LOAD TRANSFER MECHANISMS IN HIGH CAPACITY PRESTRESSED ROCK ANCHORS FOR DAMS

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ABSTRACT

Two recent major dam stabilization projects have provided the opportunity to further study basic load transfer mechanisms in high capacity prestressed rock anchors. Lake Lynn Dam, PA, is founded on a relatively weak, creep susceptible, sedimentary sequence, and anchors were required to combat sliding and overturning potential. At this project, what are believed to be amongst the largest tendons (300' long and up to 58 strands) in the world were installed. At Stewart Mountain Dam, AZ, anchors were required for the novel application of safeguarding a multicurvature thin arch dam, founded on igneous rocks, against seismic forces. At each site, an intensive full scale test program was executed prior to the installation of the production anchors.

1. BACKGROUND

The use of prestressed rock anchors dates from the raising of Cherufas Dam, Algeria, in 1934. Since then dams throughout the world have been successfully repaired using this technique, as well documented in many comprehensive books and papers (e.g. References 1-7). Anchors have been installed in gravity dams principally to increase resistance to sliding or overturning, and in a variety of miscellaneous applications such as abutment security or spillway stabilization. Most recently, anchors have been selected to guarantee the stability of a multicurvature thin arch dam - Stewart Mountain Dam, in Arizona -against seismically induced forces (Reference 8).

Each dam, of course, represents a unique case history, varying in details of construction difficulties, scope and anchor behavior, and several papers are presented annually in the United States alone describing such projects. However, there are still aspects of the design and performance of rock anchors which remain incompletely understood or appreciated,

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but which are accommodated in practice by using very conservative design approaches largely based on previously acceptable results. The two dam anchoring projects described, albeit briefly, in this paper have permitted a rare insight into the fundamentals of certain aspects of load transfer mechanisms and load holding ability. In particular, interfacial bond characteristics (i.e. at the rock-grout, and the grout-steel, interfaces) have been examined, together with the phenomenon of creep in the surrounding rock mass.

Each of these case histories, and indeed each of these topics of investigation, merits a full and detailed description, and further data will be published in due course. In this short paper, however, only an introduction can be provided, giving a glimpse of the significance of the data recorded.

2. FUNDAMENTALS OF LOAD TRANSER MECHANISMS FOR ROCK ANCHORS

When designing rock anchors, the following key topics must be addressed, inter al.:

- (a) Rock Mass Stability
- (b) Rock/Grout Interface
- (c) Grout/Steel Interface
- (d) Tendon Capacity

The overall stability of the rock mass is usually calculated geometrically (References 1-4) and appear to provide extremely conservative results (Reference 9). A great deal of literature is available which provides average working grout to rock bond stresses based on empirical methods, for those designers with no past experience with a particular rock type. Once the bond length is so determined, the grout to steel bond stresses are assumed not to control the design, provided less than 15% of the drill hole cross section is occupied by the tendon. Also, because steel properties are well known, failure should never occur in the tendon itself, provided the stressing is performed accurately. Hereafter, this paper focuses on the interfacial aspects of load transfer.

Rock anchors have been traditionally designed assuming a uniform bond stress distribution over the entire grout to rock surface area of the bond length (Reference 4). However, there are experimental and theoretical analyses which indicate that the load transfer mechanisms are more complex and nonuniform. For example, Coates and Yu (Reference 10) constructed a finite element model to investigate load transfer mechanisms between grout and rock. They found that the load distribution was heavily influenced by the ratio of the modulus of elasticity of the grout to that of the surrounding rock mass (E_a/E_r). Modular ratios less than one gave high concentrations of shear stress in the proximal or upper portion of the bond length, which dissipated rapidly with increasing depth. High modular ratios propagated a uniform distribution of shear stress, (Figure 1).

Experiments conducted by Berardi (Reference 11) confirmed that the bond stress distribution (Figure 2) was independent of the bond length, but the stress was dependent on the anchor's diameter, and the mechanical properties of the surrounding rock, especially its modulus of elasticity. Berardi used strain gages on the anchor tendons to determine the strains at the various load increments. Muller (Reference 12) and Bruce (Reference 13) also produced similar experimental results.

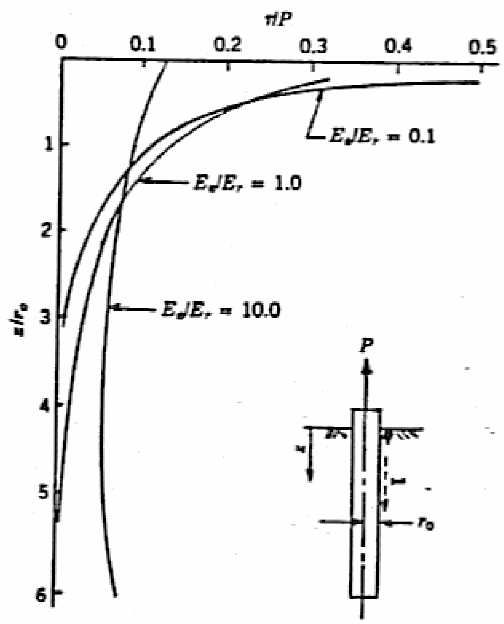


Figure 1 Variation of shear stress with depth along the rock/grout interface. (Reference 10).

