The Use of Post-tensioned Tendons at Stewart Mountain Dam, Arizona: a Case Study Involving Precision Drilling

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Abstract

Stewart Mountain Dam near Phoenix, Arizona was modified to resist potential seismic loadings by using post-tensioned tendons. This is thought to be the first use of post-tensioned tendons to rehabilitate a thin arch concrete dam.

Stewart Mountain Dam, built during 1928-1930, is a double-curvature, thin-arch dam with thrust blocks and gravity sections. The arch section is 583 feet long, has a crest width of 8 feet, a maximum structural height of 212 feet and is up to 34 feet thick at the base. At the time of construction, the importance of good cleanup on the horizontal construction joints between lifts was not recognized. These joints were left untreated, resulting in a series of unbonded horizontal planes across the arch section of the dam on 5-foot vertical intervals. The left thrust block required stabilization due to foundation problems, and post-tensioning was also chosen.

To provide the stabilization, 84 tendons were designed. Sixty-two tendons within the arch were installed at about 9-foot centers with free lengths ranging up to 216 feet and bond lengths from 35 to 45 feet. Tendon inclinations were required to vary from vertical to 8°40' off vertical. Twenty-two tendons were placed, inclined 30° off vertical, through the left thrust block. All tendons were placed in 10-inch diameter holes with a specified 9 inch per 100-foot drilling tolerance. Special considerations were necessary to complete the drilled holes for the 62 tendons within the thin-arch dam. A gyroscopic survey tool was the primary device used to assure proper alignment.

This paper discusses the background, design and construction considerations, and the problems encountered, lessons learned, and techniques used in completing the holes. The paper also details the precision drilling, primarily with the down-the-hole hammer, surveying, water testing criteria and techniques, and grouting techniques used for the installation of the 62 tendons within the arch portion.

Introduction

As part of its Safety of Dams Program, the Bureau of Reclamation (Reclamation) in the 1980's evaluated possible structural and hydraulic deficiencies at Stewart Mountain Dam, near Phoenix, Arizona. Structural and hydraulic studies performed on the dam indicated that a number of modifications would be required. Given the unique nature of the modifications, Reclamation decided to accomplish the work through two separate contracts, Stage I and Stage II.

Stage I was awarded to Kiewit Western Company, Phoenix for $18.2 million. This contract was a conventional competitive bid solicitation. The work was completed in 1989. Stage I work included: adding an auxiliary spillway on the right abutment, removal and replacement of the old penstock and bypass outlet works, modifying the crest of dam, placing concrete overlays on the gravity sections and thrust blocks, installing a drainage system, and other miscellaneous items.

In 1990, Stage II was awarded to Nicholson Construction Inc., Atlanta (Nicholson) for $6.5 million. Unlike the first contract, the Stage II contract was a negotiated procurement, because the work primarily involved the highly specialized installation of multi-strand tendons through a thin-arch dam. It is the aim of this paper to describe design assumptions and the conditions leading toward this second modification contract. This paper discusses the design and construction considerations, problems encountered, lessons learned, and techniques used in completing and sealing the drill holes for the tendons.

Background

Stewart Mountain Dam is located on the Salt River approximately 30 miles east of Phoenix, Arizona. The dam was constructed from 1928 to 1930 by the Salt River Valley Water Users Association at a cost of $2.3 million. The Salt River Project (SRP) operates the dam as part of a water-storage and power-generation system on the Salt and Verde rivers. The reservoir, Saguaro Lake, is one of the principal water sources for the Phoenix metro area.

