

THE USE OF POST-TENSIONED ANCHORAGES ON THE
ARCH PORTION OF STEWART MOUNTAIN DAM, ARIZONA

by

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ABSTRACT

Post-tensioned rock anchorages were selected by the Bureau of Reclamation (Reclamation) to stabilize Stewart Mountain Dam against potential seismic loadings. The use of such anchorages in a high, thin arch is thought to be unique. The paper addresses the design, construction and performance of the 62 anchorages installed through the arch section, highlighting special measures undertaken to reduce the risk of damage to the structure during the execution of the work.

INTRODUCTION

Stewart Mountain Dam is located about thirty miles east of Phoenix, Arizona, and was built on the Salt River in the early 1930's. The central portion is a thin arch, 583 feet long and 8 feet wide at its crest. It has a maximum structural height of 212 feet and is up to 34 feet thick at its base. It is flanked by gravity thrust blocks, two spillways (one of which was recently added in a separate safety modification program) and wing dams extending into each abutment (Figures 1 and 2).

At the time of construction, the importance of good cleanup on the horizontal construction joints between each concrete lift was not fully appreciated. These joints were left untreated and concretes of relatively high water content were used to combat the extremely high ambient summer temperatures. As a consequence, the arch section was left with a series of poorly bonded horizontal planes at 5 feet vertical intervals.

Reclamation's three dimensional finite element analysis of the dam's performance during seismic loading conditions indicated separation across various left joints could occur during the maximum credible earthquake of Richter magnitude 6.75 at a distance of 9 miles (Nuss, 1988). Coupled with the loss of confining arch action as a result of an upstream component of movement, the result could be catastrophic failure. After further study, Reclamation chose post-tensioning as the prime method of arch safety modification.

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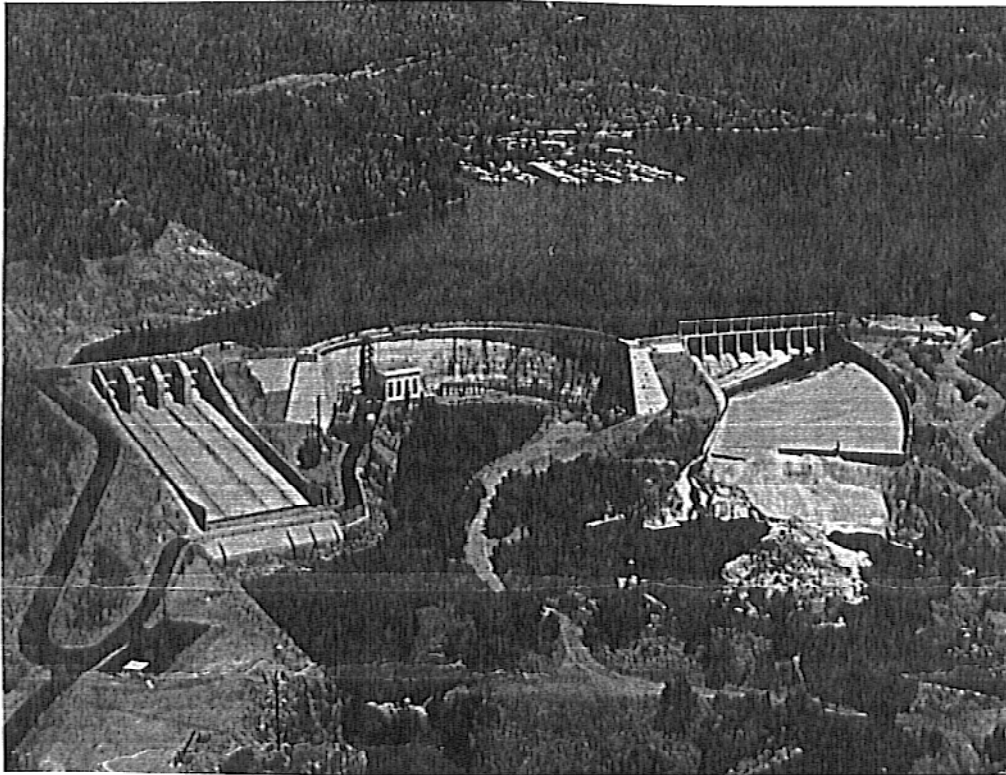


Figure 1. General view of Stewart Mountain Dam, AZ.

