

Anchoring Stewart Mountain Dam  
Against Seismic Loadings

DR. DONALD A. BRUCE  
*Nicholson Construction of America, Bridgeville, PA*

LARRY K. NUSS  
*U.S. Bureau of Reclamation, Denver, CO*

**ABSTRACT**

Stewart Mountain Dam, on the Salt River near Phoenix, Arizona, is a double curvature thin arch structure constructed over 60 years ago. At its highest point, it stands 60 m above the river bed, while the 180 m long arch crest is flanked by gravity buttresses and two spillways. Analyses had indicated a potential problem - linked to poor lift joint status - in the arch during a nearby seismic event. High capacity post tensioned rock anchors, up to 82 m long were installed through the arch to restore monolithicism and increase factors of safety for both seismic and normal loading conditions. A further 22 tendons were installed through the Left Thrust Block to guard against sliding. This project was unique in several aspects and this paper outlines the major lessons learned.

**INTRODUCTION**

Post-tensioning a thin arch concrete dam with rock anchors, to improve its ability to withstand the design earthquake is not a traditional safety of dams modification. However, just such a modification has recently been completed at Stewart Mountain Dam on the Salt River, near Phoenix, Arizona. While many concrete gravity dams have been post-tensioned to improve their stability (Bruce, 1989; Bruce and Clark, 1989; Bragg et al, 1990), this is believed to be the first time a multicurvature concrete arch dam has been repaired in this way. The southwest desert environment is not typically thought of as seismically active, yet major earthquakes have occurred there in historic times: the failure of Stewart Mountain Dam, just 30 miles upstream of Phoenix would have had catastrophic consequences for the metropolitan area.

Beyond this unique application of post-tensioned anchors, there were many other "firsts" on this project. An extensive test anchor program was conducted to establish bond lengths and load transfer mechanisms in each of the three major foundation zones; extremely tight drilling tolerances and frequent downhole surveys were required in each hole; epoxy coated strand was used to provide primary corrosion protection for the tendons; and the response of this potentially delicate structure was meticulously monitored during every phase of construction.

Designs and construction management for the rock anchor tendons were performed by the Bureau of Reclamation under their Safety of Dams Program. Nicholson Construction was awarded the contract in November 1990 through

a negotiated procurement. They were chosen as the contractor providing the best combination of technical and cost proposals: again a relatively novel feature in such works and a major factor in the subsequent success of the operation.

## DESIGN

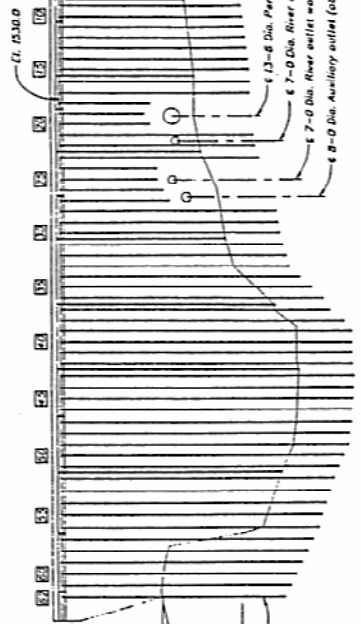
Stewart Mountain Dam was constructed from 1928 to 1930. As the importance of good cleanup on the horizontal construction joints was not recognized at the time, the joints were left untreated. This resulted in a layer of laitance on the horizontal joints, which later compromised bond across them. A three-dimensional finite element model of the dam was used to evaluate the dam's performance during various loading conditions, including seismic loads generated by the Maximum Credible Earthquake of Richter magnitude 6.75, occurring 15 km from the dam (Nuss, 1988) and so generating an estimated site acceleration of 0.34 g. The analysis indicated that the dam would lose arch action during such an event, leaving vertical cantilever sections to support themselves. Because of the lack of bond at the horizontal lift lines, the blocks in the upper portion of the dam would be free to displace under these conditions. Sixty-two tendons were thus designed to stabilize the arch, each at about 2.7 m centers, with free lengths ranging up to 66 m, and bond lengths ranging from 9 to 14 m. Their inclination varied from vertical to 8° 40'. All but 7 of the tendons (all located immediately above the outlet works openings through the dam) were anchored into the dam foundation bedrock (Figure 1). The arch

tendons each comprised twenty-two 15.2 mm diameter epoxy coated strands. Design working loads averaged 285 tonnes (range 250-340) per tendon, equivalent to about 50% GUTS.