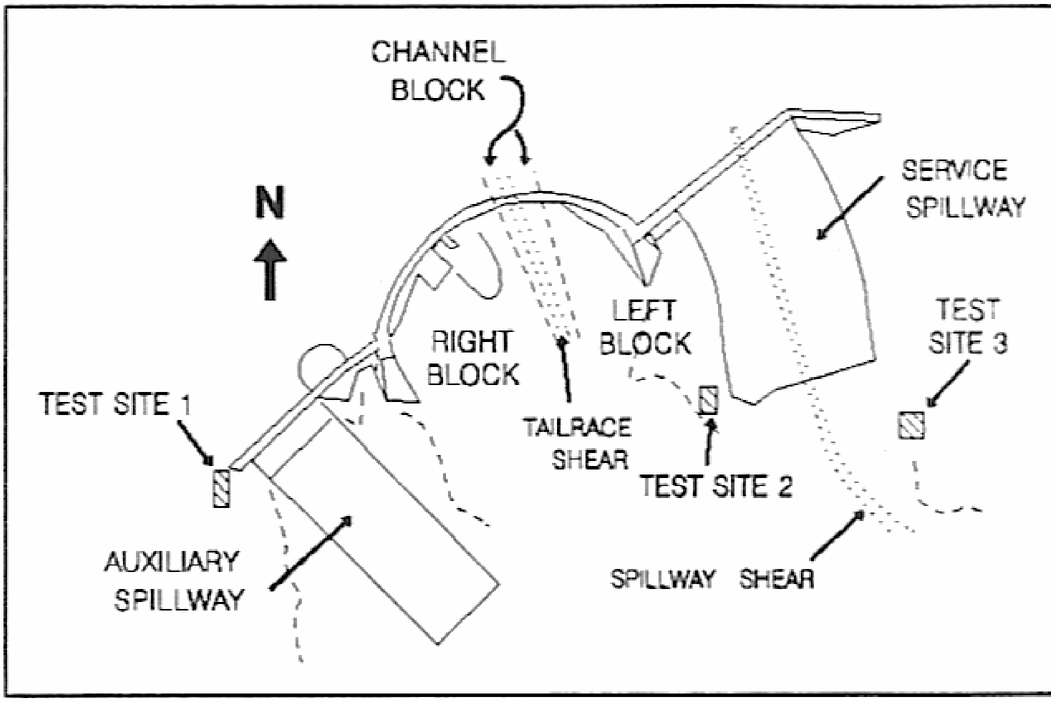

Figure 1 - Location of Main Structural Features & Anchor Test Sites

The original dam was a composite concrete structure that included a double-curvature, thin-arch dam; two concrete thrust blocks; three concrete gravity sections and a service spillway. The arch has a 212-foot structural height, an 8-foot crest thickness, a 34-foot base thickness and a 583-foot crest length. Through the arch portion, there is a 13.5-foot diameter steel penstock connected to a 13 MW powerplant and a 7-foot diameter opening serving as bypass outlet works. On the left side is a service spillway having a capacity of about 90,000 cubic feet/second. The modified dam (Figure 1) has an auxiliary spillway added under Stage I.

Construction practices in the late 1920's did not recognize the importance of cleaning horizontal construction joints before subsequent placements. Concrete placement therefore proceeded without cleanup at the top of the 5-foot high lifts. This practice produced weak layers of laitance at every lift elevation resulting in planes with little or no bond. Vertical contraction and construction joints extending the height of the dam divide the arch into nine sections. The combination of weak horizontal planes and vertical joints created discrete concrete blocks kept in place largely by mechanical arch action.

Another problem impacting concrete, which was not recognized in the 1920s, was the incompatibility of certain aggregates with cement. By the early 1940's, extensive pattern cracking was noted on the concrete surfaces, and monitored. In addition, the arch portion of the dam separated from the powerplant, necessitating new support for the powerplant roof. By 1943 the problems were determined to have been caused by alkali-aggregate reaction, a chemical reaction between aggregates within the concrete, and the cement, which produces a swelling gel. This expansion resulted in significant displacement and cracking across the dam.

Erosion of the bedrock downstream of the left thrust occurred during high flows through the service spillway in 1966, 1978, and 1980 which overtopped the right training wall. The depth of erosion extended below the downstream toe of the left thrust block.

Site Geology

The dam was built on Precambrian intrusive rocks, principally quartz diorite cut by irregular dikes of granite (GR) and smaller diabase (DI) and silicic dikes. From the right to the left abutments, the rock becomes increasingly more fractured and sheared. Two major shears are in the foundation area (Figure 1). The original river channel follows one of these shear zones, and the other is beneath the service spillway. Rock weathering is typically deep except within the active scour areas. Intense rock weathering often extends to a depth of 100 feet except in areas of active scouring. The shear zones are weathered and intensely fractured with clay infilling.

Dam Safety Investigations

Investigations to determine the conditions of concrete and rock began in 1984 (Wellendorf, 1985). Records consisting of photographs, designs and "as-built" drawings and documents were reviewed. Preconstruction investigations within the arch dam and thrust block area were evaluated. The investigation involved extensive subsurface and surface investigations for the dam and surrounding area. Sixteen drill holes were placed within the arch dam and thrust block to determine the condition of foundation rock and concrete, as well as to evaluate the uplift pressure and seepage through the foundation. Detailed surveys were conducted across the dam.

The excavation "as-built" drawings were found to be accurate to within a foot, confirmed by the exploratory drilling. The foundations were found to be excavated to moderately to slightly weathered rock except in the area of the shear zone near the center of the arch and in limited areas elsewhere the foundations. The bedrock foundation of the left thrust block was found to be intensely fractured. Embedded steel was found to be present within the thrust blocks as the design drawings had indicated. This block not only required installation of a drainage system to reduce the uplift pressures, but also required further stabilization in the form of an anchorage system to prevent sliding.

