990)
Type A: straight shaft, gravity grouted
Type B: pressure grouted during installation
Type C: pressure grouted via a sleeved pipe after initial installation grout has set
Type D: underreamed, gravity grouted

elastic modulus of the confining rock mass. In most rock conditions, and specifically where the ratio of the grout modulus to the rock modulus is less than 1, the load is transferred from tendon to rock only in the upper 5-10 feet of the bond zone: the remainder of the bond zone is in effect, the safety factor. The rigid application of "average" bond values may, however, lead to the calculation of extraordinarily and wastefully long bond zones.

Careful analysis of the elastic component of total tendon extensions during performance testing of production anchorages also confirms this phenomenon, further discussed below.

Computers have proven to be invaluable in analyzing structures to determine the amount of additional post-tensioning force and its optimal points of application (Xanthakos, 1991). They have also speeded the calculation of anchorage lengths and geometries, but based always on the basic traditional assumptions of load transfer mechanisms. They appear not to have fostered new methods of anchorage design per se.

ASPECTS OF CONSTRUCTION

Drilling

There has always been concern about the potential the drilling operation may have for damaging the structure. In earlier days, diamond drilling was common as it was considered that this highspeed, low torque method would induce minimum vibrations or flushing pressure surges, and would also drill through steel embedded in the concrete. These advantages, however, are often offset by cost, and by technical drawbacks including a restriction to smaller diameter holes, and the creation of a very smooth borehole wall, not conducive to high bond development.

Contractors involved on larger anchorage projects later adopted rotary drilling methods involving the high torque, high thrust machines otherwise used in water well drilling. Such rotary methods typically provide relatively low penetration rates in all except the softer, argillaceous geologies, and holes can have substantial deviations, given the principle of the drilling action. In addition, the drilling rigs tend to be larger, often truck mounted, and thus frequently difficult to move and position on dams with restricted access.

The use of percussion drilling techniques was often discouraged, and is still prohibited in certain areas. Although top drive percussion is rare in such works, given its limitations on depth, diameter and linearity, down-the-hole rotary percussion has always been favored in certain quarters for such work, and its popularity is rapidly growing. A compact rotary head, and a mast system capable of even moderate pull up and thrust are adequate to move and rotate a drill string. The percussive energy is provided by a down-the-hole hammer, located immediately above the drill bit, and powered by compressed air. This rotary percussive method has been proved to be the fastest, cheapest and straightest way of drilling holes of diameters 4 inches or more through rock and concrete to depths of over 300 feet (Bruce, 1989).

Most recently, the work conducted at Stewart Mountain Dam, AZ (Bianchi and Bruce, 1993) provided an excellent opportunity to demonstrate the advantages of down-the-hole drilling. These included:

- 10 inch diameter holes drilled to over 260 feet with deviations of less than 1 in 200;
- penetration rates of over 60 feet/hour recorded in concrete and granite;
- drill masts could be set up in very restricted access areas to accuracies measured in minutes, in both inclination and bearing;
- the effect of the compressed air flush on lift joints was minimal. As shown in Table 1, movements induced during drilling were of the same order as those arising from diurnal temperature fluctuations.
- the impact of the hammer vibration was minimal. Figure 2 shows that the Peak Particle Velocities induced by the drilling were well under the Owner's acceptance criterion at the hammer impact frequency.

It will be noted that Stewart Mountain Dam was regarded as a very delicate structure, being a thin arch double curvature structure of suspect seismic stability. Each hole was overdrilled, as is standard practice, by 3-5 feet to form a "sump".

Thus, although current practice features a variety of drilling methods, there is no doubt that down the hole drilling is becoming the most popular and accepted choice, and the results from Stewart Mountain Dam only underline this shift of opinion.