In addition to the arch tendons, 22 tendons were designed for the Left Thrust Block of the dam to stabilize this portion of the structure against a potential failure plane at or just below the structure/foundation contact (Figure 2). The thrust block tendons varied in length from 12 to 38 m (free length) plus 12 m bond length, and comprised 28 strands. Design load for each tendon was 450 tonnes (60% GUTS).

## GEOLOGY

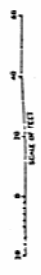
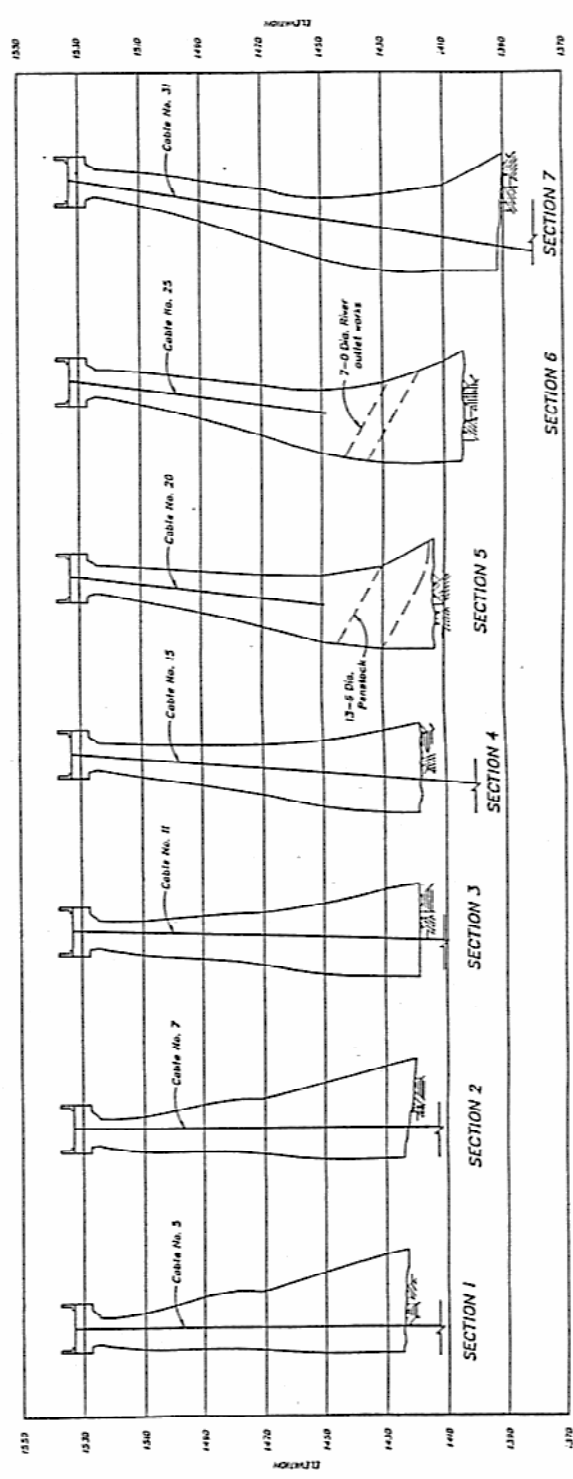
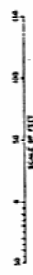
Most of the arch dam foundation consisted of hard, pre-Cambrian quartz diorite. The diorite was cut by irregular dikes of hard, medium grained granite, which varied in orientation and thickness. A fault divided the arch dam foundation into three zones - the zone to the right of the fault, the zone to the left of the fault, and the fault zone itself. Each zone had distinct mechanical properties, joint systems, and permeabilities. The rock underlying the right portion of the dam was hard, slightly weathered to fresh and generally of excellent qualities. The rock to the left of the fault (which included the Left Thrust Block foundation) was slightly inferior, being more fractured, sheared and weathered. The fault and the surrounding fractured zone were very intensely fractured and moderately to slightly weathered. During the design phase, it was assumed that 32 of the arch tendons would be anchored in the



**ELEVATION**

DEVELOPED ALONG E OF ARCH ROADWAY

**KEY PLAN**



**NOTES**

Bottom of free length shall be established 10 feet below the foundation contact, except for cables 37-42. Free length shall be established by the contractor and adjustments to the free length dimensions and the theoretical cable elongations may be required once the actual foundation contact is established.

