Stewart Mountain Dam Stabilization

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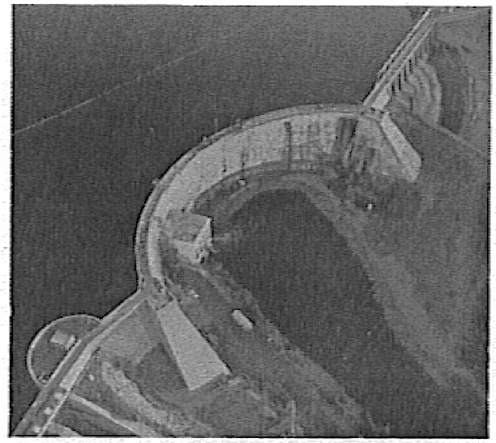
Introduction

Post-tensioning a thin arch concrete dam to improve its ability to withstand the design earthquake is not a traditional safety of dams modification. However, just such a modification was recently 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 (References 1, 2, 3), this is believed to be the first time a multi-curvature concrete arch dam has been post-tensioned. And while the



Aerial view – Stewart Mountain Dam.

southwest desert environment is not typically thought of as seismically active, major earthquakes have occurred there in historic times and the failure of Stewart Mountain Dam, just 30 miles upstream of Phoenix would have catastrophic consequences for that metropolitan area. Beyond this unique application of post-tensioned anchors, there were many other "firsts" on this project. An extensive pullout test 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; epoxy coated strand was used to provide primary corrosion protection for the tendons; and the behavior of this potentially delicate structure was meticulously monitored during every phase of construction. Designs and construction management for the post-tensioned tendons were performed by the Bureau of Reclamation under their Safety of Dams Program. Nicholson Construction Inc. 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