Unbonded lift lines were evident in core recovery. Core breaks nearly always occurred along the lift lines. Seepage had been noted along the lift lines soon after the filling of the reservoir, and also during drilling, when new seepage was noted across the concrete faces along the lift lines. Within the arch section, core drilling with water often required grouting and redrilling. Loss of drill fluids along the lift lines was a common occurrence particularly within the upper 50 feet of the arch. During the drilling of hole DH-503-SM, the generator was shut down when drill fluid from the borehole travelled along a lift line and daylighted along the downstream face within the powerplant (Figure 2).

The original seepage control for the gravity section and thrust blocks consisted of square redwood drains and a single line grouting program. Grouting was performed principally in areas of gravity sections and the thrust blocks. Grout records failed to indicate mix design(s) or the pressure applied during the operation. Grouting apparently did little more than backfill the holes, as the takes were small.

The concrete strength varies considerably from base to crest as well as within the structure. Compressive strength values average 4,950 lb/in^2 in the arch, 4,840 lb/in^2 in the left thrust block, and 3,220 lb/in^2 in the right thrust block. There was no steel reinforcement within the arch except for around the penstock and outlet works. However, within the thrust block, steel reinforcement was placed in mats at 5-foot centers.

A review of measurement points at the top of the dam indicates that there was a rapid volumetric increase from 1937 through 1968, but that the dam had been relatively stable from 1969 to present. The alkali-aggregate reaction within the concrete was found to have permanently displaced the dam crest 6 inches upstream and 3 inches upward (Nuss, 1987).

Water testing confirmed that the foundation and concrete appeared to have high local permeabilities. This testing consisted of mostly falling head tests within the concrete portion of the investigations. Pumps having a capacity of approximately 35 gal/min for most of the boreholes were unable to maintain a constant head within the concrete dam. Packer tests performed within the bedrock indicated that, generally, the bedrock was tight, ranging from 0 to 2.0 gal/min (0 to 23 lugeons). In a few intervals, like within hole DH-404-SM, higher takes of up to 53 gal/min, at 50 lb/in^2 pressure,(>100 lugeons) occurred. No packer tests were performed across the concrete/foundation contact, although drill fluid losses were often reported in the vicinity of this interface.

Vibrating wire piezometers were installed within the bedrock beneath the dam. High uplift pressures were found within the foundation indicating that the grouting operations during the original construction were not effective. Of particular concern were the areas under the wing dams and the thrust blocks.

Final design and pre-contract activities

Most of the dam required stabilization based on the revised MCE (Maximum Credible Earthquake) calculation and the foundation information obtained during the investigations. Seepage along the lift lines was determined to be unsightly but was not detrimental to the structure's stability (Nuss, 1987). It was determined that stabilization would be best achieved for most of the structures by the installation of drains within the structures and/or adding overlays. The principal exceptions were in the areas of the arch and left thrust block. Stabilization of the arch and left thrust block required the installation of post-tensioned tendons.

Designs were significantly impacted by the open lift lines combined with earthquake potential, foundation conditions around the left thrust block, and high uplift pressures. Technical memoranda SM-220-01-87 (Nuss, 1987) and SMC-3110-01-90 (Nuss, in progress) contain the details of the analysis and the recommended improvements to the dam. One of the most significant improvements to the dynamic stability was the installation of tendons across the arch dam.

Since the installation of tendons through a thin-arch dam was believed to be a unique event, concerns were raised as to how the anchor loads on the dam would impact the structure. It was determined that stressing of the tendons would require a specific tensioning sequence and an evaluation period was recommended to determine if there were any adverse deflections or other unusual responses to these loads (Nuss, in progress). During the evaluation period, the strands required corrosion protection. Conventional tendon corrosion protections were inappropriate for the conditions at this site. Encasement of the strands or tendon with sheathing or grease, providing a corrosion-inhibiting environment such as calcium hydroxide, and use of galvanized strands were all options studied and rejected. Epoxy-coated strand impregnated with grit was chosen as the corrosion protection system (Nuss, 1992).

A major construction concern was the ability to drill the tendon borehole within a confined area (Figure 2). The drilling of large diameter boreholes has been performed on a variety of dams (Bruce, 1987). Drilling tolerances as small as 6 in/100 ft have been specified. However, surveys at most of the sites were performed with magnetic survey tools with uncertainties between 0.1° on vertical holes (2 in/100 ft), to 0.2° on 10° off vertical holes (4 in/100 ft), and to 0.8° on 30° off vertical (18 in/100 ft) [Wolf and deWardt, 1981]. The uncertainty of the magnetic tools was often equal to the tolerance specified. At Stewart Mountain, drilling tolerances required the drill holes to fall within a 12 inch radius at a 100 foot length for an ideal alignment. Electrical interference from the powerplant could impact the bearing reading and within the thrust block, steel reinforcement was pervasive throughout the concrete and would significantly impact the magnetic readings. Technical memorandum 3610-89-28 (Bianchi, 1989), based on published information and conversations with contractors, determined that the drilling could be performed within a 1-foot radius at a depth of 100 feet and that surveying tools were available with a maximum uncertainty of 2.5 in/100 ft for any of the arch tendon holes.