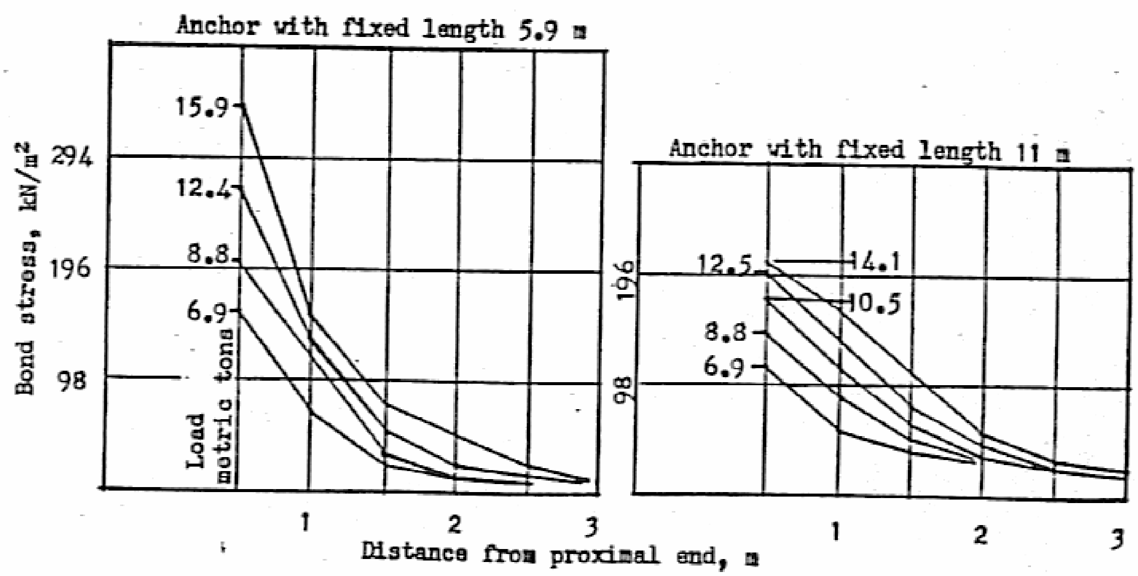


Figure 2 Distribution of bond along fixed anchor length (Reference 11).

These experiments highlighted a fundamental problem in addressing the question of interfacial bond stresses: we believe them to be controlled by the rock and the grout, and yet it is the tendon alone which can be instrumented and analyzed. The authors believe that in most rock anchor installations the amount of apparent bond zone debonding is in fact a grout/steel phenomenon, controlled, however, in its rate and extent by the degree of lateral restraint afforded by the surrounding rock mass.

The two sites in question afforded the opportunity to further investigate this, bearing in mind that one site featured a relatively weak rock mass (ie. low E_r), and the other had a relatively strong rock mass (ie. high E_r).

3. LAKE LYNN DAM

3.1. Introduction

Lake Lynn Dam is a concrete gravity structure nearly 1000 feet in length and a maximum of 125 feet high. As an outcome of the Federal Energy Regulatory Committee (FERC) licensing procedures, the stability of the dam was found to be unevenly matched against the Cheat River's probable maximum flood (PMF). One of the measures adopted to improve the factor of safety against overturning was to increase the effective weight of the dam with the use of 75 high capacity rock anchors (Reference 14).

3.2. Test Program

In 1972, sixteen anchors were installed after the dam's first safety inspection. Based on the results of the test anchor for those anchors, an allowable average grout to rock bond stress of 37.6 psi was anticipated for the design of the new anchors. Before production began on the 75 new anchors, a special test anchor was installed in the same Pennsylvanian clayey siltstone (Casselman formation) in which the production anchors were to be founded: a rock mass which was thought to be creep susceptible. The main thrust of the test anchor program was to verify the bond stress potential and to determine the creep characteristics of the rock under anchor load.

An exploratory hole was cored to investigate the rock at the test site, as shown in Figure 3. The rock quality designation (RQD) was found to vary little with an average of 74%, which is considered fair to good quality rock mass. The unconfined compressive strength of the NX-sized cores, however, ranged between 1000 and 1500 psi only. While no in-situ testing was performed, the modulus of elasticity of the rock mass was estimated to be 150 ksi (Reference 15).

