The Stabilization of Concrete Dams by Post-tensioned Rock Anchors: The State of American Practice

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ABSTRACT
Permanent post-tensioned rock anchors have been used for almost 30 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 commanding considerable national attention, namely attitudes towards corrosion protection and long term performance monitoring, are highlighted.

INTRODUCTION

Permanent post-tensioned rock anchors have been used in America for almost 30 years to permit existing concrete dams meet contemporary safety standards. Anchors 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 anchors include the provision of:

- 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 (Bruce and Groneck, 1994)
- combating the effects of alkali/aggregate reaction
- security of tunnel portals and open cuts
- stabilization of lock structures (Bruce et al., 1996).

Such dam repairs are conducted throughout the country and extend from private utility-owned dams in the northern boundaries, 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 anchors to continue to rise - assuming, always, that current dam safety criteria are not "readjusted" downwards.

At this juncture in the United States, there is an admirable level of general competency in anchor technology, although there remain a certain number of details of a practical and philosophical nature where it differs from practice in other countries. Indeed, one of the most fundamental differences is that the United States has no national standard or code for rock and soil anchors. The Recommendations of the Post-Tensioning Institute (PTI) (1996) come closest, but these have often been 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.

COMMENTS ON BASIC DESIGN PROCEDURES

The basic design methods for rock anchors 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 anchor embedment length accordingly. Most still acknowledge the real or potential presence of less competent rock for the uppermost 3 m, and so permit bond zones to be commenced only below such elevations in supposedly fresher, less permeable, better quality material.

Regarding the design of the bond zone itself, rock anchors for dams invariably fall into Littlejohn's (1990) Type A:straight-shafted with gravity pressure grouting (Figure 1). The choice of assumed rock-grout bond values is traditionally based empirically on the unconfined compressive strength of the rock material, or on the results of past successful applications, which is valid as long as any potential variations due to differing 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 more 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 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 1.5 to 3.0 m of the bond zone: the remainder of the bond zone is in effect, the safety factor. The rigid application of "average" bond values can therefore lead to the calculation of extraordinarily and wastefully long bond zones.

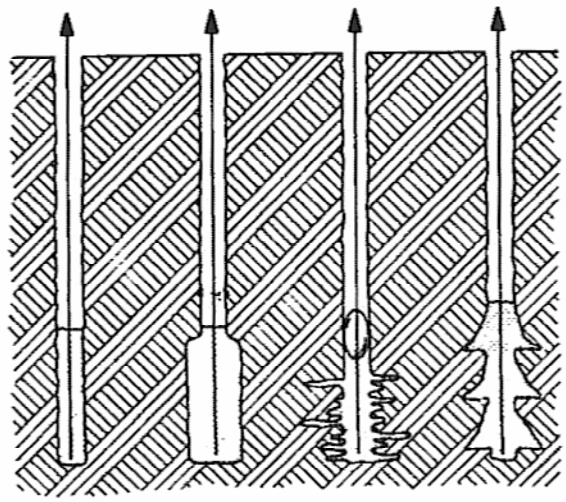


Figure 1. Main types of cement grouted anchors (Littlejohn, 1990).
Type A: straight shaft, gravity grouted.
Type B: pressure grouted during installation.
Type C: pressure grouted via a sleeved pipe after initial installation grout has set.
Type D: underreamed, gravity grouted.

Computers have predictably 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 anchor lengths and geometries, based, however, 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 high-speed, low-torque method would induce minimum structural vibrations or flushing pressure surges, and would also drill through any 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 capacity.

Contractors involved on larger anchor 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 unacceptably large 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 actively discouraged largely due to ignorance, and is still prohibited in certain regions. 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 growing. A compact rotary head, and a stable mast system capable of even moderate pull-up and thrust are adequate to rotate and move 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 100 mm or more through rock and concrete to depths of over 100 m (Bruce, 1989).