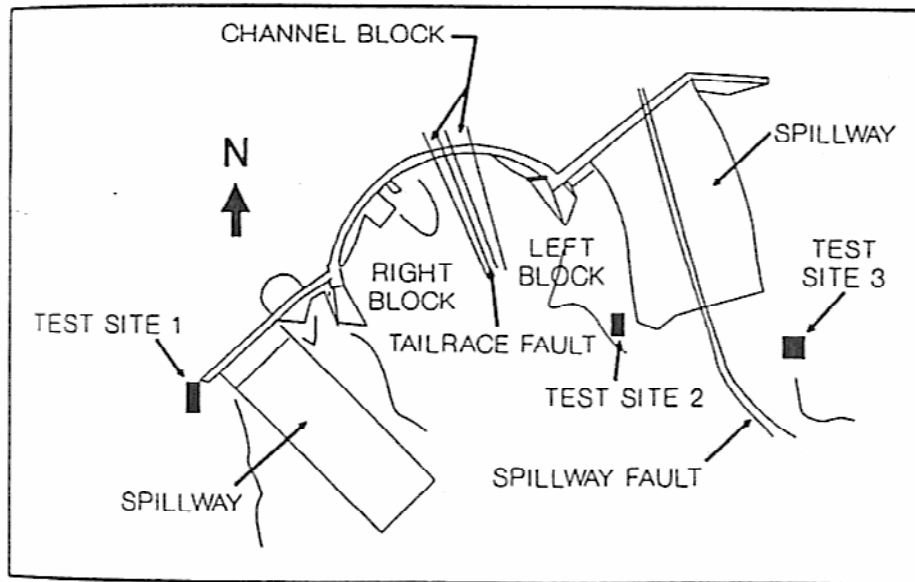


Figure 2. Major features of Stewart Mountain Dam.

Post-tensioned anchorages were also designed to stabilize the contiguous Left Thrust Block, but this work is outside the scope of this paper. General background is provided by Bruce, et al. (1991) and Bruce, et al. (1992).

GEOLOGY

Most of the arch dam foundation consists of hard, Precambrian quartz diorite. The diorite is cut by irregular dikes of mostly hard, medium grained granite, which vary in orientation and thickness. A fault divides the arch dam foundation into three blocks (Figure 2) - the block to the right of the fault (Right Block), the block to the left of the fault (Left Block), and the fault zone itself (Channel Block). Each block has distinct mechanical properties, fracture systems, and permeabilities. The rock underlying the right portion of the dam is mostly hard, slightly weathered to fresh quartz diorite, and generally of excellent quality. The rock to the left of the fault (which includes the Left Thrust Block foundation) was slightly inferior, being more fractured, sheared and weathered quartz diorite with dikes of granite. The fault and the surrounding fractured zone were very intensely fractured and moderately to slightly weathered quartz diorite with a diabase dike.

TEST ANCHOR PROGRAM

Prior to the installation of the production anchors, an intense test anchor program was run and analyzed. The practicality of the foreseen construction methods, and the validity of various design assumptions were examined (Bruce, et al., 1991a; Scott and Bruce, 1992). Pairs of vertical anchorages (Table 1) were installed 12

		SITE 1 (Right Block)	SITE 2 (Left Block)	SITE 3 (Channel Block)
Anchor A	Free Length Bond Length	19 ft 10 in 10 ft 2 in	18 ft 4 in 11 ft 8 in	18 ft 2 in 11 ft 10 in
Anchor B	Free Length Bond Length	10 ft 1 in 19 ft 11 in	18 ft 8 in 21 ft 4 in	27 ft 0 in 13 ft 0 in
Anchors A and B	Strands	28 ea	28 ea	28 ea
Anchors A and B	Max. Test Load (at 80% GUTS)	1310 kips	1310 kips	1310 kips

Table 1. Details of test anchorages as installed.