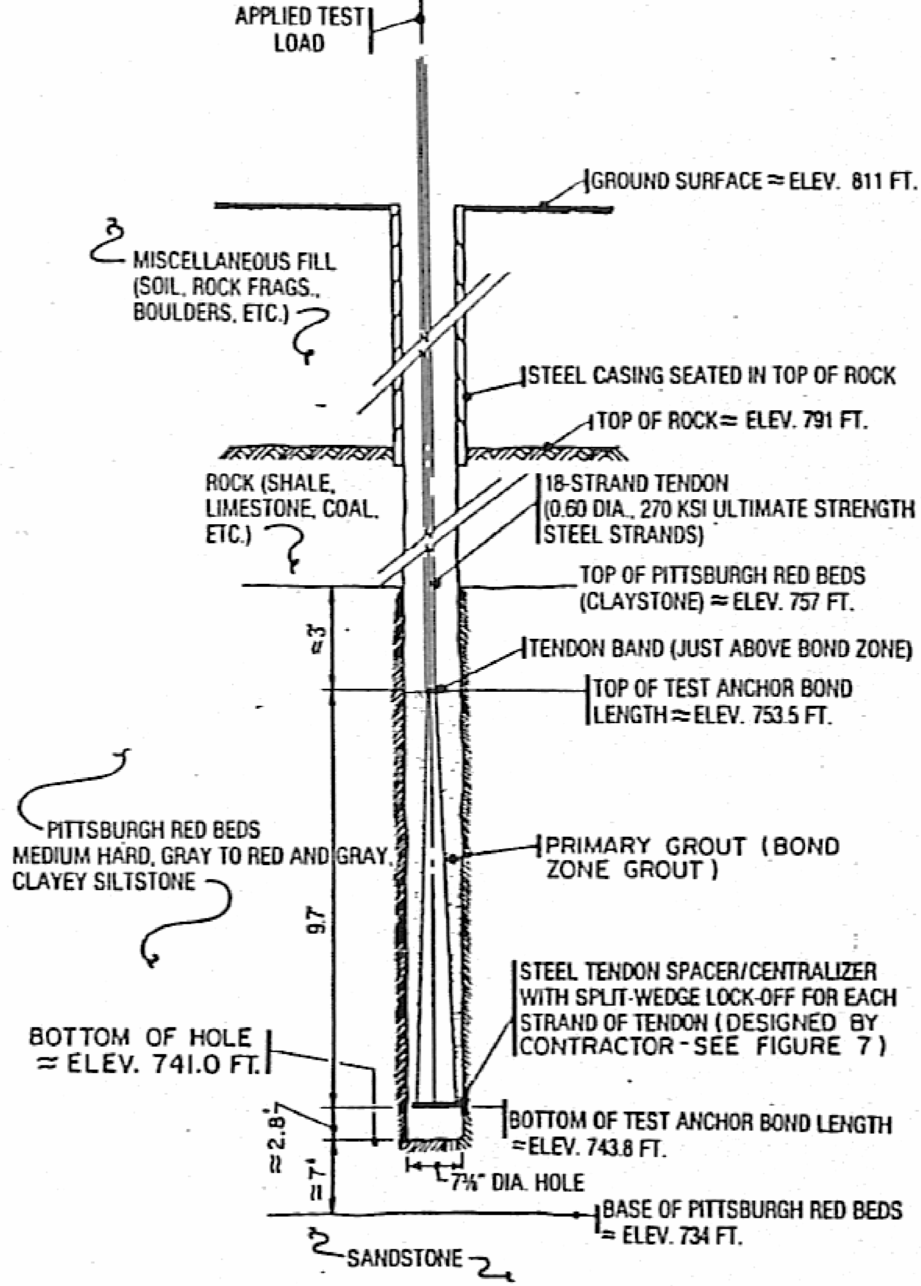


Figure 3 Test anchor and geology details, Lake Lynn Dam, PA

Following the exploratory drilling, the same hole was overdrilled with a 7-3/8 inch diameter down-the-hole-hammer (rotary percussive with air). The top of the bond zone was located 57.6 feet beneath the surface and was 9.7 feet in length. An 18 strand (0.6" diameter) tendon was inserted into the drilled hole and the bond zone was grouted using a neat cement grout with a water/cement ratio of 0.45 (by weight).

After the grout reached 3000 psi (UCS), the anchor was tensioned in 108 kip progressive cycles, in the form of an extended Performance Test (Reference 16) to 831 kips (80% GUTS) (Figure 4).

Analysis of the elastic component of extension indicated an apparent anchor debonding of 38 inches at 831 kips. At this same load, the total permanent displacement was 3.82 inches.