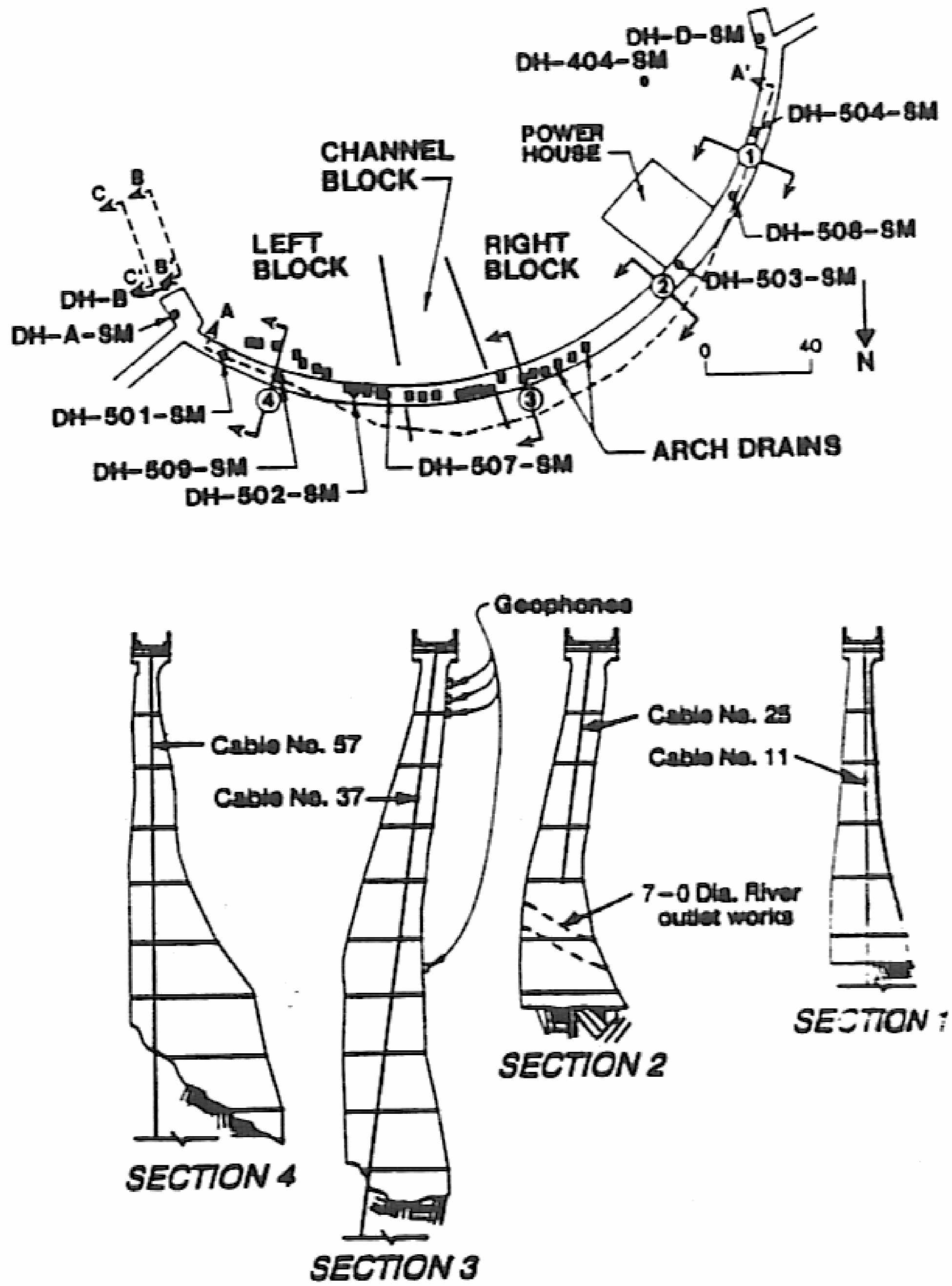

Figure 2 - Location of Exploration Holes, Arch Drains, & Typical Sections

Because of the relatively unique nature of the tendon installation, it was determined that a specialty contractor would be required. Much of the other work required more standard repairs or construction activities, such as adding the auxiliary spillway on the right abutment, adding overlays, removing and replacing a crest structure and drilling a drainage system.

The specialized nature of tendon installation and the uniqueness of this job made it essential to obtain an experienced speciality contractor. Specific concerns were the drilling, surveying, installing, and tensioning along the crest of the arch. Evaluation criteria were developed to determine parameters for the type of equipment and methods used in conformance with the specification, and what contingencies would be used if specific situations arose, with particular emphasis on drilling a hole within tolerance and on minimizing induced vibrations. The negotiated procurement process was determined to be the best method for evaluating contractors and their construction methods. Once the decision was made, the installation of tendons was removed from the Stage I contract, as was the related work such as the backfilling area around the left thrust block and gravity section.