The recent work conducted at Stewart Mountain Dam, Arizona (Bianchi and Bruce, 1993) provided an excellent opportunity to demonstrate the advantages of down-the-hole drilling. This work included:

- 250 mm diameter holes drilled at various angles to over 80 m depth with deviations of less than 1 in 200;
- penetration rates of over 20 m/hr recorded in mass 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, despite the use of compressed air at a pressure of 2.5 MPa.

It will be noted that Stewart Mountain Dam was regarded by the Owner 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 1 to 1.5 m 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.

<u>Hole Deviation and Measurement</u>
Acceptable tolerances for hole deviation are specified for each project, and reflect the geometry of the dam-anchor system and the criticality of the structural assumptions with respect to load application. As tabulated by Bruce (1989), these tolerances have typically ranged from 1 in 60 to

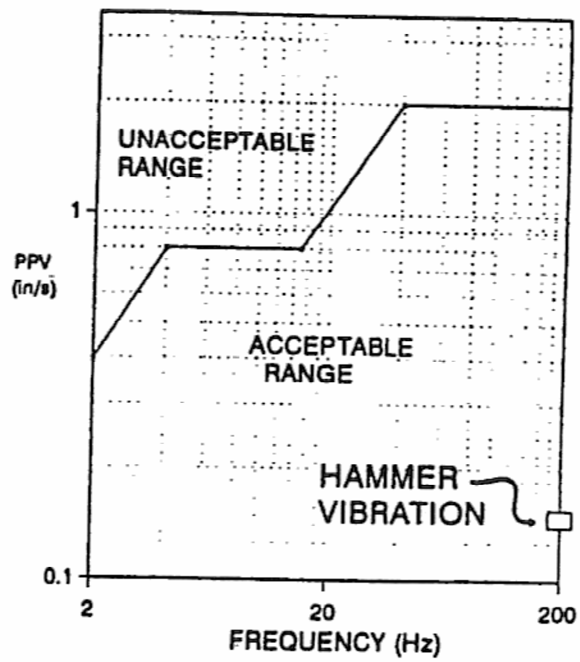
Table 1. Movements recorded across horizontal lift joints in the concrete of the dam during down-the-hole drilling. Hole 37, Stewart Mountain Dam, AZ (Bianchi and Bruce, 1993).

Elevation of Crackmeter (m)	Maximum Recorded Movement (mm)	Typical Daily Movement due to Temperature Effect only (mm)	Approximate Distance from Meter to Hole (m)
463.42	0.061	0.072	1.5
460.38	0.055	0.072	1.7
457.30	0.104	0.110	1.8
454.28	0.130	0.088	1.8
451.28	*	0.090	1.7
448.11	*	0.096	1.7
445.07	*	0.117	1.7
442.03	*	0.112	1.7

* No discernable movement was detected during the drilling operation.
\+ Crackmeter mounted on downstream face of the dam adjacent to drill hole.

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. With the current concerns for seismic stability, anchors for tall intake structures are being increasingly used. The drilling of these structures is placing challenging demands on both the drilling contractors (using closely controlled pilot holes through reinforced concrete followed by reaming to full diameter, with minimal vibration), and the drill hole deviation instrumentation specialists.

Hole deviations have traditionally been measured only after drilling, using various types of inclinometer/gyroscope instruments, as a verification process. These have had various drawbacks, including accuracy, sensitivity, and the time needed to process and analyze the data. Recently, the U.S. 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.



Elevation of Geophone (m)	Approx. Drill Bit Elevation at Max. Peak Vector Sum (m)	Maximum Peak Vector Sum (mm/sec)	Approx. Distance from Geophone to Hole (m)*
463.64	464	0.90	1.5
462.42	443	0.43	1.5
461.14	482	1.68	1.5
433.78	437	3.73	2.4

* 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).