Hole Deviation and Measurement

Acceptable tolerances for hole deviation are specified for each project, and reflect the geometry of the dam-anchorage system and the criticality of the structural assumptions. As tabulated by

Elevation of Crackmeter (feet)	Maximum Recorded Movement (in)	Typical Daily Movement due to Temperature Effect only (in)	Approx. Distance from Meter+ to Hole (ft)
1520.39	0.00239	0.00284	5.0
1510.45	0.00218	0.00284	5.5
1500.34	0.00409	0.00432	5.8
1490.43	0.00510	0.00348	5.8
1480.56	*	0.00353	5.5
1470.17	*	0.00376	5.5
1460.21	*	0.00459	5.5
1450.23	*	0.00440	5.5

* No discernable movement was detected during the drilling operation.

\+ Crackmeter mounted on downstream face of the dam adjacent to drill hole.

Table 1. Movements recorded across horizontal lift joints during down-the-hole drilling. Hole 37, Stewart Mountain Dam, AZ (Bianchi and Bruce, 1993).

Bruce (1989), these tolerances have typically ranged from 1 in 60 to 1 in 240, with most being around 1 in 100. Hole straightness is less frequently addressed, although it is wise to consider the possibility of the tendon free length being in contact with the borehole wall during stressing and to generate appropriate straightness criteria reflecting both hole and tendon geometry.

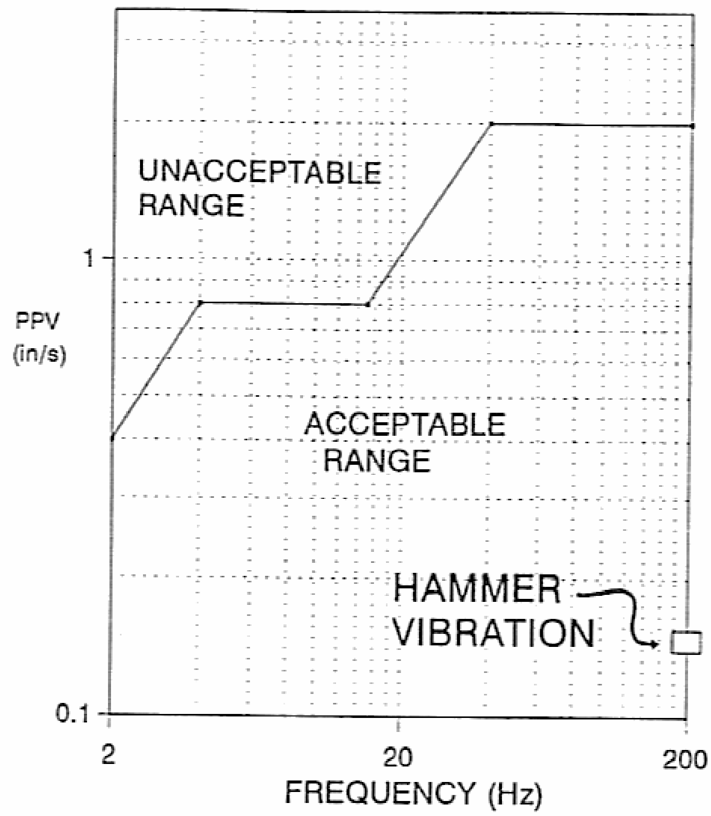
Hole deviations have traditionally been measured after drilling, using various types of inclinometer/gyroscope instruments. These have had various drawbacks, including accuracy, sensitivity, and the time needed to process and analyze the data. The US Bureau of Reclamation has developed an extremely accurate method based on optical principles, but this can operate, economically, only in completed holes, and, practically, only in dry holes.

In the unique case of Stewart Mountain Dam, where hole positions had to be identified at 10 to 20 foot intervals during the drilling of each hole, to provide early warning of the need to correct possible deviations, a rate gyro inclinometer was adapted from the oil exploration industry. This device allowed fast and easily interpreted data to be made available at the drill site, to an accuracy of 1 in 400. These extraordinary advantages are, of course, reflected in the price - a factor which rules it out of common practice.

Water Testing

It is common practice to subject part of each hole at least to a permeability test after drilling. Should the hole, or section of

Vibration Monitoring of Hole 37



Elevation of Geophone (ft)	Approx. Drill Bit Elevation at Max. Peak Vector Sum (ft)	Maximum Peak Vector Sum (in/sec)	Approx. Distance from Geophone* to Hole (ft)
1521.12	1521	0.039	5.0
1517.14	1454	0.017	5.0
1512.94	1581	0.066	5.0
1423.15	1433	0.147	8.0

* Geophone mounted on downstream face of the dam adjacent to drill hole.