The negotiated procurement process required that offerors submit both a technical and a cost proposal which were evaluated separately. The technical proposal addressed specific concerns, qualifications of key personnel listed within the specification, and detailed the approach to the work. These were evaluated by a technical proposal evaluation committee. After completion of the technical evaluation, a cost/price evaluation was made by a cost/price evaluation committee on the cost proposals for all offers classified as technically acceptable. The evaluation factors were provided within the specification paragraphs. Award was determined by evaluating the combined technical and cost/price scores for all offerors.

Stage I modification

The main intent of the Stage I contract was to provide additional flood protection as well as to improve existing operational features of the dam. The work included the construction of a new auxiliary spillway, removing, modifying and replacing the crest of the dam, adding six to eight-foot thick overlays on the thrust blocks and gravity sections, replacing one river outlet works and power penstock, plugging two existing outlet works, providing additional drainage features, and grouting selected portions of the dam foundations, as well as various other items requested by SRP for ease of maintenance.

The primary improvement which affected the Stage II contract was the modification which added three feet to the top of dam across the entire arch section of the dam. Post-tensioned anchor work required a bearing surface through which the high concentrated loads could be transferred and distributed to the non-reinforced arch structure. As a result, adding an additional 3 foot reinforced cover with blockouts provided an adequate area capable of supporting the 40 million pound load exerted on the arch by tensioned tendons as well as providing a wider work area, easier access, and recesses for anchor heads.

Other improvements for Stage II, made under Stage I, were construction of blockouts within the left thrust block overlay and reduction of uplift pressures within the foundation. The grouting in the left thrust block helped by controlling seepage, and potentially reducing borehole caving and the amount of pressure grouting required in the thrust block bond lengths during the tendon installation. The installation of drainage within the thrust block and within the bedrock reduced the uplift pressure and seepage.

The only other item impacting the Stage II modification was the fact that the system of drainage holes' drill reports and core provided additional subsurface information. This assisted in determining the three rock mass categories used during the pullout tests. The only potential negative impact of the Stage I work was that by adding all the drains within the thrust block and arch, more potential obstructions were created if the tendon holes were not drilled within a tolerance of 9-inches/100 feet (Figure 2).

Stage II modification

The work within the arch portion of the dam consisted primarily of:

1) drilling, surveying, water testing, and, as necessary, pressure grouting 62 tendon holes ranging in inclination from vertical to 8° 40'off vertical;
2) furnishing and installing 62 tendons each consisting of 22 epoxy-coated strands, varying in free length from approximately 40 to 213 feet;
3) determining the recommended bond length, and grout mix design for the bond and free lengths;
4) grouting the bond length and stressing the tendon;
5) evaluating the tendon during the 100 day evaluation period;
6) when required, retensioning the tendons;
7) and grouting the free length, cutting the excess strands and backfilling the blockouts with concrete.

The left thrust block work consisted primarily of:

1) drilling, surveying, water testing, and, as necessary, pressure grouting 22 tendon holes inclined at 30° below vertical;
2) furnishing and installing 22 tendons, each consisting of 28 epoxy-coated strands;
3) grouting the bond length and stressing the tendons;
4) grouting the free length;
5) cutting the excess strands and furnishing and placing reinforced concrete within the blockout;
6) and placing and compacting backfill at the toe of the left thrust block and left gravity section.

Some of the key equipment mobilized to site included a Casagrande Model C-12 diesel hydraulic track drill, equipped with a down-the-hole hammer; two Model 950/350 air compressors; a 12 ft^3 colloidal grout plant, a backup grout plant; and an Eastman-Christensen (EC) Seeker-1 rate-gyro survey system.

Bond length determination test. - Prior to any production drilling of tendon holes, a full scale anchor test was required (Scott and Bruce, 1992). Three sites were selected to reflect major rock mass categories (Figures 1 and 2). The intent of the test program was to determine the anchoring requirements within different bedrock conditions. Other benefits from the test program included providing an opportunity to identify and correct any unforeseen construction difficulties through a full-scale trial run and to provide the field and design staff with an opportunity to be become familiar with the procedures and capabilities of the equipment.

The specifications required that two tests be conducted at each of the three sites, that each site be cored (N-size) prior to drilling the tendon hole and that other procedures be similar to the production tendon installation. This involved using the Casagrande C-12 rig to drill the hole, performing water tests to determine the water tightness of the boreholes, developing and using the proposed grout mix design, and using a maximum tendon pullout load of 150 percent of design load. Nicholson satisfied the requirements in the proposal, performed a more rigorous water testing procedure recommended by Houlsby (1976), added a test load based on the highest actual loads expected, performed actual surveys using the Seeker, and used tendons consisting of 28 epoxy-coated seven-wire strands.