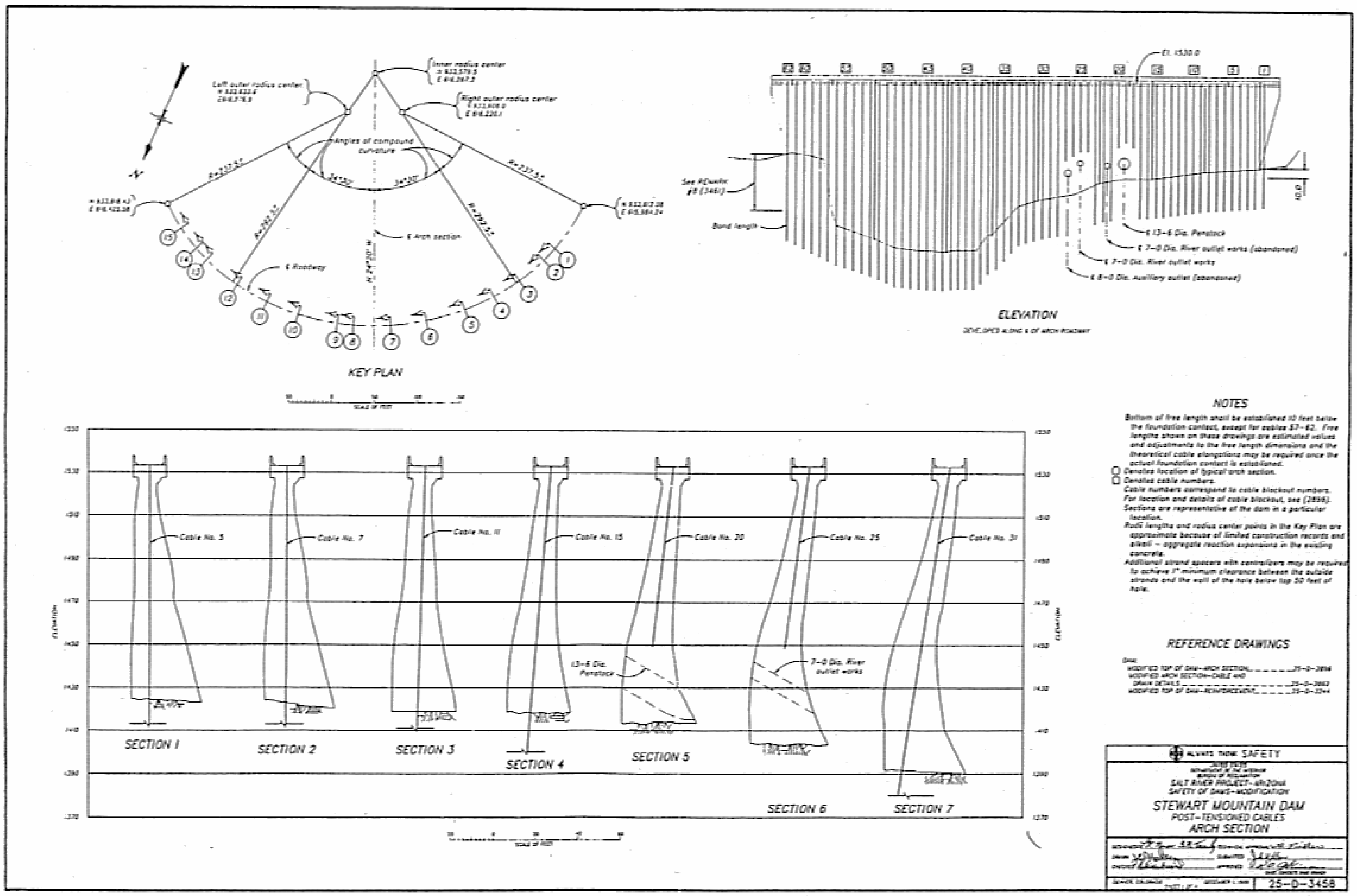


Figure 1. Stewart Mountain Dam – Post-tensioned cables, arch section.

in the subsequent success of the operation.

Design

Stewart Mountain Dam was constructed from 1928-1930. 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 (Reference 4). 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 liftlines, 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 nine feet centers, with free lengths ranging up to 216 feet, and bond lengths ranging from 30 to 45 feet. Their inclination varied from vertical to 8 by 40 feet. All but seven of the tendons (all located immediately above the outlet works openings through the dam) were anchored in the dam foundation (Figure 1). The arch tendons each comprised twenty-two 0.6 inch-diameter epoxy coated strands. Design working loads averaged about 625 kips (range 545-740) 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 40 to 125 feet (free length) plus 40 foot bond length, and comprised 28 strands. Design load for each tendon was 985 kips (60% GUTS).

Geology

Most of the arch dam foundation consists of hard, pre-Cambrian quartz diorite. The diorite is cut by irregular dikes of hard, medium grained granite, which vary in orientation and thickness. A fault effectively divides the arch dam foundation into three distinct zones): the zone to the right of the fault, the zone to the left of the fault, and the fault zone itself. Each zone has unique mechanical properties, joint systems, and permeabilities. The rock underlying the right portion of the dam is hard, slightly weathered to fresh and generally of excellent qualities. The rock to the left of the fault (which includes the left thrust block foundation), is slightly inferior, being more fractured, sheared and weathered. The fault and the surrounding fractured zone are very