TEST SITES	HOLE	STRENGTH (MPa) ⁸	DILATOMETER TESTS		EMPIRICALLY DERIVED		SETTLEMENT MODULUS GPa ^{3,4}
			NUMBER OF TESTS	MODULUS (GPa) ⁴	RMR ¹	MODULUS (GPa) ²	
1 (Right Block)	1A	137	1	2.76	59	16.77	n/a ⁷
	1B	222	4	7.58	55	13.34	n/a ⁷
2 (Left Block)	2A	114	0 ⁵	-	49	9.44	n/a ⁷
	2B	143	4	2.07	46	7.94	n/a ⁷
3 (Channel Block)	3A	n/a ⁹	1	0.07	-	n/a ⁶	0.10
	3B	n/a ⁹	0 ⁵	-	-	n/a ⁶	0.17

- ¹ Rock mass classification according to Bieniawski (1984)
² Mass modulus using equation of Serafim and Pereira (1983), $E = 10^{(RMR-10)/40}$
³ Inferred using influence factors from Poulos and Davis (1974)
⁴ Poisson's Ratio assumed to be 0.2
⁵ Obstruction in hole prevented testing in bond zone
⁶ Out of data range used to develop empirical relationship
⁷ Surficial fill at sites 1 and 2 precluded evaluation
⁸ Compressive strength based on average of 5 point load tests (on core)/hole
⁹ Rock core at site 3 was too fractured for point load tests

Table 2. Summary of geological conditions encountered at each test anchorage location.

feet apart in each of the three different structural blocks (Table 2 and Figure 1). In each pair, one anchorage, designated 'A', was intended to have a 10 foot long bond length, while the other, 'B', was to have a 20 foot long bond length. They were cyclically loaded to investigate load holding phenomena including the amount of apparent tendon debonding (Table 3) and creep (Figure 3).

A wealth of fundamental information was obtained, confirming earlier postulations about load transfer mechanisms (Littlejohn and Bruce, 1977). In the context of Stewart Mountain Dam, however, this test program basically verified that the original Reclamation designed bond lengths had satisfactorily high safety factors in the Right and Left Blocks rock but merited a slight lengthening when installed into the poorest quality Channel Block material. The production anchors proceeded accordingly.

LAYOUT OF ANCHORAGES

Sixty-two tendons were designed for installation at approximately 8 foot centers along the crest. Free lengths varied to over 222 feet, while bond lengths ranged from 30 to 46 feet. Inclination varied along the dam, from vertical to 8° 40' upstream (Figure 4). All but 7 of the tendons (located immediately above the river outlet work openings through the dam) were founded into the dam foundation bedrock (32 in the right foundation zone, 15 in the left foundation zone, and 8 in the fault zone). Each tendon consisted of twenty-two 0.6 inch diameter epoxy coated strands. Design working loads (DL) averaged 630 kips (range 550 to 750), equivalent to about 50% GUTS (Guaranteed Ultimate Tensile Strength).

APPARENT DEBONDING AT TEST LOAD		
ANCHORAGE	ACTUAL FOR EACH ANCHORAGE	AVERAGE FOR SITE
1A	21 in.	23 in.
1B	25 in.	
2A	40 in.	42 in.
2B	44 in.	
3A	Failed: 142 in. bond length	Possibly 108 in.
3B	73 in.	

Table 3. Summary of apparent tendon debonding.