In the special case of Stewart Mountain Dam, Arizona, where hole positions had to be identified at 3 to 6 m 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 drilling 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 qualities are, of course, reflected in the price - a factor which rules it out of consideration in more mundane applications.

Water Testing

It is common practice to subject at least the lower part of each hole to a permeability test after drilling. Should the hole, or section of the hole, accept more water than the specified demands, 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 displacement of the grout prior to its setting. This is a common problem in many older dams built on horizontal, argillaceous sediments, or on 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 anchor grout subsequently escapes from the borehole, the relationship between fissure geometry and cement particle size is critical.

Littlejohn therefore recommended that pregrouting be carried out only at stage permeabilities of 10 Lugeons or more. This equivalates to a flow of about 1.5 liters/min at an excess head of 0.1 MPa, 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 traditionally is weaker (below), then this extra emphasis on borehole water tightness is not necessarily wasteful. Any hole encountering artesian pressure is pregrouted, regardless of the magnitude of inflow. After pregrouting, redrilling is usually accomplished by rotary drilling within 12 to 24 hours, using air or water flush.

Grouting

High speed, high shear cement grout mixers are now widely specified and 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 (up to 2 % acceptable) without the need for additives. Types 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 duration, preproduction test program.

Although some specifications call for the use of special additives to meet various fluid grout goals, neat cement grouts, properly mixed and placed, are nearly always adequate. The most notable exception is when grouting anchors 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 tendon 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 potentially compromising bond development.

Regarding quality control and assurance, cement is usually delivered and measured by the 43 kg bag, and water by a calibrated tank, or by water meter. Quality assurance is still mainly provided, retrospectively, by crushing cubes, the conventional 28-day strength target being 21 MPa. 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 work.

Tendon Assembly, Installation, and Grouting
Bar tendons tend to be restricted to shorter anchors (say 15 m) and lower capacities (say about 40 tonnes). Most commonly, multistrand tendons are used, and the trend is towards high capacity and considerable length: tendons of 58 strands over 90 m long were installed at Lake Lynn Dam, Pennsylvania (Bragg, et al., 1990), and several such case histories now can be cited.

Tendons are commonly factory assembled, and delivered to site in coils about 3 m in diameter. On certain occasions, they have been placed in their holes by helicopter, but most commonly this is achieved by using mechanical uncoilers or simply by long mast crane. All specifications call for "controlled" tendon installation.

The component strand is typically 15 mm diameter, 7 wire, with low relaxation properties. Spacer/centralizer units are specified in the bond zone at regular intervals (usually around 3 m), with intermediate steel bands to provide a "noded" or rippled effect used only in smaller tendons. These spacers usually guarantee a minimum interstrand spacing of 6 to 13 mm and a minimum outer grout coverage of 13 mm. Spacers in the free length are less common, and more widely distributed. Practical and theoretical considerations limit the amount of borehole that can be occupied by the strand to less than 15 percent 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 can also make the grouting operation more complex to control.

CORROSION AND CORROSION PROTECTION

Virtually every rock anchor installed in a dam is regarded as permanent. Corrosion protection is therefore a vital and integral part of anchor 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), Littlejohn (1990), and PTI (1996) in particular. The major point of difference between U.S. and foreign practice has been in the concept of double corrosion protection. Foreign engineers, following their national codes, may 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 2 mm wide at 100 mm 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.

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 anchor 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 and filled strand, with individual sheaths in the free length, permitting one phase of grouting;
e) as d) except for the addition of a full length "group" sheath.

In accordance with PTI (1996), individual owners are responsible for specifying the degree of in-hole corrosion protection they want to pay for. 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 trend to not use a conventional top anchor anchorage: after primary grouting and stressing, secondary grouting is conducted. However, in this case, the upper 6 m 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 anchor is removed and the strands cropped off level with the dam crest (Bruce and Clark, 1989). In general, however, there is no doubt that specifications are becoming far more demanding in terms of corrosion protection, and that epoxy coated/epoxy filled strand is the material of choice. The use of additional plastic protection to individual or groups of strands ensures that the European definition of "double corrosion protection" is being observed.