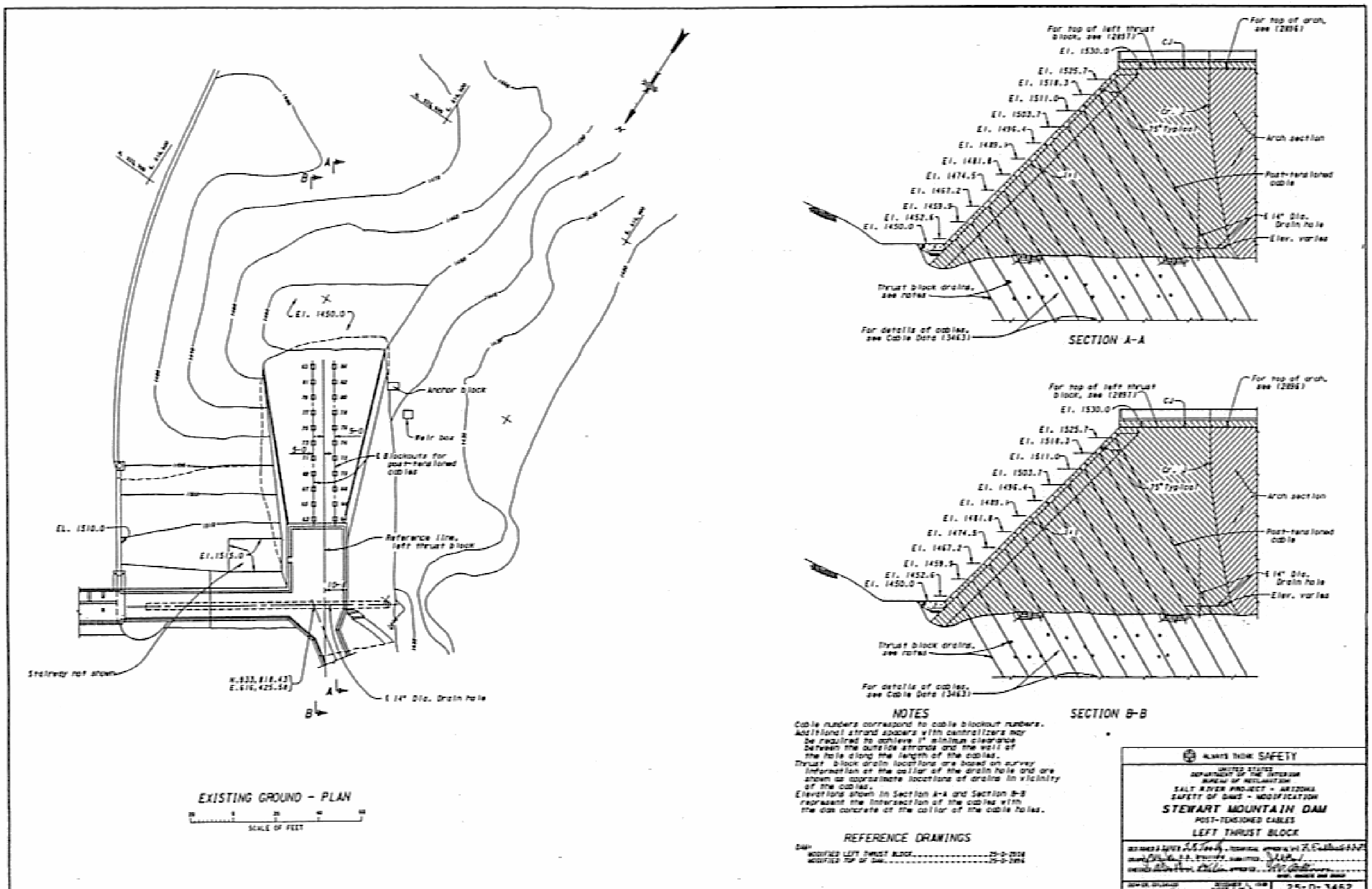


Figure 2. Stewart Mountain Dam — Post-tensioned cables, left thrust block.

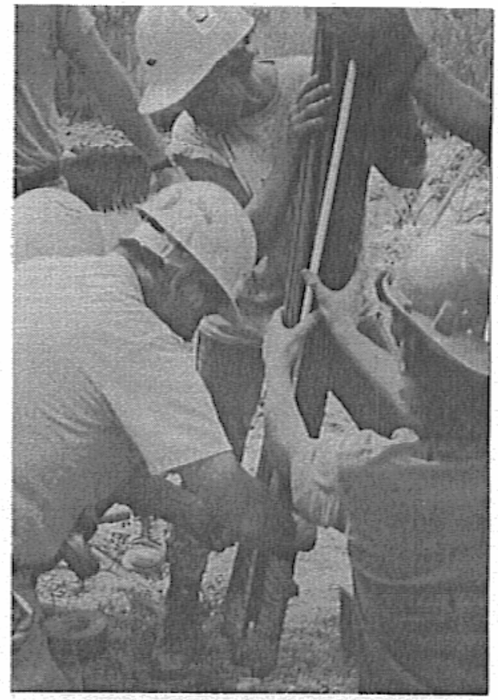
intensely fractured and moderately to slightly weathered. During the design phase, it was assumed that 32 of the arch tendons would be anchored in the right foundation zone (with seven of the tendons in this area anchored in the dam concrete); 15 of the arch tendons would be founded in the left foundation zone; and eight 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 12 feet 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 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 x 10^6 psi (Site l) to 0.5 to 2.5 x 10^6 psi (Site 2) to probably around l x 10^5 psi 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 10 inch-diameter with a down the hole hammer. Tendons consisted of the special epoxy coated strands (each 0.6 inch-diameter), suitably spaced and noded in the bond length and tremie grouted with a stable cement grout.



Placing test anchor and tell tales.

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

Table 1. Details of test anchors, Stewart Mountain Dam.

	Apparent Debonding at Test Load	
Anchor	Actual Site	Average
1A	21 in.	23 in.
1B	25 in.	
2A	40 in.	42 in.
2B	44 in.	
3A	Failed: 142 in. bond	Possibly
3B	73 in.	108 in.

Table 2. Calculated apparent tendon debonding lengths, test anchors, Stewart Mountain Dam.

Laboratory tests on the grout mix indicated that the grout had an elastic modulus of 2.4 to 2.7 x 10^6 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 test 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 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).

Permanent movements were smallest for Site 1 anchors and greatest in Site 3 anchors, reflecting the overall quality of the rockmass. 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 rockmass 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 amounts were at loads of 75 to 100% WL, and were less at higher loads. In addition, whereas 3A showed the classic progressive failure pattern, 3B showed values at 133% (0.057 inches in ten minutes) lower than at 100% (0.064 inches in ten minutes). When restressed to 133% a second time, the creep was lower still (0.045 inches in ten minutes).

These data are consistent with the permanent extension phenomena outlined above, and point to an irregular "ratchet" type rockmass 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 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.