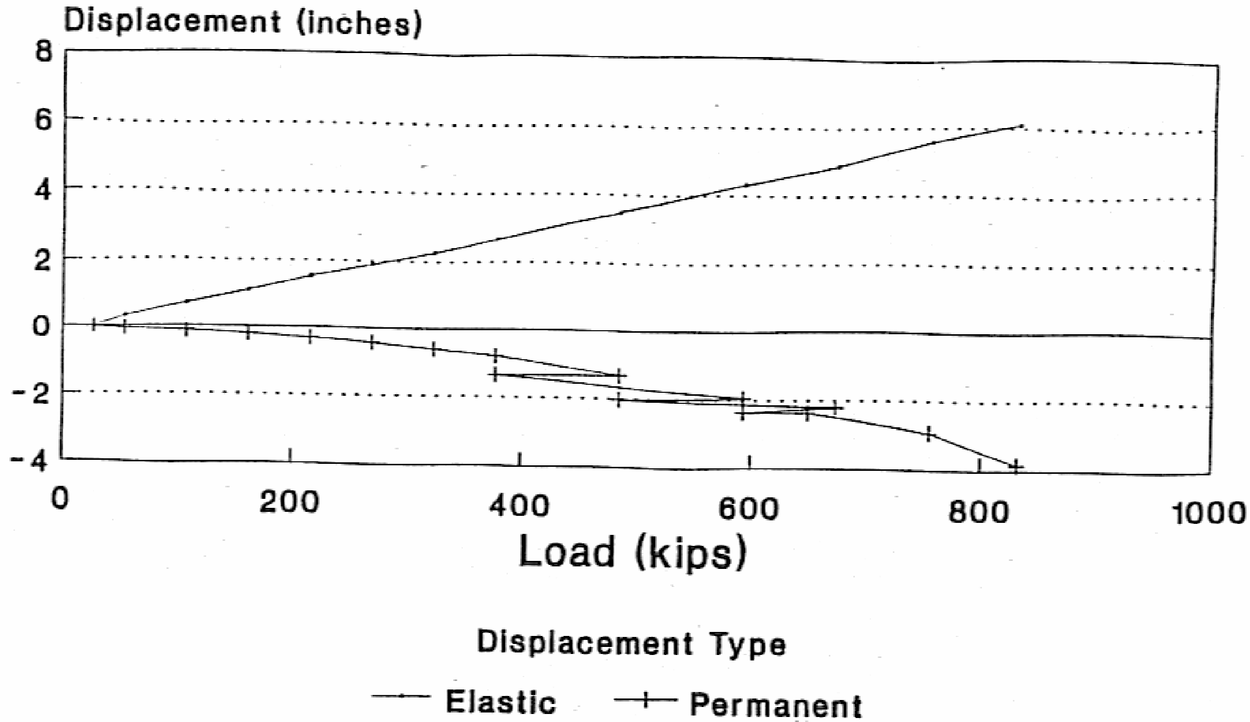


Figure 4 Components of extension, Test Anchor, Lake Lynn Dam, PA.

Whereas the apparent tendon debonding was surprisingly small, this measured permanent movement was atypically large for an anchor in rock. Despite the fact that this test anchor had safely resisted an averaged grout/rock bond stress of over 308 psi, there was no indication in the elastic or creep performance (detailed below) that interfacial failure had occurred. This unusually large permanent movement was therefore not considered to reflect an anchor to rock relative slip, but a crushing ot the highly compressible carbonferous horizons above the bond zone, with the rock/grout contact still intact.

To explore fully the creep phenomenon, twenty-four hour load holds were applied at each cycle maximum load. Also, certain load steps were repeated after higher loads had been achieved. At low stress levels and at each of the repeated load steps, a stabilizing trend was noted. The stabilizing trend resulted from a decreasing creep rate at each load cycle. The amount of creep was found to generally increase with load, as expected. However, it was determined that the amount of creep diminished for those load steps which were repeated (Figure 5, Table 1). This was arguably the most important observation of the test anchor program, for applying the PTI allowable creep values would have indiated unsatisfactory anchor performance, whereas the anchor, as a load carrying and load holding element was clearly efficient. Because of the major improvement obtained after reloading, the project specifications were modified to include this reloading option, rather than routinely rejecting the anchor on the basis of PTI recommendations.

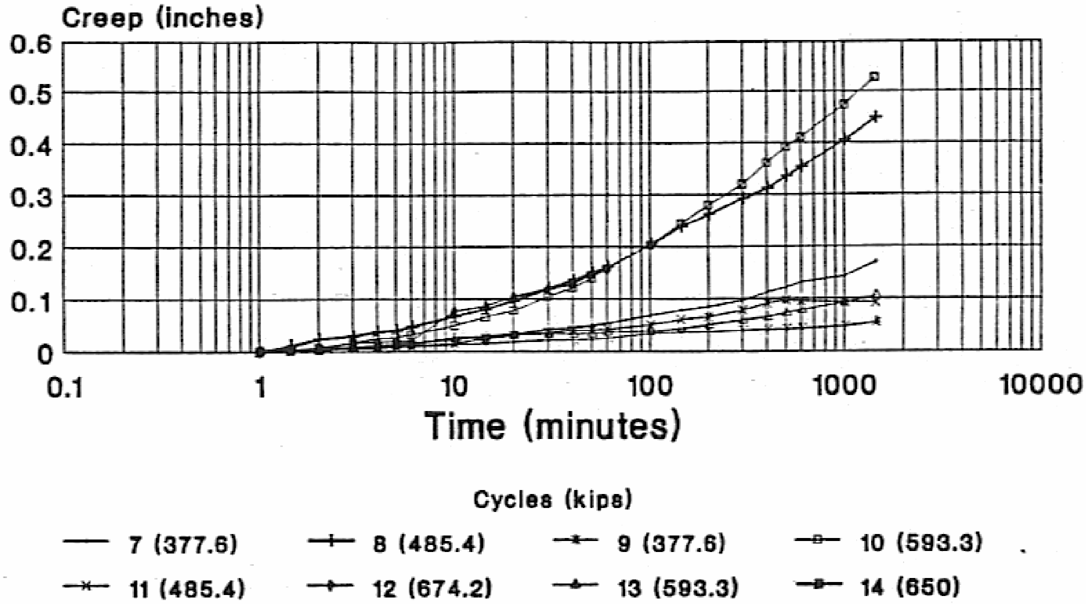


Figure 5 Creep performance at each load cycle maximum, Test Anchor, Lake Lynn Dam, PA (Note: data from lower cycles to 324 kips omitted for clarity).