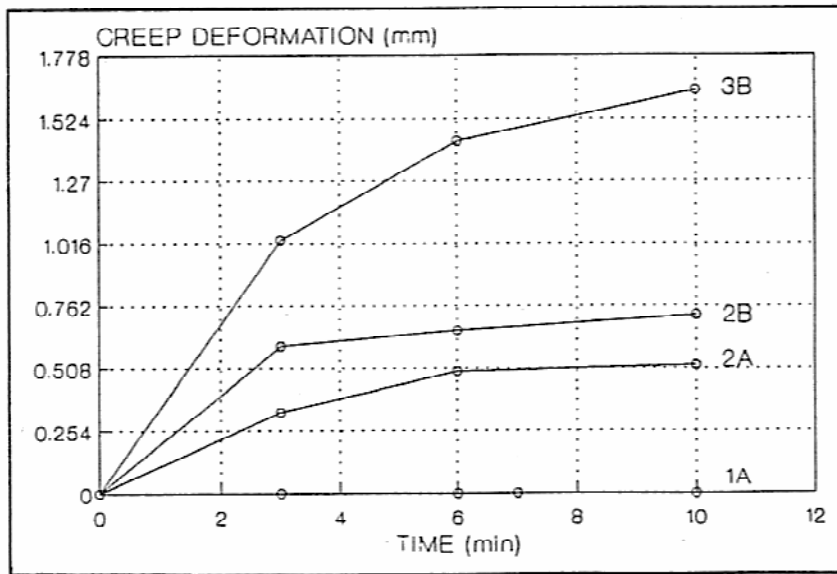


Figure 3. Typical short term creep data for test anchorages at Design Load (DL). NOTE: 1B data not valid; 3A did not reach DL.