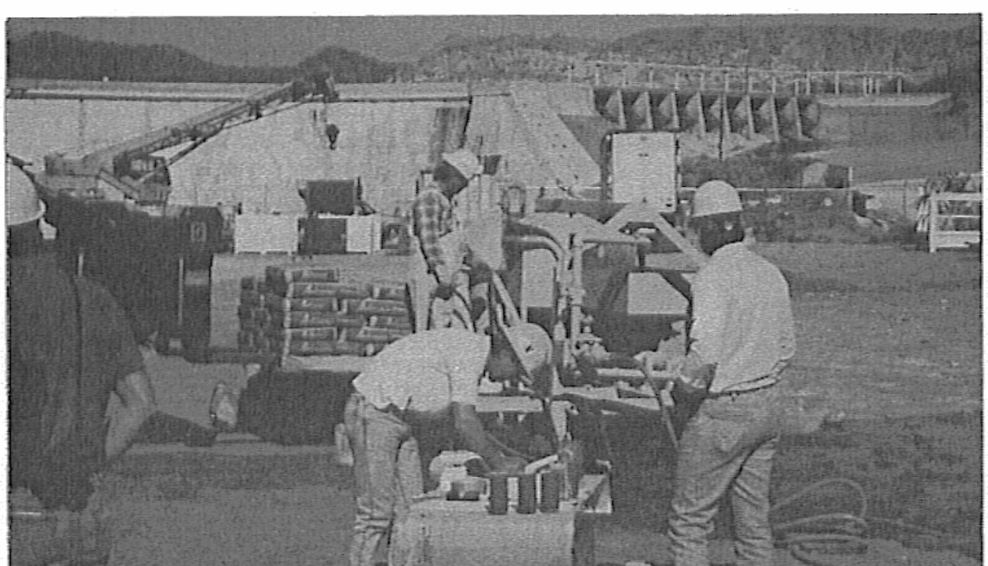
Production Anchors

Recesses, four feet nine inches square and approximately two feet deep, had been formed in the dam crest under a previous contract. At the precise location, bearing and inclination, a 12 inch-diameter hole was cored about five deep at each anchor position. A ten inch-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 ten-inch anchor holes were then drilled using a down the hole hammer, mounted on a new Nicholson Casagrande C12 long stroke, diesel hydraulic track rig. Special hammer and rod attachments were used to promote hole straightness. In accordance with the specifications, the position of the hole was measured at ten foot intervals in the upper 50 feet of each hole, and thereafter at 20 foot intervals to final depth: a maximum of 270 feet. This high frequency of measurement, and the precision required – to within three inches in 100 feet – demanded very special attention. Nicholson worked with their friends in 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 drill's position to be accurately measured through the drill rods, but modification of its 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.

As a further check, USBR personnel ran independent precision optical surveys 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 five feet of a free face was hardly noticed by the dam. This drilling method is extremely cost effective and so helps keep 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. Again special precautions were taken to ensure hole correctness and direction. Every hole was water pressure tested,



Preliminary grout tests.

and pregrouted and redrilled if necessary – prior to a final acceptance survey. Most test stages – which ranged in rock and concrete from 50 to 130 feet proved tight, but other stages needed as many as three pretreatments to allow the specification to be met – 0.02 gpm per lineal foot of hole at five psi excess pressure.

The special epoxy coated strand tendons, supplied 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. 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.

Quantities

Overall, the following quantities were recorded:

	Dam Crest	Thrust Block
Rock	3730 feet	1517 feet
Concrete	8142 feet	1336 feet
Water Tests	252Nr	90Nr
Redrilling	3615 feet	1620 feet

Stressing commenced 14 days after grouting. Twelve tendons were subjected to cyclic Performance Tests, as per PTI (Reference 5) to verify in detail the correct operation of these tendons. The other anchors were tested simply, as per the PTI Proof Test provisions. Given the high loads, and long free lengths, extensions as long as 17.3 inches were recorded at Test Load on the longest tendons (permanent extension of 0.343 inches). 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, 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 manual 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%) and secondary grouting. Again the anchors were proved to have performed well, while no discernible movements were induced in 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 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.
- **Drilling Technology:** using appropriate planning, tooling, equipment and expertise, ten-inch holes can be drilled fast and extremely straight and accurately through concrete and hard rock to depths of over 270 feet. Such methods appear to have absolutely no deleterious affects on the structure. And more systems now exist to pinpoint this accuracy to within inches at this depth.
- **Tendon Technology:** the relatively new product of epoxy coated strand appears workable in the field, and seems to give excellent bonding characteristics.
- **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 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 write separate technical and price proposals, independently assessed, assured 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 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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