CYCLE	MAXIMUM LOAD (kips)	CREEP IN 1440 MINS (x0.001")	STABILIZING TREND?	CREEP (x0.001") DIVIDED BY LOAD (kips)
1	53.9	ZERO	Yes	Zero
2	107.9	46	Yes	46
3	161.8	58	Yes	36
4	215.8	92	No	43
5	269.7	107	No	40
6	323.6	160	No	49
7	377.6	170	No	45
8	485.4	449	No	93
9	377.6R	53	Yes	14
10	593.3	527	No	89
11	485.4R	92	Yes	19
12	674.2	500 Est.	No	75
13	593.3R	105	Yes	18
14	650R	75 Est.	Yes	12

Table 1 Summary of creep performances, Test Anchor, Lake Lynn Dam, PA.

3.3 Production Anchors

Details of the production anchors are summarized in Table 2. The production anchors acted very elastically. The average permanent displacement was 1.51 inches, with the maximum permanent displacement being 4.58 inches. Analysis of the Performance Test data proved negligible amounts of apparent tendon debonding. Regarding creep, all except four anchors proved acceptable under PTI regulations, probably reflecting the relatively low average grout-rock bond stress mobilized. Rather than rejecting these four anchors, however, they were restressed to the same Test Load and reevaluated: each then proved wholly acceptable. This was a clear indication of the value of conducting such an intensive test program: the alternative, namely that of somehow replacing the four anchors, would have been extremely costly, if practical at all in the prevailing site conditions.

4. STEWART MOUNTAIN DAM, AZ

4.1. Introduction

Stewart Mountain Dam is located on the Salt River approximately 41 miles northeast of Phoenix, Arizona. It is founded largely on fissured, jointed PreCambrian granites and diorites. During the 1920's, when this 200' high multicurvature, thin-arch dam was being built, the significance of effective construction joint clean-up was not fully appreciated. As a result, horizontal construction joints at 5' intervals had a thin layer of laitance and were very weak, exhibiting little or no cohesion. The U.S. Bureau of Reclamation therefore believed that the arch section was no longer a monolithic structure as designed but a series of unbonded blocks held together by gravity and natural arch dam action. Individual concrete blocks high in the arch would therefore be unstable during a major seismic event because of poor construction joint bond, large inertial forces, and loss of arch action along vertical joints.

The Bureau considers the MCE (maximum credible earthquake) to be of magnitude 6.75, 15 km away, producing an estimated site acceleration of 0.34 g. This seismic event would cause instability of the blocks high in the arch. In order to improve the stability of the vertical cantilever sections and increase factors of safety during normal and seismic loadings, post-tensioned, high capacity rock anchors in the Arch (62 anchors) were judged to be the least expensive and most viable solution. It was also decided to install 22 post-tensioned anchors in the left thrust block to protect against a potential sliding failure at or near the thrust block foundation contact.

In advance of the production works, a full scale test program was called for by the USBR to verify design assumptions with respect to the adequacy of the bond lengths in rock. The opportunity was seized by both the USBR and the contractor to expand this test to more closely examine load transfer mechanisms.

4.2. Test Anchor Program

A pair of vertical anchors (A and B) were installed 12' apart in each of three test sites, representative of the three major rock zones expected to underlie the dam (Figure 6). Details are summarized in Table 3.

		Site 1	Site 2	Site 3
Anchor A	Free Length	19'10"	18'4"	18'2"
	Bond Length	10'2"	11'8"	11'10"
Anchor B	Free Length	20'1"	18'8"	27'0"
	Bond Length	19'11"	21'4"	13'0"
Anchor A/B	Strands	28	28	28
Anchor A/B	Max. Test Load (at 80% GUTS)	1310 kips	1310 kips	1310kips

Table 3 Details of Test Anchor, Stewart Mountain Dam, AZ

In all geotechnical aspects Site 1 rock was slightly superior to Site 2 rock which was in turn very superior to the highly weathered and shattered material of Site 3. Unconfined compressive strengths (Point Load Test) averaged uniformly over 26,000 psi in Site 1, and 19,000 psi in Site 2, while only small fresh samples of similar strength could be tested from Site 3. Rock mass E values ranged from perhaps 1 to 3×10^6 psi (Site 1) to 0.5 to 2.5×10^6 psi (Site 2) to probably around 1×10^5 psi in Site 3.

Each anchor hole was first cored to NX diameter and subjected to a Houlsby Type water test (Reference 17), and dilatometer testing (to estimate insitu E value), prior to being redrilled to full 10" diameter with a down the hole hammer. Tendons consisted of special epoxy coated strands (each 0.6"