Figure 2. Data from geophone monitoring during down-the-hole drilling through concrete. Stewart Mountain Dam, AZ (Bianchi and Bruce, 1993) (USBR Acceptability Criterion)

the hole, accept more water than a criterion states, then it is pregrouted and sealed with a neat cement grout. Such pregrouting is often required in advance in holes which intersect large water bearing fissures at the concrete-rock contact. In such circumstances, bulking agents (such as sand), or flow control additives (such as sodium silicate) are added to help resist washout of the grout prior to its setting. This is a common problem in many older dams built on "horizontal" argillaceous sediments, or in karstic limestone terrains. Equally, holes which interconnect during drilling are routinely pregrouted and redrilled.

Water tightness criteria are typically of the form "0.001 gallons/inch diameter/foot/minute at an excess pressure of 5 psi". As pointed out by Littlejohn (1975), this is not an altogether logical approach: for example, once the hole is filled with water, the outflow reflects the fissure characteristics, not the borehole diameter. In addition, holes may be water permeable, but not grout permeable, and, as the whole point of the exercise is to assure that no anchorage grout subsequently escapes from the borehole, the relationship between fissure geometry and cement particle size is critical.

Littlejohn therefore recommends that pregrouting be carried out only at stage permeabilities of 10 Lugeons or more. This equivalates to a flow of about 0.4 gal/minute at an excess head of 15 psi, and so can be two or three times more generous than the criterion quoted above, depending on hole diameter and assumed stage length. However, since U.S. practice in tendon protection against corrosion is weaker (below), then this extra emphasis on borehole water tightness is not necessarily wasteful. Any hole encountering artesian pressure is usually pregrouted, regardless of the magnitude of inflow. After pregrouting, redrilling is usually accomplished by rotary drilling within 12-24 hours, using air or water flush.

Grouting

High speed, high shear cement grout mixers are now widely used. These ensure uniform and intimate mixing of the cement particles and the water. This efficiency permits the preferred lower water content grouts (w/c = 0.40 to 0.45 by weight) to be used, leading directly therefore to higher and earlier strengths and reduced bleed potential (2-4% acceptable) without the need for additives (Figure 3). Type I/II cement is most common, with Type III restricted to cases where unusually high early strength is required, such as in the case of a short, preproduction test program.

Although some specifications call for the use of special additives to meet various goals, there is no doubt that neat grouts, properly mixed and placed are nearly always adequate. The most notable exception is when grouting anchorages in high temperatures or where long pumping distances are unavoidable. Here, plasticizer/retarding agents, in small amounts, have proved useful in the mixing and injection phases without causing any long term strength problems. On the other hand, additives that cause expansion by producing gas are now discredited for a variety of reasons including grout consistency and corrosion potential. Likewise, gelling or thixotropic additives are also avoided, partly due to the extreme sensitivity of the grout properties to their concentration, and partly due to their presence compromising bond development.