From the bond length tests, the recommended bond length for the production tendons was determined to be 35 feet within the quartz diorite and granite as results from site 1 (quartz diorite) and site 2 (combination of quartz diorite and granite) were similar. Test site 3 was within a very intensely to intensely fractured mylonitized granite with fractures dipping subvertically. The 10-foot bond zone hole failed at 98 percent of design load (985 kips). From site 3 results, all holes falling within the shear bedrock (holes 40 through 47 and areas of hole 53 and 54) were designated to have a bond length of 45 feet.

Survey testing. - The drilled tendon holes for the arch and thrust block required precision drilling because of the proximity of free faces on the arch dam and the drainage systems installed under the Stage I contract (Figures 2 and 3). No offsetting of the tendon hole was allowed. The specification required a drilling tolerance within a 9-inch deviation at a length of 100 feet with an uncertainty allowance of 3 inches for the survey equipment (thus maximum 12 inches per hundred feet apparent deviation). The specification permitted advancement of tendon holes 10 feet ahead of the survey. This allowed surveying of the hole without withdrawing the down-the-hole hammer.

In addition, within the arch section, a curvature tolerance was required because no friction loss on the tendon sidewall could be tolerated in the upper 50 feet of a hole: the use of centralizers on the tendons in the upper 50 feet was prohibited. The curvature was measured relative to a line extending from the

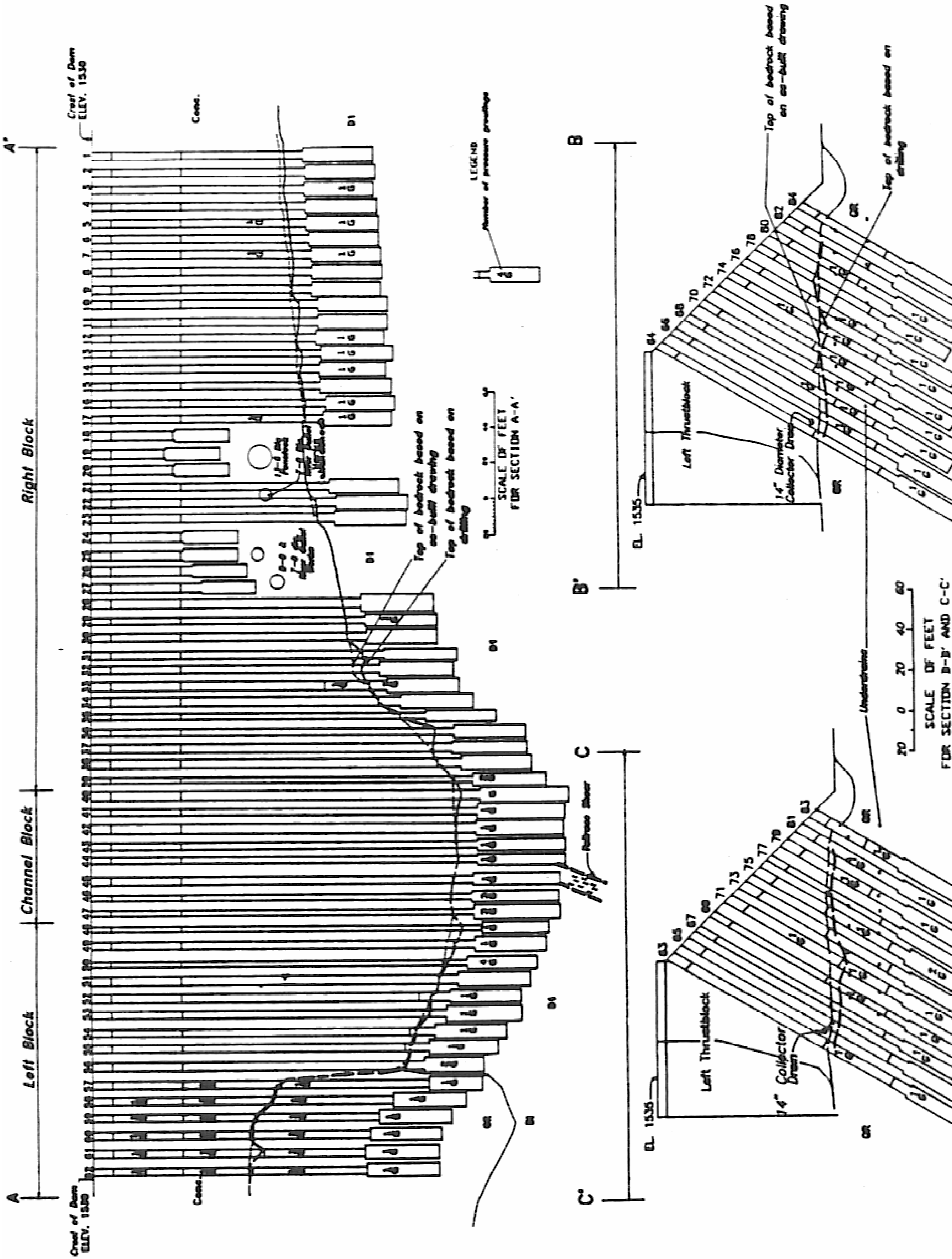

Figure 3 - Vertical Sections through Tendons showing Grout Intervals, and Number of Tests