□ Cable location at typical arch section.

○ Cable numbers correspond to cable blockout numbers. For location and details of cable blockout, see (2896). Sections are representative of the form in a particular location and radius center points in the Key Plan are approximate because of limited construction records and detail - appropriate reaction expansions in the existing concrete.

Additional strand spacers with centerlines may be required at the top of the hole to maintain the spacing between strands and the wall of the hole below for 50 feet of hole.

**REFERENCE DRAWINGS**

- DAU: MODIFIED TOP OF DAM-ARCH SECTION - - - - - 25-D-2888
- MODIFIED ARCH SECTION-CABLE ADJ - - - - - 25-D-2882
- MODIFIED TOP OF DAM-ARCH SECTION - - - - - 25-D-3341

**ALWAYS THINK SAFETY**

PROPERTY OF THE UNITED STATES GOVERNMENT

SALT RIVER PROJECT-ARIZONA

SAFETY OF DAMS-MODIFICATION

**STEWART MOUNTAIN DAM**

POST-TENSIONED CABLES

**ARCH SECTION**

DESIGNED BY: *[Signature]* ENGINEER, CIVIL

CHECKED BY: *[Signature]* ENGINEER, CIVIL

DATE: *[Signature]*

PROJECT NO. 25-D-3458

**Figure 1. General arrangement of tendons, Arch Section.**

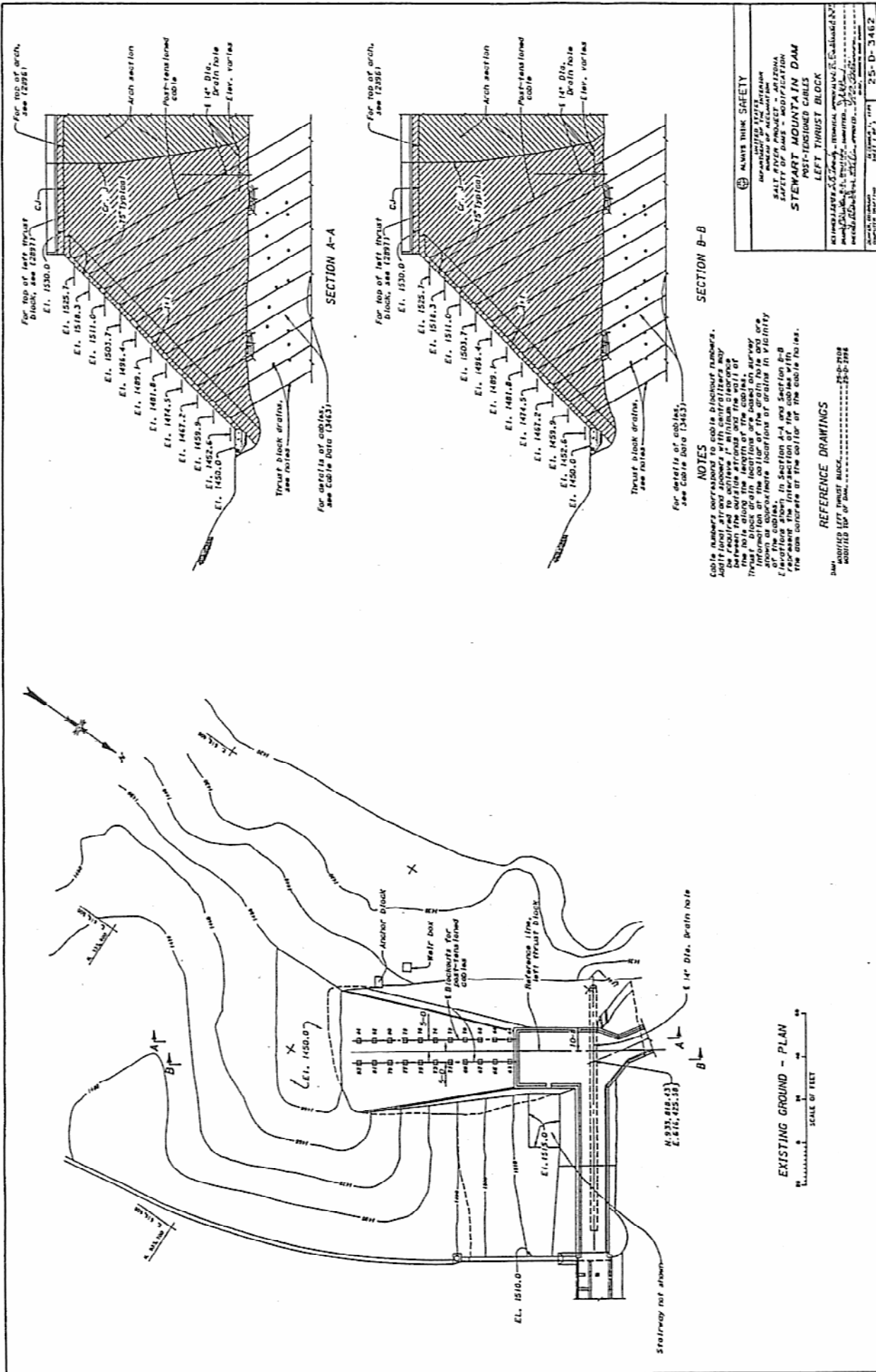


Figure 2. General arrangement of tendons, Left Thrust Block.

right foundation zone (with 7 of the tendons in this area anchored in the dam concrete); 15 of the arch tendons would be founded in the left foundation zone; and 8 of the arch tendons would be founded in the fault zone. All 22 of the Thrust Block tendons were founded in the left foundation zone.

#### TEST ANCHOR PROGRAM

A pair of vertical anchors (A and B) were installed 3.6 m apart in each of three test sites, representative of the three major rock zones expected to underlie the dam. Details are summarized in Table 1.

In all geotechnical respects, 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 180 Mpa in Site 1, and 130 Mpa in Site 2, while only small fresh samples of similar strength could be tested from Site 3 cores. Rock mass E values

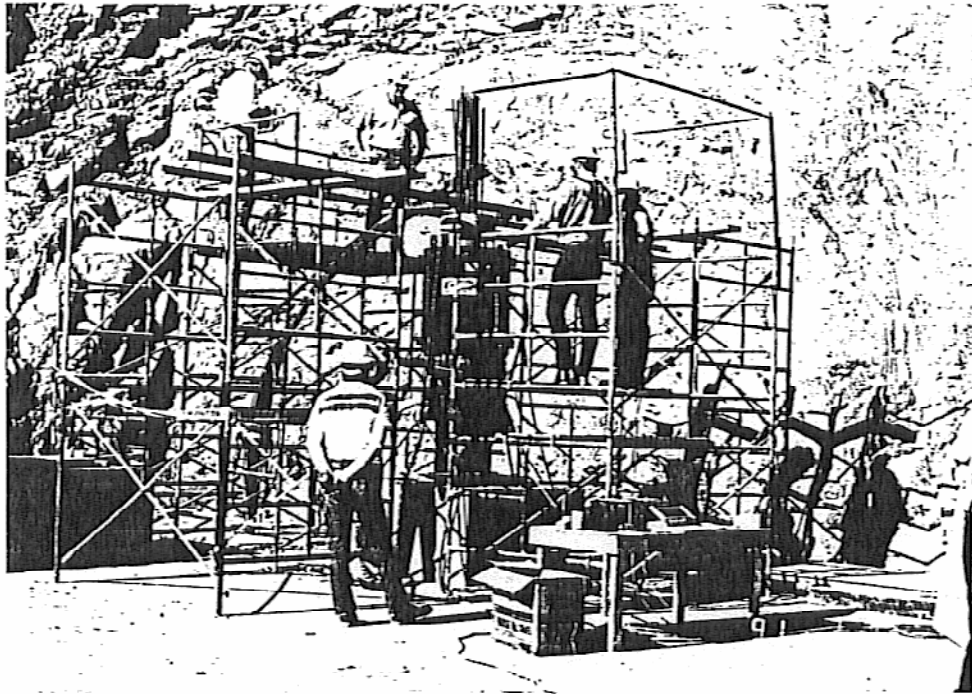
ranged from perhaps 7000 to 20,000 Mpa (Site 1) to 3500 to 17,000 Mpa (Site 2) to probably around 700 Mpa in Site 3.