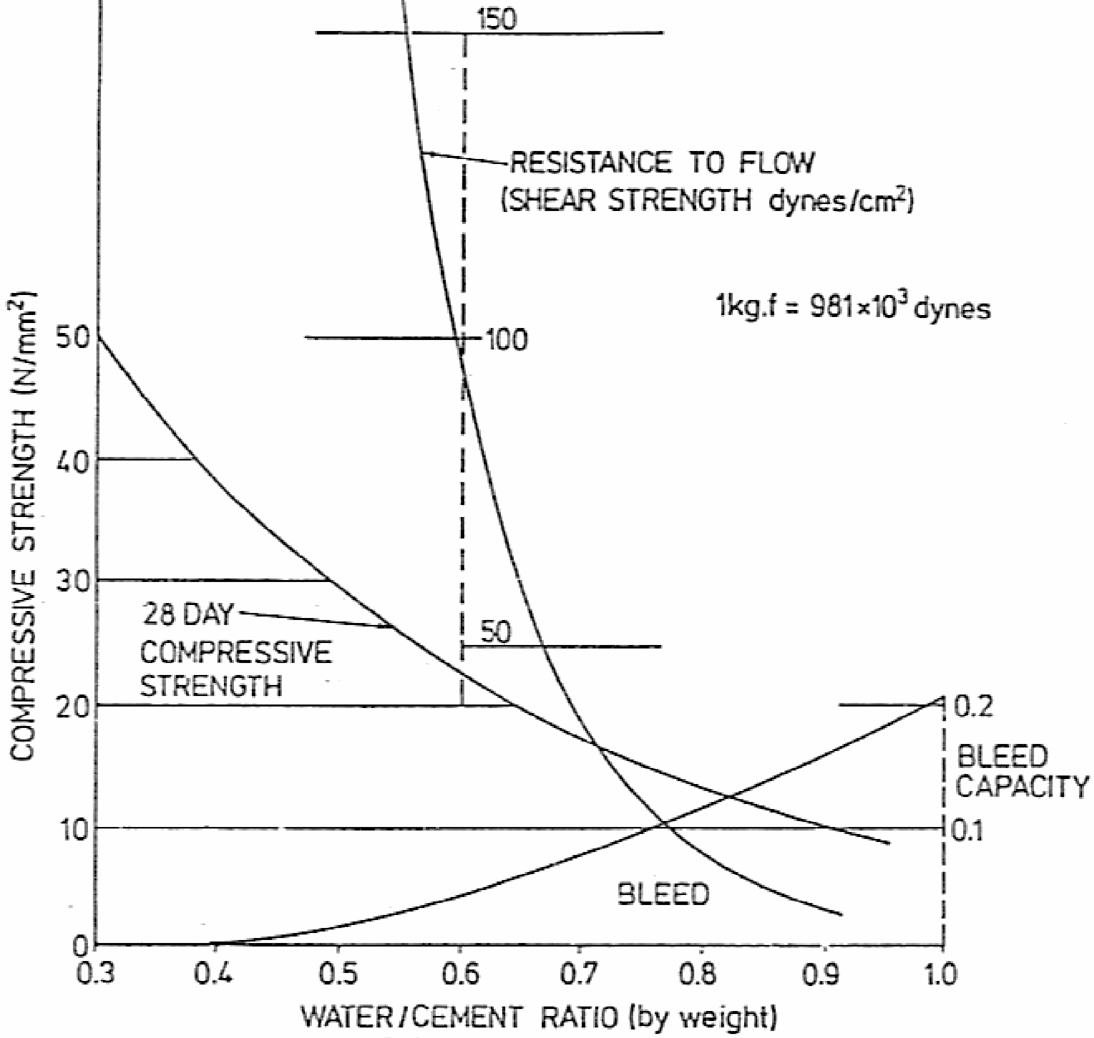


Figure 3. Effect of water content on grout properties
Note: $1\ N/mm^2 \approx 145$ psi (Littlejohn and Bruce, 1977)

Regarding quality control and assurance, cement is usually delivered and measured by the bag, and water by calibrated tank, or by water meter. Quality assurance is still mainly provided, retrospectively, by crushing cubes, the conventional 28 day strength target being 3000 psi. More recently, attention is being paid to testing the fluid properties of the grout also, and the flow cone (fluidity) Baroid Mud Balance (specific gravity and hence w/c ratio), and measuring cylinder (bleed potential) are becoming commonly specified controls.

Special measures are often specified for grouting in especially cold or hot conditions. However, it is most common to simply avoid such conditions by appropriate scheduling of the work.

Tendon Assembly, Installation, and Grouting

Bar tendons tend to be restricted to shorter anchorages (say 50 feet) and lower capacities (say about 40 tons). Most commonly, multistrand tendons are used, and the trend is towards high capacity and considerable length: tendons of 58 strands over 300 feet long were installed recently at Lake Lynn Dam, PA (Bragg, et al., 1990).

Tendons are commonly factory assembled, and delivered to site in coils about 8-10 feet in diameter. On certain occasions, they have been placed in their holes by helicopter, but most commonly this is achieved by using mechanical uncoilers (Figure 4), or simply by long mast crane. All specifications call for "controlled" tendon installation.