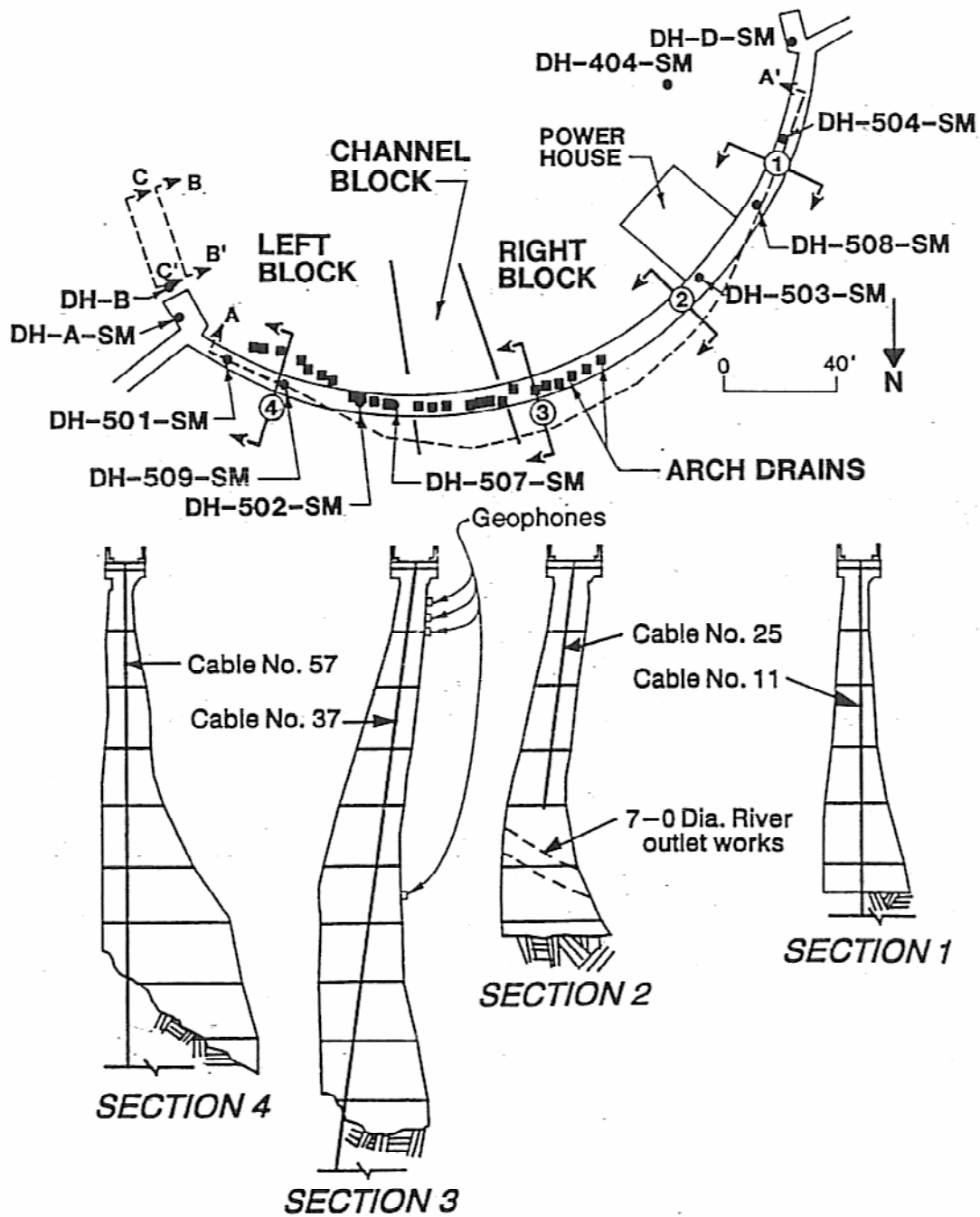


Figure 4. Typical sections through the arch, showing inclination of anchorages, positions of toe drains, and locations of geophones.

CONSTRUCTION

Recesses, 4-1/2 feet square and 2 feet deep had been formed in the dam crest under a previous contract. In these, at the precise location, bearing and inclination, a 12-inch diameter hole was cored about 5 feet deep. A 10-1/2-inch diameter steel guide tube was surveyed and cemented into each hole to ensure that each anchorage hole would commence in the exact, prescribed attitude: angles were measured by independent state-of-the-art methods to within minutes.

The 10-inch holes were then drilled using a down-the-hole hammer, mounted on a diesel hydraulic track-mounted drilling rig. Special hammer and rod attachments were used to promote hole straightness. In accordance with the specifications, the position of each hole was measured at 10-foot intervals in the upper 50 feet of the arch, and at 20-foot intervals thereafter to final depth - a maximum of over 264 feet. The specifications called for a maximum allowable deviation of 1 in 125, and a measurement accuracy of 1 in 400. This tight tolerance was dictated by the thin cross section of the arch section, and the need, for structural reasons, to have the anchorage forces applied at the precise designed locations.

An Eastman Christensen rate gyro inclinometer (Seeker-1) was adapted from its usual oil field duties. This instrument not only allowed the drill bit's position to be measured with the specified accuracy - through the drill rods - but modifications to the associated computer hardware ensured that the acceptability of each hole's path could be demonstrated simply, and virtually in real time. This minimized "down time" in the construction cycle.

As a further check, Reclamation ran random, independent precision optical surveys in completed holes, using a Pentaprism instrument. These surveys invariably showed that even the rate gyro instrument overestimated the amount of deviation (Figure 5) while every hole proved to be within specified tolerances even despite old steel beams encountered in the concrete in certain places (Bianchi and Bruce, 1992).

During the drilling of Hole 37, in a more sensitive part of the arch, close instrumentation of the structure's downstream face was conducted. Crack meters placed across the lift joints indicated tiny movements at most (Table 4) while the geophones indicated equally insignificant peak particle velocities (Figure 6). During every drilling (and grouting) activity, the dam's underdrain system was flushed with water and monitored closely to ensure that the drains were not compromised. As shown in Figure 7, the short term impact of the drilling and water testing activities on piezometric levels was dissipated soon after these were completed. Various other structural, environmental and personnel safety issues were addressed, as described by Bruce and Triplett (1992).