STRESSING AND TESTING

The PTI Recommendations (1986, now 1996) 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 common practice of setting the Alignment Load (AL) on individual strands using a monojack. In this way, AL, usually about 5 to 10 percent 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, particularly at or near the Test Load (TL).

At DWL, tendon stresses are typically 50 to 70 percent Guaranteed Ultimate Tensile Strength (GUTS), while at TL, tendon stresses over 80 percent GUTS are prohibited. Test safety factors are therefore at least 1.33, although rarely over 1.50.

The analysis of stressing data is also usually conducted according to PTI Recommendations 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 percent actual free length, and b) that of a tendon of length 100 percent actual free length plus 50 percent bond length. However, on certain projects where two stage grouting is to be performed, the lower limit is raised to 90 percent actual free length, since free length friction should not be a factor.

As an extra aid to analyzing stressing data in both Performance and Proof Testing, 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 to 15 percent 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 and resolves certain "gray areas" in earlier versions of the PTI Recommendations. 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 at the stressing head to further refine the apparent permanent movement component. This pull-in may be as much as 10 mm at 80 percent GUTS. As an additional refinement, jack base and structural movements may be monitored, but this is rare except in the case of thin, delicate structures (Bianchi and Bruce, 1993).

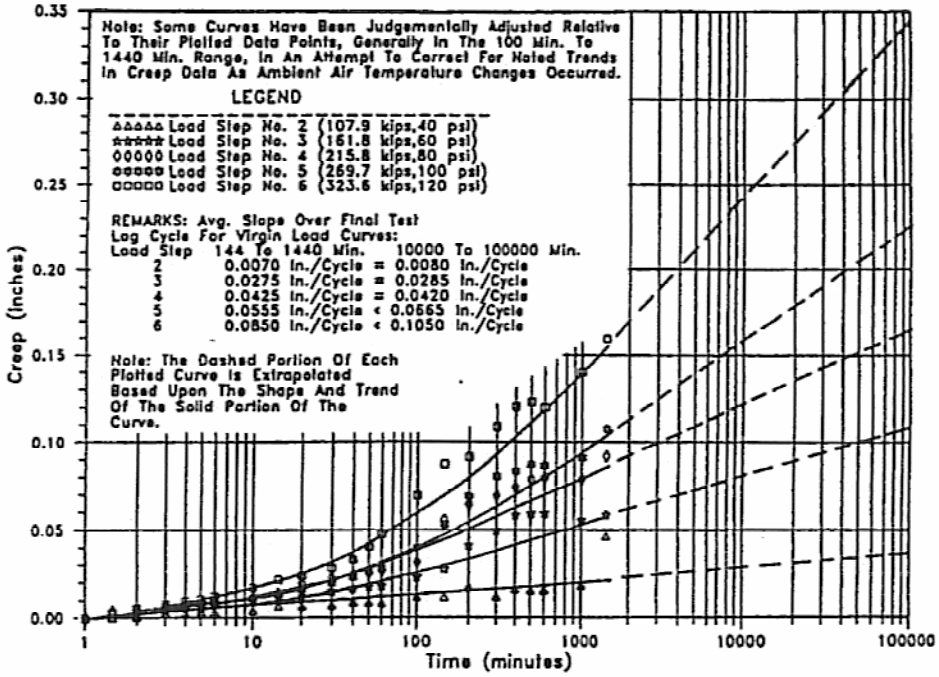
With the trend continuing towards the installation of high capacity anchors (1000 tonnes) in rock potentially capable of long term compression under load (e.g., carboniferous sequences), special attention is being more routinely devoted to extended creep testing (PTI, 1996), and special preproduction tests (e.g., Wimberly et al., 1993). Such tests have been intensively conducted and meticulously analyzed (Figure 3) to the benefit of the dam anchor community at large. A second major time dependent issue that has achieved considerable recent study (e.g., Bruen, 1996; Various, 1996) is the understanding of the special creep performance of the epoxy filled strand. Research has shown that applying the routine anchor creep criteria as developed for bare strand, will automatically lead to the rejection of anchors using epoxy filled strand, even when the anchor installation is otherwise perfectly acceptable. This material-related issue has now been addressed in the new PTI Recommendations (1996), and is well known in the industry.