Each anchor hole was first cored to NX diameter and subjected to a multipressure water test, and dilatometer testing (to estimate in situ E value), prior to being redrilled to full 254 mm diameter with a down-the-hole hammer. Tendons consisted of the special epoxy coated strands (each 15.2 mm 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 about 17,000 Mpa at 28 days, and an unconfined compressive strength of over 40 Mpa 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 test load - or failure (Photograph 1). With the exception of Anchor 3A (the shorter anchor in the

		SITE 1	SITE 2	SITE 3
Anchor A	Free Length	6.1 m	5.6 m	5.5 m
	Bond Length	3.1 m	3.6 m	3.6 m
Anchor B	Free Length	6.1 m	5.7 m	8.2 m
	Bond Length	6.1 m	6.5 m	4.0 m
Anchor A/B	Strands	28	28	28
Anchor A/B	Max. Test Load (at 80% GUTS)	595 tonnes	595 tonnes	595 tonnes

Table 1. Details of Test Anchors, Stewart Mountain Dam, AZ



Photograph 1. Stressing Test Anchor 1B.

worst rock, and which underwent grout/rock failure at 440 tonnes) all anchors achieved the maximum test load of 595 tonnes with relative ease.

Analysis of the elastic extensions and the "tell tale" data permitted the amount of apparent tendon debonding to be calculated (Table 2). 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 conventional approach of designing on "average" bond values). This was exactly in line with the results of mathematical and laboratory tests first collated by Littlejohn and Bruce (1977).

APPARENT DEBONDING AT TEST LOAD		
ANCHOR	Actual	Test Site Average
1A	533 mm	} 584 mm
1B	635 mm	
2A	1016 mm	} 1067 mm
2B	1118 mm	
3A	Failed: (3607 mm of bond)	Possibly 2743 mm
3B	1854 mm	

Table 2. Calculated apparent tendon debonding lengths, Test Anchors, Stewart Mountain Dam, AZ

Permanent movements were smallest for Site 1 anchors and greatest in Site 3 anchors, reflecting again 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.

Creep was basically not significant in Sites 1 and 2, but it was interesting that although the amount generally increased with load, the highest values were at loads of 75-100% WL, and were less at higher loads. In addition, whereas 3A showed the classic progressive failure pattern, 3B showed values at 133% (1.45 mm in 10 minutes) lower than at 100% (1.63 mm in 10 minutes). When restressed to 133% a second time, the creep was lower still (1.14 mm in 10 minutes).

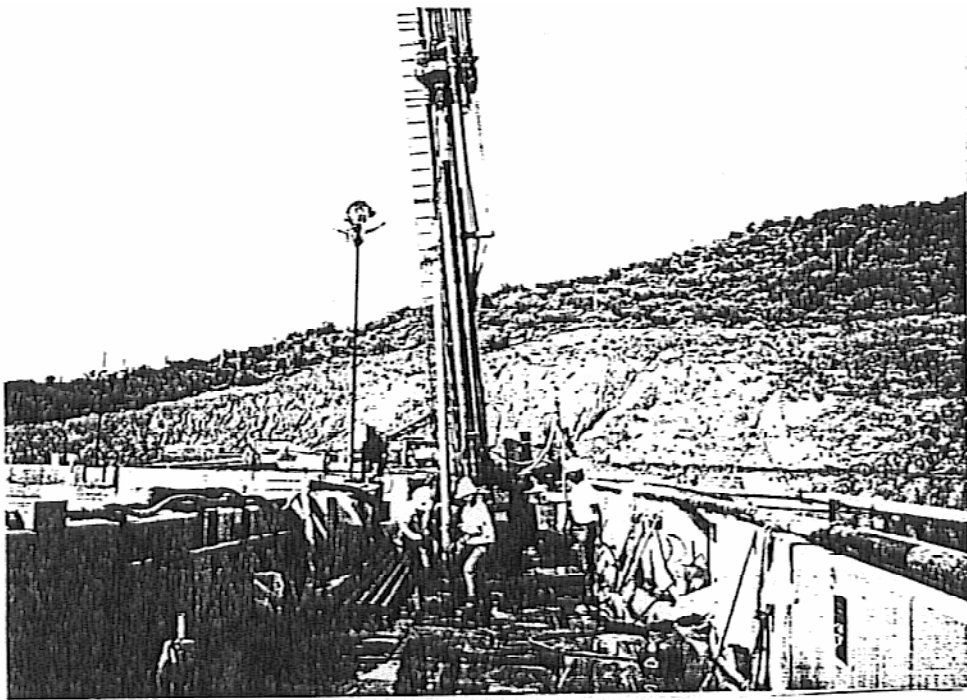
These data are consistent with the permanent extension phenomena outlined above, and point to an irregular "ratchet" type rock mass response, at odds with the smoother more predictable performance assumed in theory, and usually found in soils. It is proposed that this rock mass improvement was in this 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 itself was not considered feasible given its high material strength.