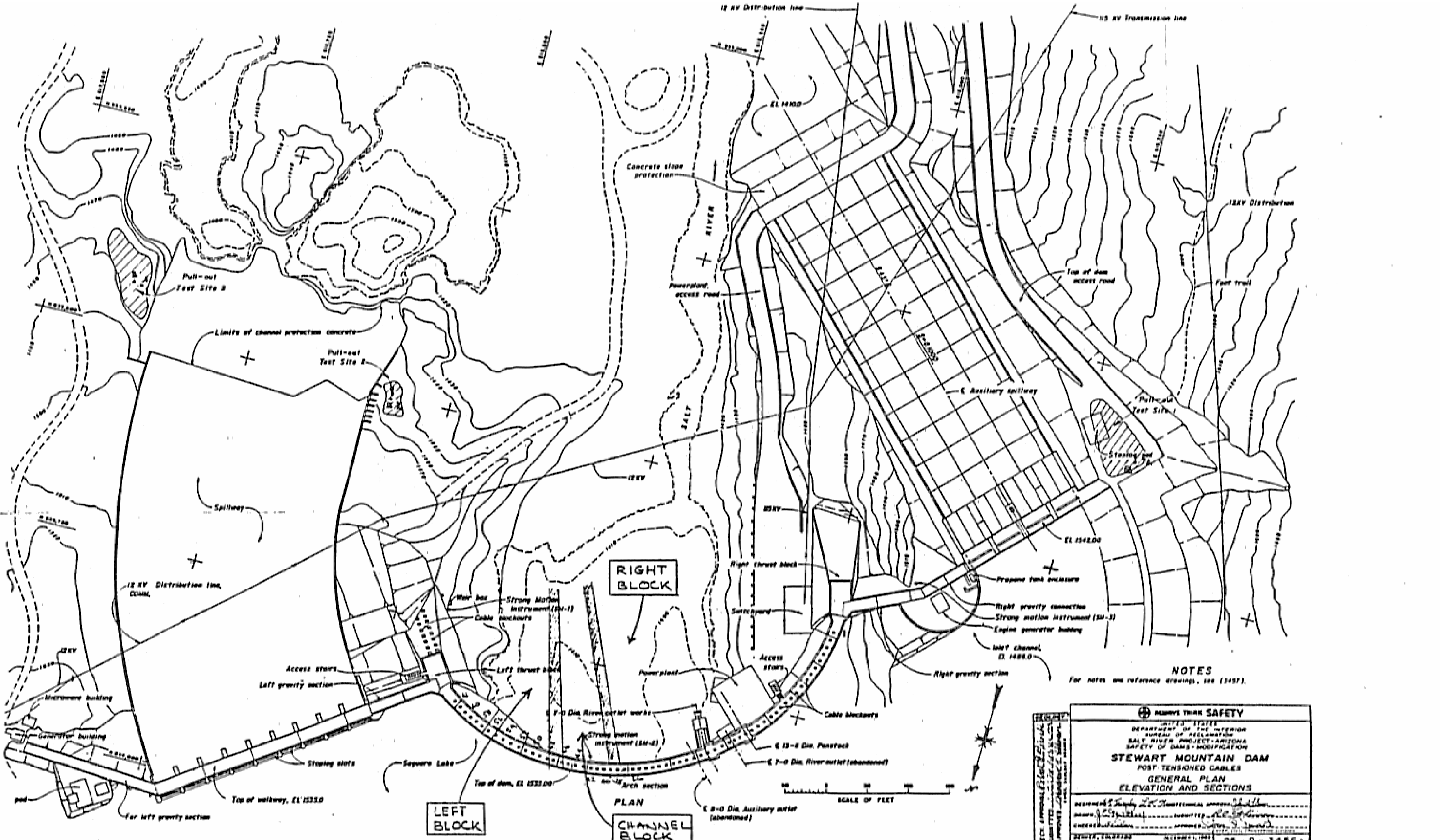


Figure 6 Layout of anchors, Stewart Mountain Dam, AZ.

diameter), suitably spaced and noded in the bond length and tremie grouted with a stable cement grout. Laboratory tests on the grout mix indicated that the grout had an elastic modulus of 2.4 to 2.7x10^6 psi at 28 days, and an unconfined compressive strength of over 6000 psi at the same age. Each tendon incorporated groups of single point extensometers (tell tales) in the bond zone.

Each anchor was then cyclically tested in 25% WL (Working Load) increments to the safe maximum load -or failure. With the exception of Anchor 3A (the shorter anchor in the worst rock, and which underwent grout/rock failure at 968 kips) all anchors achieved the maximum test load of 1310 kips with relative ease.