center point at the collar to the actual center point location at the 50-foot depth. The perpendicular distance between this line and the actual centerline of the hole at any point within the 50-foot length had to be less than 0.5 inches in order to ensure a 1-inch minimum clearance between the outside strands of the tendon and the sidewall of the hole.

Nicholson performed a test to confirm the sensitivity of the Seeker-1 instrument by suspending a metal pipe on the face of the auxiliary spillway. The pipe was independently surveyed by conventional means and the two sets of results compared. The biggest problem with the test was that on the day of the test the wind was significant and the pipe tended to sway in the wind: the pipe was only secured at both ends. The readings fluctuated accordingly, but did prove the acceptability of the method.

Drilling and survey testing on the arch. - The final acceptance of the drilling and survey procedure was performed on tendon hole 37 within the arch. This hole required an inclination of 7° 10' upstream, on a bearing of 344°. Based on the as-built top of excavation the anticipated length in concrete was 187 feet. The specified free length was therefore 197 feet (187 ft plus 10 additional feet below the concrete/rock contact) plus 35 feet for the bond length. The actual total length would be determined by the location of rock encountered during drilling.

The specifications required the tendon hole be surveyed at 10 foot intervals for the first 50 feet and at 20 foot intervals along the remaining length. Data from the survey readings were processed within the time it took to withdraw the survey tool from the borehole and reconnect the drill rods to the down-the-hole hammer. Initially, although the readings from the Seeker were within the specified tolerances, the data required conversion into formats that an inspector could appraise quickly (e.g. see Figure 4).

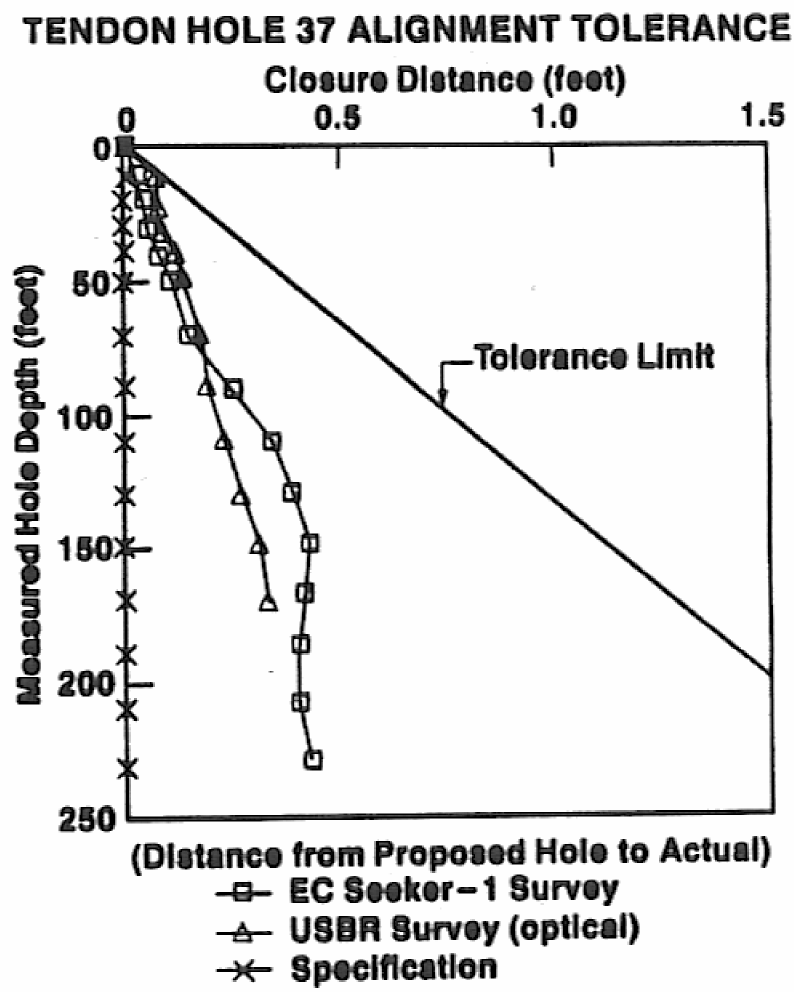

Figure 4

To promote hole straightness, the C-12 rig utilized a 10-inch button carbide bit on a down-the-hole hammer with a spiral overhammer stabilizer. The 6-foot long hammer was followed by an over-the-barrel stabilizer on the first 6-inch diameter drill rod. The 6-inch diameter drill rods were in 10-foot lengths to a depth of 50 feet, then 20-foot lengths thereafter. The inclination of the mast was confirmed by a digital level; bearing was checked by reference to the radial scribe marks on the dam crest.