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

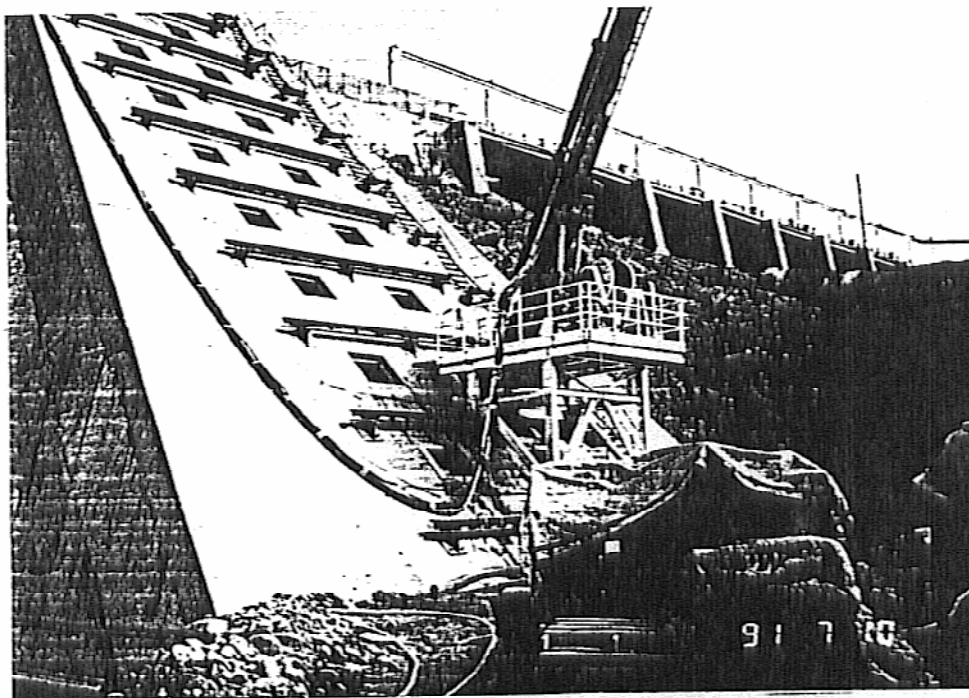
## PRODUCTION ANCHORS

1.45 m square recesses, 0.6 m deep, had been formed in the dam crest under a previous contract. At the precise location, bearing and inclination, a 300 mm diameter hole was cored about 1.52 deep at each anchor entry position. A 250 mm diameter steel guide tube was then surveyed and cemented into this hole to thereafter ensure the anchor hole drilling rig would have the exact prescribed starting orientation: angles were measured by independent state of the art methods to within minutes of accuracy.