Each hole was then water pressure tested in ascending stages from 50 to 130 feet long. The acceptance criteria were 0.001 gallon/ft/inch diameter/minute at an excess pressure of 5 psi (in the bond length), and twice that in the free length. Pregrouting was also specified if interhole connections between adjacent holes occurred. Most stages met these criteria, but others had to be pregrouted and redrilled (as many as four times) before the specified degree of impermeability could be achieved.

The tendons were assembled off-site under factory controlled conditions, and transported in coils with great care. At site each

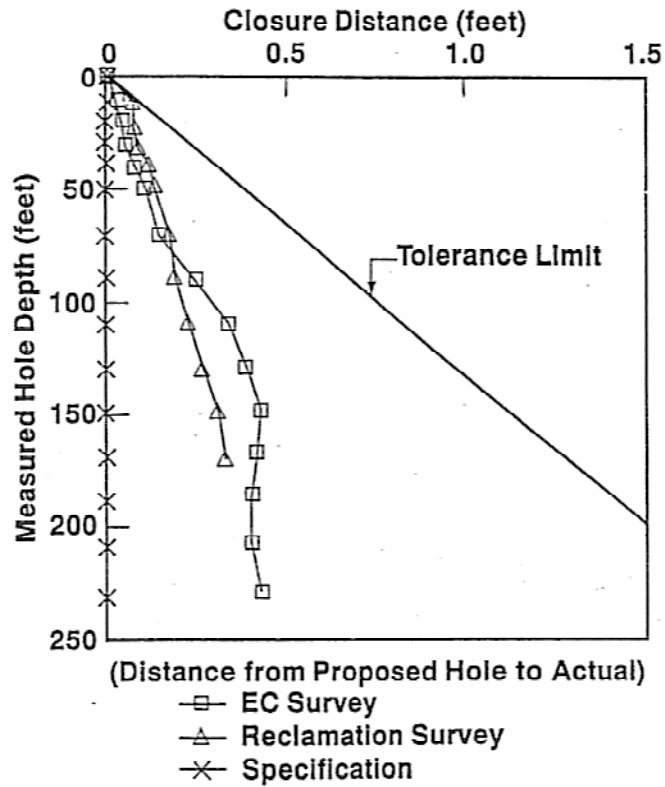


Figure 5. Typical hole deviation data, as monitored in Hole 37.

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 4. Movements recorded across horizontal lift joints during down-the-hole drilling. Hole 37, Stewart Mountain Dam, AZ

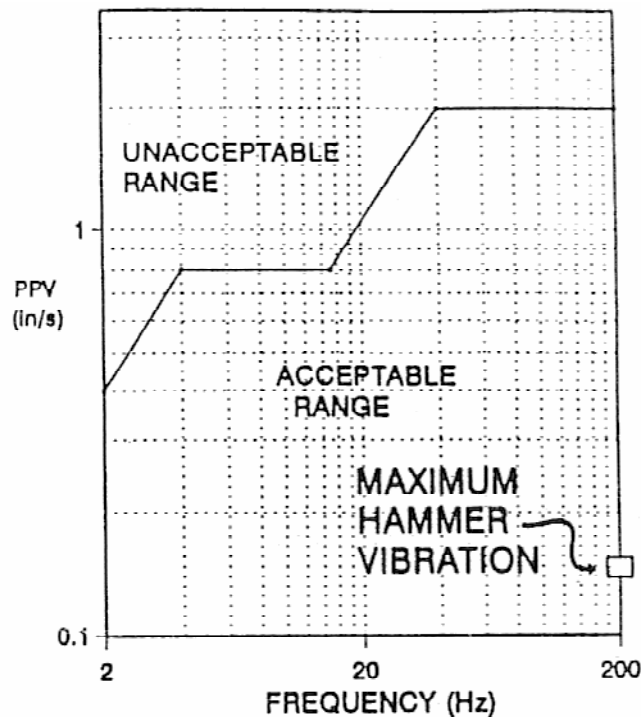


Figure 6. Data from geophone monitoring during down-the-hole drilling through concrete. Stewart Mountain Dam, AZ (Reclamation Acceptability Criterion).