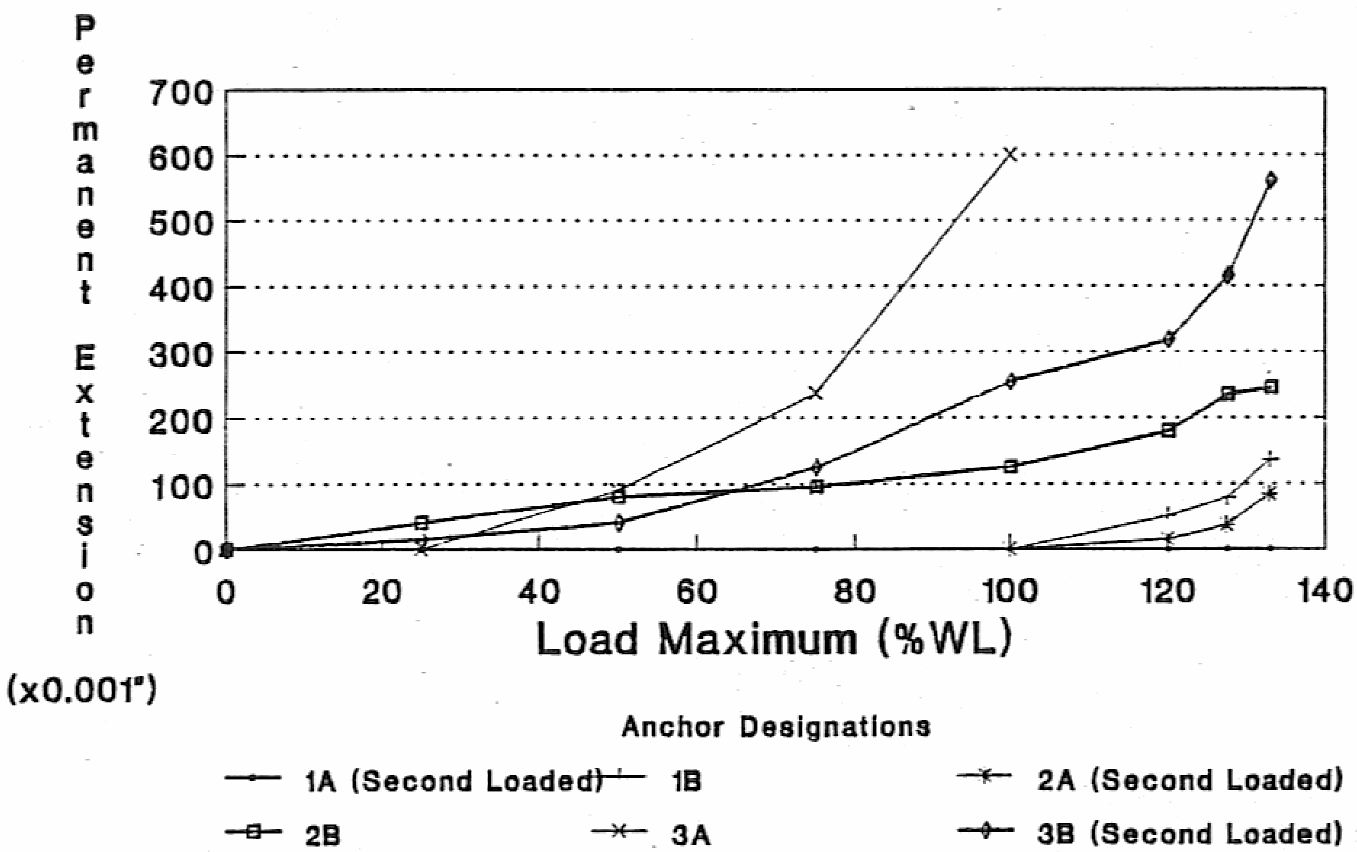
Analysis of the elastic extensions and the "tell tale data" permitted the amount of apparent tendon debonding to be calculated (Table 4). The relative amounts were exactly in line with the quality of the rock mass, especially as reflected in the variation of E value. Basically, therefore, it was proved that the more competent the rock mass (i.e. the lower the E grout:E rock ratio) the less was the extent of apparent debonding, and the higher was the bond stress concentration at the proximal end of the anchor (and hence, the more erroneous the approach of designing on "average" bond values).

Anchor	Apparent Debonding at 133%	
	Actual	Site Average
1A	21"	23"
1B	25"	
2A	40"	42"
2B	44"	
3A 3B	Failed 142" bond 73"	Possibly 108"

Table 4 Calculated apparent tendon debonding lengths, Test Anchors, Stewart Mountain Dam, AZ.

Permanent movements, corrected for measured wedge pull in, are shown in Figure 7. Again they are smallest for Site 1 anchors and greatest in Site 3 anchors, reflecting the overall quality of the rock mass. In addition, the second anchor stressed at each site had smaller permanent movements (as well as less debonding and creep) than the first, strongly indicative of some type of rock mass improvement during the loading of the first anchor. This phenomenon, clearly demonstrated is easy to accept and understand, although to the authors' knowledge, has not been previously documented. This has significance when assessing relative production anchor performance.

Creep was basically not significant in Sites 1 and 2, but it was interesting that although the amount generally increased with load, the highest amounts were at loads of 75-100% WL, and were less at higher loads (<u>Figure 8</u> for example). In addition, whereas 3A showed the classic progressive failure pattern, 3B showed values at 133% (0.057" in 10 minutes) lower than at 100% (0.064" in 10 minutes). When restressed to 133% a second time, the creep was lower still (0.045" in 10 minutes).



<u>Figure 7</u> Net permanent displacements, Test Anchors, Stewart Mountain Dam, AZ.