The 254 mm diameter anchor holes were then drilled using a down-the-hole hammer, mounted on a new Nicholson Casagrande C12 long stroke, diesel hydraulic track rig (Photograph 2). Special hammer and rod attachments were used to promote hole straightness. In accordance with the specifications, the position of the hole was measured at 3 m intervals in the upper 15 m of each hole, and thereafter at 6 m intervals to final depth: a maximum of 82 m. This high frequency of measurement - and the precision required - to within 75 mm in 30 m - demanded very special attention. Nicholson worked with Eastman Christensen from Bakersfield, California, to adapt their Seeker 1 rate gyro inclinometer from its usual oil field duties. The Seeker's suitability was proved during the test anchor program and in parallel task specific tests. This instrument not only allowed the bit's position to be accurately measured through the drill rods, but modifications of the computer software ensured that the acceptability of the hole's progress could be demonstrated within minutes - at the rig, to minimize "down time" in the construction cycle.



Photograph 2. Drilling on the Arch Section.



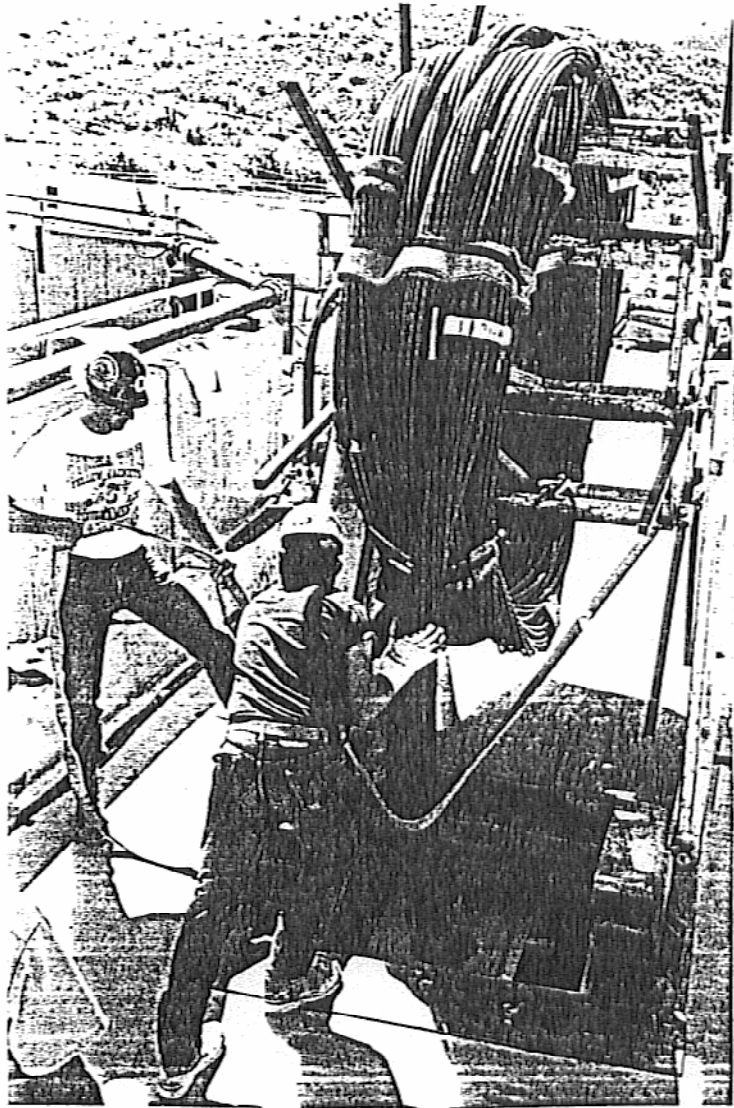
Photograph 3. Drilling with the frame mounted mast.



As a further check, USBR personnel ran independent precision optical surveys using a Pentaprism instrument, on randomly selected holes: these confirmed the immaculate straightness of the holes, and their correct bearing and inclination. Every hole proved acceptable.

Another series of tests was run during the early drilling operations. Geophones and crack meters were fixed at the downstream face of the dam adjacent to the drill hole. These proved that the maximum fissure apertures and vibrations induced by drilling were incredibly tiny: barely of the order of those induced by natural temperature fluctuations. This observation is of major significance for dam engineers: even for a "delicate" structure, the drilling of a hole by rotary percussion within 1.5 m of a free face was hardly noticed by the dam. This drilling method is extremely cost effective and so ensures the viability of anchors as an economic solution for all manner of dam stabilization problems.