coil was then placed on a mechanical uncoiler which gently lowered the tendon into the hole. During insertion, any "windows" in the epoxy coating were repaired with a quickset, patching epoxy. Spacers and centralizers were placed at regular intervals in both the bond and free lengths.

Specially researched low water content, plasticized grouts were prepared in a colloidal mixer, and pumped via a Moyno pump to each tendon's primary tremie tube. Close control over grout volumes were exercised to provide the exact bond length. Fluid and set grout properties were rigorously recorded as routine quality control and assurance during construction.

For these arch anchors alone, the following quantities were recorded:

Rock drilling	3730 feet
Concrete drilling	8143 feet
Water tests	252 each
Redrilling	3615 feet

Stressing and Performance

Stressing commenced 14 days after grouting. Nine anchorages were subjected to cyclic performance tests as per Post Tensioning Institute (PTI) (1986) recommendations to verify in detail the correct operation of the production units. The other anchorages were tested more simply, as per the PTI Proof Test provisions, but modified to include a cycle to AL (Alignment Load), after the Test Load (133% Working Load - WL) had been sustained, before locking off

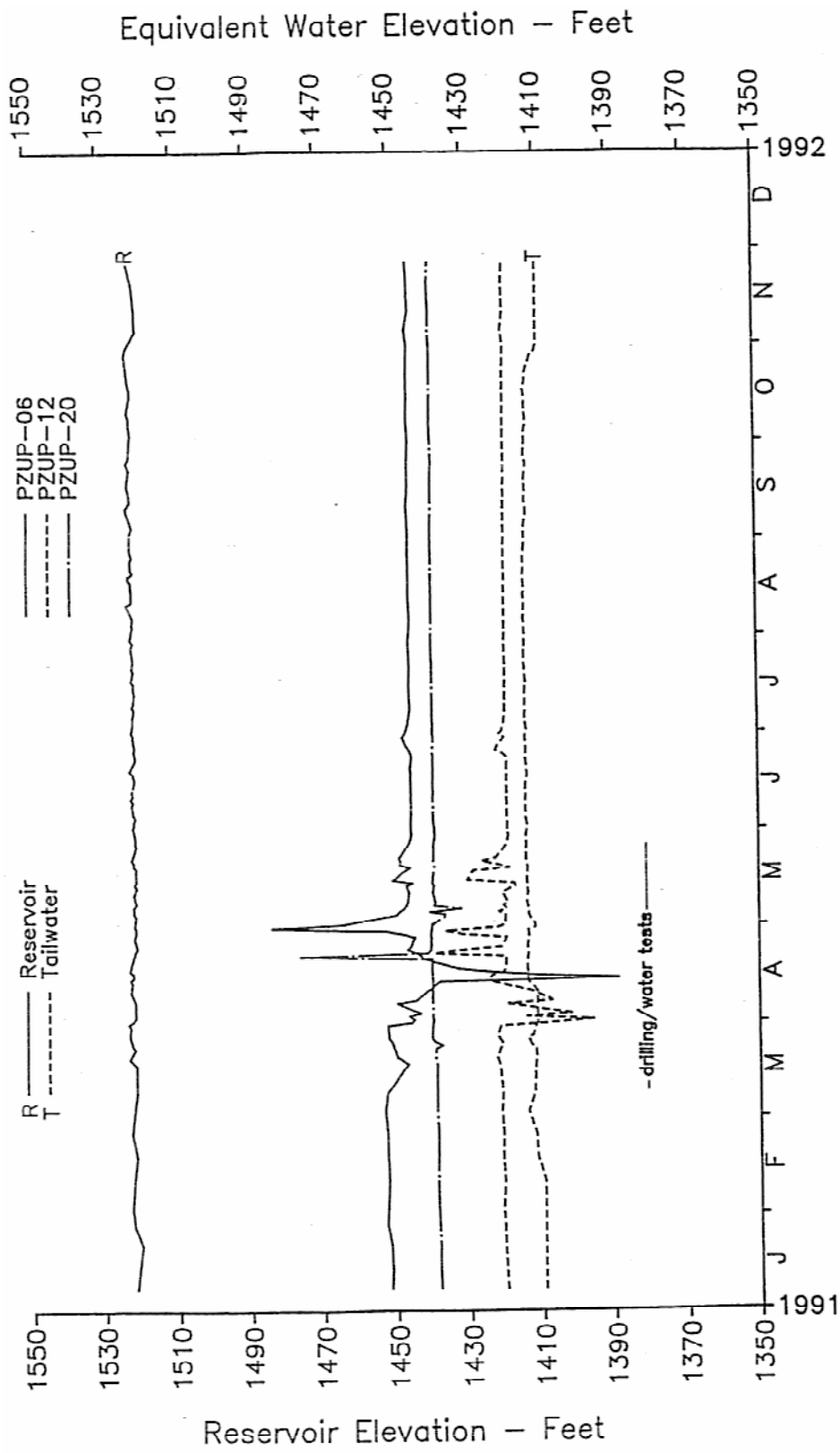


Figure 7. Arch drain piezometer records during anchorage construction.

at the interim lock-off load (117% WL). Alignment Load (5% of WL) was set with monojacks to ensure equal loading of the strands during the subsequent multijack operations.

Net elastic extensions as long as 16 inches were recorded on the longest tendons at Test Load, while no permanent extension greater than 0.9 inch was measured.

Precision monitoring of the dam during stressing confirmed that no structural deflections were imparted by the post-tensioning. This was probably helped by the Reclamation's idea of trying to minimize any loading impact by building up the load gradually in each block of the arch: Anchor 60 followed by Anchor 58, by Anchor 6, 4, 13, 11 and so on.