The component strand is typically 0.6 inch diameter with low relaxation properties. Spacer/centralizer units are specified in the bond zone at regular intervals (usually around 10 feet), with intermediate steel bands to provide a "noded" or rippled effect. These should guarantee a minimum interstrand spacing of 1/4 inch, and a minimum outer grout coverage of 1/2 inch. Spacers in the free length are less common, and more widely separated. Practical and theoretical considerations limit the amount of borehole that can be occupied by the strand to less than 15% of its volume. Tremie tubes are attached during initial fabrication and are most usually located centrally within the tendon. Nose cones are added to minimize the risk of tendon or hole damage during installation.

There are still differences in opinion regarding the acceptability of the strand surface condition. At one extreme are inspectors who will tolerate no rust on the surface: this zeal is misguided, as it is well known that the presence of a light, non flaky corrosion will actually enhance grout/steel bond development. Equally, the presence of rust states that no other surface coating is present, in the form of grease, lubricant or other oils resulting from the manufacturing process.

Grouting is either conducted in one operation (i.e., bond length and suitably decoupled free length, followed by stressing), or two operations (i.e., grout bond length, stress, then grout free length). This is a project specific decision, with the engineer compromising the advantages and problems of each method to optimize the performance. Two-stage grouting, for example, does clarify the stressing analysis, but also makes the grouting operation more complex to control.

CORROSION AND CORROSION PROTECTION

Virtually every rock anchorage installed in a dam is regarded as permanent. Corrosion protection is therefore a vital and integral part of anchorage design and construction. On the global stage, it is perhaps only in this aspect that U.S. practice is perceived as being deficient, even though considerable advances have been made in the last few years following the works of FIP (1986) and Littlejohn (1990) in particular. The major point of difference between U.S. and foreign practice is in the concept of double corrosion protection. Foreign engineers, following their national codes, do not regard cement grout as an acceptable barrier to corrosion, in that it carries the potential for microfissuring under load. This fissuring can be as severe as 1/10 inch wide at 4 inch centers (Graber, 1981) under which conditions the protective alkaline environment can be depassivated quickly in the presence of aggressive anions, notably chloride. An acceptable barrier is one which can be inspected prior to installation. Therefore, a tendon incorporating a plastic sheath, and grouted in place with a normal cement grout is regarded as a singly protected tendon overseas, but a doubly protected tendon in the U.S. The least protected part of the tendon defines the class of protection, and joints or transitions are particularly vulnerable.

Figure 4. Photograph of tendon uncoiler

American engineers may argue, with a certain justification, that most dams are founded on "good", impermeable rock which is then further grouted, if necessary, prior to anchorage installation. In short, the real danger of water percolating through possible microfissures in both rock mass and grout - and then finding a flaw in the plastic protection is generally regarded as a tolerable risk.

Within the last few years, attitudes toward long multistrand tendon protection have undergone the following chronological progression:

a) bare strand in bond zone, individual sheaths on the free length steel;
b) as a) except for a full length, outer "group" sheath of corrugated plastic (polypropylene or polyethylene);
c) epoxy coated strand (and two phase grouting);
d) epoxy coated strand, with individual sheaths in the free length, permitting one phase of grouting.

In the current absence of a national policy towards corrosion protection, individual owners are responsible for specifying the degree of hole corrosion protection they want to pay for. In contrast, the need to efficiently protect the top anchorage hardware - typically more at risk to atmospheric corrosion and mechanical damage - is more widely understood, and so more consistently addressed. Indeed, there is a growing trend to not use the conventional top anchorage hardware: after primary grouting and stressing, secondary grouting is conducted. However, in this case, the upper 20 feet or so of the free length is left uncoated and so the strand is bonded via the grout to the dam over this length. When the grout has set, the temporary top anchorage is removed and the strands cropped off level with the dam crest (Bruce and Clark, 1989).