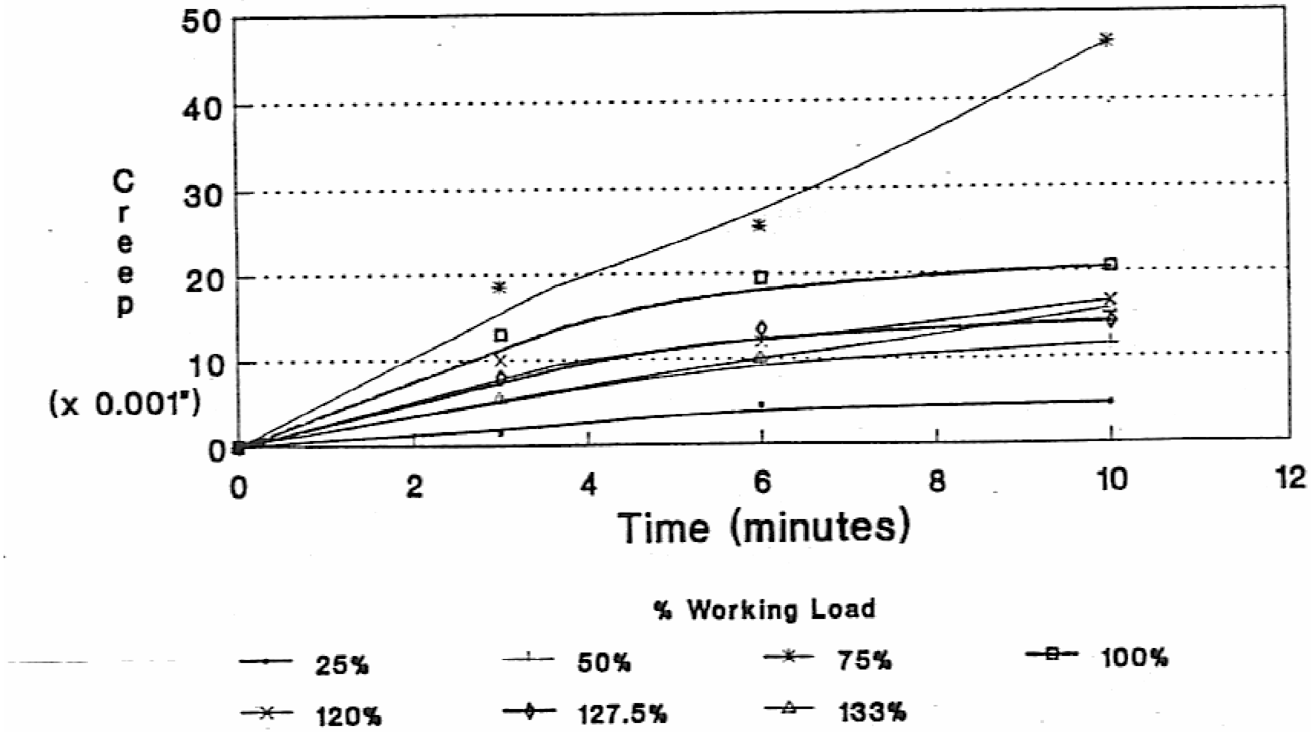


Figure 8 Creep data at each cycle maximum, Test Anchor 2A, Stewart Mountain Dam, AZ.

These data are consistent with the permanent extension phenomena outlined above, and point to an irregular "ratchet" type rock mass response, at odds with the smoother more predictable performance assumed in theory, and usually found in soils. It is proposed that this rock mass improvement was in this case due to a "tightening up" of the fissures and joints in the mass, in the region around and above the bond zone. Crushing of the rock mass, as at Lake Lynn, was not considered feasible given the material strength of the rock.

Overall, the test verified that the originally designed bond lengths had satisfactorily high safety factors in the Type 1 and 2 rock, but merited a slight increase when installed in the poorest quality Type 3 material. The production anchors proceeded accordingly.

4.3. Production Anchors

Prior to drilling the anchor holes, special steel ducts, 5' long, were grouted into cored holes in exact orientation at the anchor entry points. Drilling then proceeded with a diesel hydraulic track rig operating a 10" down-the-hole hammer. Hole bearing and inclination were checked precisely at 10-20' intervals, by a special electronic rate gyroscope/inclinometer tool to ensure correct progress.

During both drilling and stressing, USBR personnel monitored dam movements, joint openings and vibrational effects. These phenomena were absolutely minimal- even in such a delicate thin arch structure: indeed the strains generated by diurnal temperature variances proved markedly larger.

Each hole was then watertested and waterproofed (where necessary) in stages prior to the careful installation of the tendon. Table 5 summarizes the anchor details.

	Arch Section	Thrust Block
# of Anchors	62	22
Free Length	38'-216' (avg.=150')	40'-125' (avg.=82.7')
Bond Length	30'-47' (avg.=36.7')	40'
Strands per Tendon	22	28
Working Load	545-740 (kips)	985 (kips)
Test Load	725-931 (kips)	1150 (kips)
Performance tests	10	2

Table 5 Details of production anchors, Stewart Mountain Dam, AZ.

Stressing progressed very smoothly and analysis of the load extension data confirmed almost perfect elastic performance with minimal debonding and permanent set. Likewise there was negligible creep at Test Load. Vibrating wire load cells have been incorporated in six anchors to monitor long term performance. The dam's performance for a 100 day period after the stressing of all the anchors will be instrumented very closely, prior to final lock off.

5. CONCLUSIONS

These two test anchor programs, and the subsequent observations on the production anchors, have given invaluable information on aspects of load transfer and creep performance in high capacity rock anchors. These are not esoteric researches: they touch upon fundamentals governing the safe design and correct acceptance of such anchors. Throughout the nation, the trend towards rehabilitating concrete dams with anchors will continue to demand more understanding as higher capacities will be sought in rock masses perhaps not always the most competent. This paper is a brief insight into this improved awareness.

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