Lift-off tests are routinely conducted immediately after initial lock off, to verify suitable jack to tendon load transfer.

LONG TERM PERFORMANCE AND SPECIAL TESTING

In common with the rest of the world, few data are published on the long term performance of anchors in service. In the vast majority of cases, top anchorages are concreted in, after final stressing, and are therefore inaccessible. In other cases, restressable heads, or vibrating wire 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 very rarely 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 or creep of the epoxy filled strand. 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 dam crest caused no differential strains between adjacent construction blocks.

One encouraging trend is the willingness of more progressive 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 anchors, 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. Such tests, the details of which are provided to bidders, also help the owner to limit financial exposure by reducing the probability of a claim by the contractor on the basis of changed or unforeseen conditions.



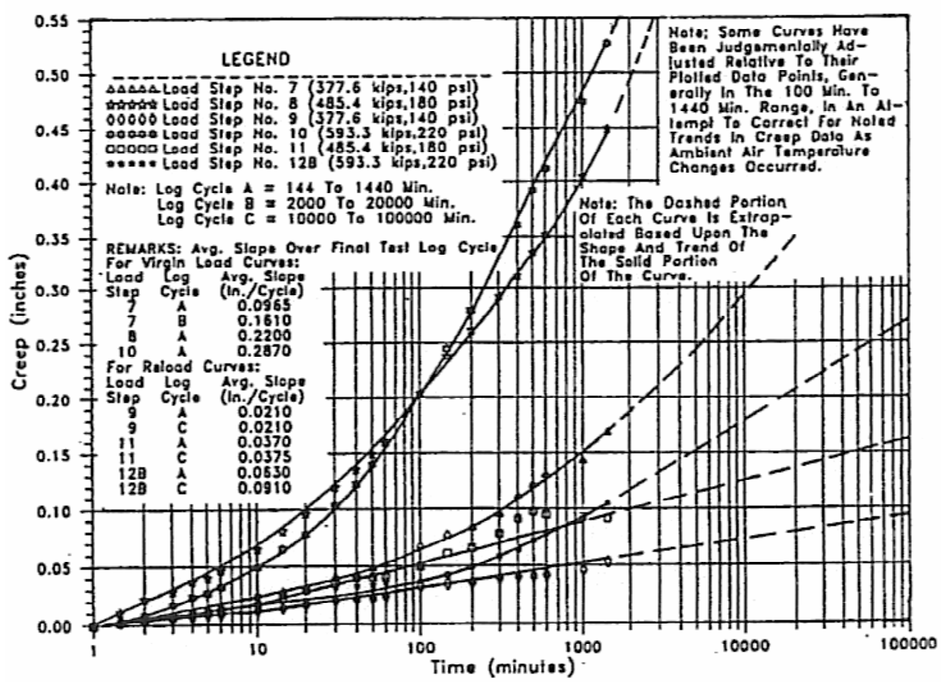


Figure 3. Analysis of Creep Testing Data for Special
Pre-production Tests at Lake Lynn Dam (Wimberly et al., 1993).

FINAL REMARKS

Prestressed rock anchors 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 for construction excellence.

However, recent years have also seen an improvement in standards with respect to corrosion protection, and major contributions to the understanding of long term performance both of the rock mass and the tendon itself. With dam anchoring projects continuing apace, there is optimism that technical levels will continue to rise and that new findings can be shared with colleagues worldwide.

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