For the Thrust Block holes, a massive frame was erected up the face of that structure. This carried platforms to which was affixed the Casagrande drill mast (Photograph 3). Again, special precautions were taken to ensure hole correctness and direction. Every hole was water pressure tested, and pregrouted and re-drilled if necessary - prior to a final directional acceptance survey. Most test stages - which ranged in rock and concrete from 15 m to 40 m proved tight, but other stages needed as many as three pre-treatments to allow the specification to be met - 0.2 litres per minute per lineal metre of hole at 0.03 Mpa excess pressure.



Photograph 4. Placing the epoxy coated strand tendons in the Arch.

The special epoxy coated strand tendons, assembled off site by DSI Inc., were placed in reels on special uncoilers and transported to the holes. Using extreme care to prevent abrasion of the epoxy coating, each tendon was slowly placed to full depth (Photograph 4). A specially researched high strength, plasticized grout was then tremied into each hole to provide the exact bond length. Fluid and set grout properties were rigorously recorded as routine quality control throughout construction.

## QUANTITIES

Overall, the following quantities were recorded:

	<u>Dam Crest</u>	<u>Thrust Block</u>
Rock Drilling	1137 m	462 m
Concrete "	2482 m	407 m
Water Tests	252 ea	90 ea
Redrilling	1102 m	494 m

Stressing commenced 14 days after grouting. Twelve tendons were subjected to cyclic Performance Tests, as per PTI Recommendations (1986) to verify in detail the correct operation of these tendons. The other anchors were tested more simply, as per the PTI Proof Test provisions. Given the high loads, and long free lengths, extensions as long as 440 mm were recorded at Test Load on the longest tendons (permanent extension of 9 mm). Creep and lift-off checks rounded out the initial verification of the anchors: in all aspects, every anchor proved to have outstanding qualities, with details closely mirroring the conclusions of the test program.

Each anchor was proved to 133% of design working load and destressed to alignment load, prior to interim lock-off at 117%. Monitoring of the dam during stressing confirmed no structural deflections as a result of the imposition of this extra load. This was probably helped by the USBR's idea of trying to minimize any loading impact by building up the load gradually in each block of the dam: Anchor 60 was followed by Anchor 58, by Anchor 6, 4, 13, 11, and so on. The structure and the anchors were then monitored for a further 100 days after stressing before final lock-off (at a minimum of 108.5% of design

working load) and secondary grouting. Again the anchors were proved to have performed well, while no discernible movements were induced in the arch of the dam.

## LESSONS

The strengthening of Stewart Mountain Dam is, in itself, a major case history which will prove of interest to practitioners worldwide. However, there are several features which render it unique, and promise to make it one of the key dam rehabilitation projects of the decade:

- Application: high capacity anchors for a delicate double curvature thin arch dam to resist seismic effects.
- Research and Development: the intensive test program permitted confirmation of many of the intricate theories of load transfer in hard rock anchors - and, surprisingly - a clear reminder that even hard rock masses can be altered by prestressing (Bruce, et al., 1991).
- Drilling Technology: using appropriate planning, tooling, equipment and expertise, 254 mm diameter holes can be drilled fast and extremely straight and accurately through concrete and hard rock to depths of over 80 m. Such methods appear to have absolutely no deleterious effects on the structure. And more - systems now exist to pinpoint this accuracy to within millimeters at such depth.
- Tendon Technology: the relatively new product of epoxy coated strand appears workable in the field, and seems to give excellent

bonding characteristics. Strong controls must be exercised over handling and installing such tendons to prevent abrasive damage to the epoxy coating.

● Anchor/Structure Interaction: if Stewart Mountain is typical of the current quality of such dams, then we can conclude that the application of tens of thousands of tons of prestress causes no structural distress to double curvature thin arches.

Despite these technological conclusions, we believe that one of the great lessons of Stewart Mountain Dam was the procurement and contracting procedure. Far in advance of bidding, the USBR interviewed, unofficially, a wide range of specialists in all facets of the industry. As a consequence, the specifications, though

by necessity very rigorous, were both eminently practical and right 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 a hint of contractual dispute or litigation. The bottom line: a technically superb job, perhaps a little profit for the contractor, but certainly a great deal of fun - and pride - for all the individuals involved!

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