The structure and four anchorages with vibrating wire load cells were then monitored for a further 100-day period prior to the anchorages being finally locked off at a minimum of 108.5% Working Load. No structural movement was measured, and lift-off data confirmed that each anchorage had performed exactly as predicted, in that period. The free length of each tendon was then tremie grouted both to provide a further layer of corrosion protection, and to structurally bond the stressed tendon to the surrounding concrete.

The work concluded with the infilling of the recesses with non-shrink concrete.

CONTRACTUAL ASPECTS

This contract was let using relatively innovative procurement procedures. Far in advance of bidding, Reclamation obtained information from specialists in all facets of the industry. As a consequence, the specifications, though by necessity very rigorous, were both eminently practical and up to date. The decision to invite separate technical and price proposals - independently assessed - ensured that not only was the best qualified contractor chosen, but also that he was motivated to contribute "heart and soul" to every stage of the project's execution. As a consequence, the work was carried out virtually as an engineering joint venture, at site and head office levels, between equally committed parties. The project was completed within program and under budget without contractual dispute or litigation.

FINAL REMARKS

This case history illustrates many innovative technical features: the precision drilling and its monitoring, the use of epoxy coated strand, and the close study of the structure during the various construction activities. However, this project's main value for many will be its clear illustration of how successful thoughtful, alternative contract procurement procedures can work to the benefit of all the contracted parties. Although not officially a "Partnering" project, the works at Stewart Mountain Dam benefitted at every turn from professional partnerships developed at all levels of responsibility and participation.

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KEY WORDS

Rock anchorages
Dam rehabilitation
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Post tensioning
Test anchorages
Precision drilling
Permeability testing
Grout