STRESSING AND TESTING

The PTI Recommendations (1986) form the most common basis for conducting both the routine Proof Tests, and the more onerous Performance Tests. Load-extension data are recorded on the first load cycle, which often generates more anomalous information than if data were recorded only on the second cycle, after certain permanent movements had been eliminated (e.g., bedding in of head plate). Experience with long multistrand tendons (e.g., Bruce and Clark, 1989; Bragg, et al., 1990; Bruce, et al., 1991a) has led to the setting of the Alignment Load (AL) on individual strands using a monojack. In this way, AL, usually about 5% of the Design Working Load (DWL) is precisely placed on each strand: subsequent multijack loading is therefore conducted in the knowledge that each strand is accepting equal load and so no unforeseen overstressing will occur.

At DWL, tendon stresses are typically 50-60% GUTS (Guaranteed Ultimate Tensile Strength) while at Test Load (TL) tendon stresses over 80% GUTS are prohibited. Test safety factors are therefore at least 1.33, although rarely over 1.50.

The analysis of stressing data is also conducted according to PTI Recommendations (1986) and acceptability gauged by the relation of actual extensions to "control envelopes" generated by theoretical extensions of acceptable free lengths. For example, the actual elastic extension generated at TL should be between a) that of a tendon of length 80% actual free length, and b) that of a tendon of length 100% actual free length plus 50% bond length.

As an extra aid to analyzing stressing data, it is becoming more common to cycle the load back to AL, after TL has been achieved, prior to raising it again to the final lock off load (typically 5-15% over DWL). This extra cycle provides a means of easily partitioning the elastic and permanent components of total tendon extension at TL. Analysis of the former, by reference to the relationship

$$\text{Extension} = \frac{\text{Load x Length}}{\text{Area x Elastic Modulus}}$$

will permit the amount of apparent tendon debonding to be calculated. This is extremely useful in evaluating basic anchorage performance.On sites with very high quality rock, and where by far the greatest component of total tendon extension will be purely elastic, it is prudent to monitor wedge pull-in to further refine the apparent permanent movement component. This pull-in may be as much as one third of one inch at 80% GUTS. As an additional refinement, jack and structural movements may be monitored, but this is rare except in the case of thin, delicate structures (Bianchi and Bruce, 1993).

LONG TERM PERFORMANCE AND SPECIAL TESTING

In common with the rest of the world, few data are published on the long term performance of anchorages in service. In the vast majority of cases, top anchorages are concreted in, after final stressing, and are therefore unaccessible. In other cases, restressable heads or load cells have been incorporated, but the data, if monitored, are used for internal purposes, and never considered sufficiently interesting for publication. Likewise, structural monitoring of anchored dams is often conducted, but again never published. One may conclude, however, that no long term problems have been noted, with the load losses being, predictably and wholly, due to natural relaxation of the tendon. Against this silent background, the data from Stewart Mountain (Bianchi and Bruce, 1993) are particularly useful, especially the confirmation that the gradual and uniform application of prestress along the crest causes no differential strains between adjacent construction blocks.

One encouraging trend is the willingness of more enlightened owners and consultants to sanction preproduction tests in advance of the main works. For example, the test at Lake Lynn Dam (Bruce et al., 1991) was conducted to establish ultimate grout/rock bond stresses, and to research time dependent performance in a compressible, creep susceptible sedimentary sequence. The latter data in particular, proved of great value in understanding otherwise unexpected phenomena during the stressing of the subsequent production anchorages, and so defused a potentially confrontational situation. The tests done at Stewart Mountain Dam (Scott and Bruce, 1992) contributed directly to that particular job's requirements, but also to the technology at large. One hopes that similar tests will be encouraged - and the results published - in upcoming dam repairs of similar type.

FINAL REMARKS

Prestressed rock anchorages have become a popular and reliable solution to many structural problems inherent in older concrete dams. In the United States, the scale and complexity of these problems has fostered the skill and experience of the dam remediation community to achieve an excellent international reputation. However, certain aspects such as attitudes towards corrosion protection, and long term performance monitoring need still to be addressed in a more systematic fashion. These are challenges facing all involved in the technology, but through the spirit of partnering we have grounds for optimism that these challenges will be fruitfully fulfilled.

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