During the drill evaluation test, Reclamation personnel monitored the behavior of the dam with eight Geokon crackmeters and four geophones located on the downstream face of the dam (Figure 2). The crackmeters were variable resistance vibrating wire, continuous reading instruments, sampling every 30 seconds during drilling and recorded on an SR10 Data Recorder. The crackmeters were positioned vertically across alternate lift lines in the upper 83 feet of the dam. The geophones were bolted to the dam face and three were placed at intermediate locations in the upper 20 feet of the dam, while the fourth one was located approximately 110 feet below the dam crest. In monitoring the crackmeters, any movement in excess of 0.05 inches between any two readings would require further evaluation. Results during the drilling showed maximum movement varied from 0.00218 to 0.00510 inches overall (Table 1). For each of the monitored locations, the maximum crack movement occurred at precisely the time the drill bit was at the elevation of the lift line. Except for the moment when the bit passed across the lift line, the effect of drilling could not be differentiated from normal crack movement caused by temperature variations.

Table 1 - Summary of Crackmeter Data

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	[1]	0.00353	5.5
1470.17	[1]	0.00376	5.5
1460.21	[1]	0.00459	5.5
1450.23	[1]	0.00440	5.5

[1] No discernable movement was detected during the drilling operation.

The four geophones measured peak particle velocity (ppv) on a continuous basis. In general, the maximum peak vector sum was greatest when the drill bit was closest to the measurement location (Table 2). The maximum peak vector sum of 0.147 inches/sec was well within the allowable criterion of 2 inches/sec for blasting vibration normally (Figure 5) accepted for structures by Reclamation (Reclamation, 1980) and Corp of Engineers (Department of Army, 1972).

The entire length of the hole was drilled in the dry, and progress was slowed only momentarily when steel was encountered at depths of 12 and 44 feet. The Seeker's reading indicated a deviation in the total length of hole of 5.48 inches in 230 feet or 2.38 inches in 100 feet. Curvature yielded a maximum deviation of 0.5 inches within the upper 50 feet of the hole. The borehole survey using the Seeker was checked by Reclamation's optical device, with both in close agreement and well within the required tolerances (Table 3).

Table 2 - Summary of Geophone Data

Elevation of Geophone (ft)	Approx. Drill Bit Elevation at Max. Peak Vector Sum (ft)	Maximum Peak Vector Sum (in/sec)	Approx. Distance from Geophone to Hole (ft)
1521.12	1521	0.039	5.0
1517.14	1454	0.017	5.0
1512.94	1581	0.066	5.0
1423.15	1433	0.147	8.0

Initial Core Drilling. - Overcoring of the upper 4 feet of each hole was performed by a subcontractor, Concrete Coring Company of Phoenix. To assure the coring operation attained the correct alignment, each hole was surveyed and checked by both Nicholson and Reclamation. An alignment and centering guide was positioned within each concrete blockout. The drilling equipment consisted of two hydraulic-driven drill motors mounted on a 4-inch by 5-foot drill mast located exactly over the centering guide. However, at 11 sites, a 6-foot steel reinforcement was used as a centering guide rather than a wooden dowel. At these sites, concrete coring had to be extended to a depth of 8 feet to remove the steel guide.

After each hole was cored, 5-foot long, 10-3/16-inch internal diameter steel cylinders were installed. The alignment was set by a tangential stringline, while radial inclination was set with a digital level. It was then grouted in place with a non-shrink grout. Within 4 days of setting each tube in place, the C-12 was positioned over the hole and the drilling began.

Production drilling on the arch. - Drilling on the arch tendon holes began in mid March, 1991 and was completed by early May, 1991, except for tendon hole 11 which had a broken bit that required grouting and coring to remove. Drill cuttings were removed by compressed air from tandem 350 psi, 900 cfm, Sullair compressors delivering through the center of the drill string. Minimizing high instanteous pressures was addressed in the technical proposal. At test site 3, clay infilling within the bedrock plugged the stabilizer groves and a resulting temporary pressure buildup was noted. Nicholson modifications to the discharge ports of the stabilizer prevented such buildups from occurring, so that no lift lines on the arch or the left thrust block were threatened. One incident occurred during drilling hole 62 when large air bubbles were seen surfacing upstream of the dam. However, once the hole was advanced, the crack was apparently sealed by the cuttings. Another unusual observation occurred during drilling hole 40: seepage was observed coming out at the base of the arch drain collar in the vicinity of the hole. This condition disappeared after two days.

Vibration Monitoring of Hole 37

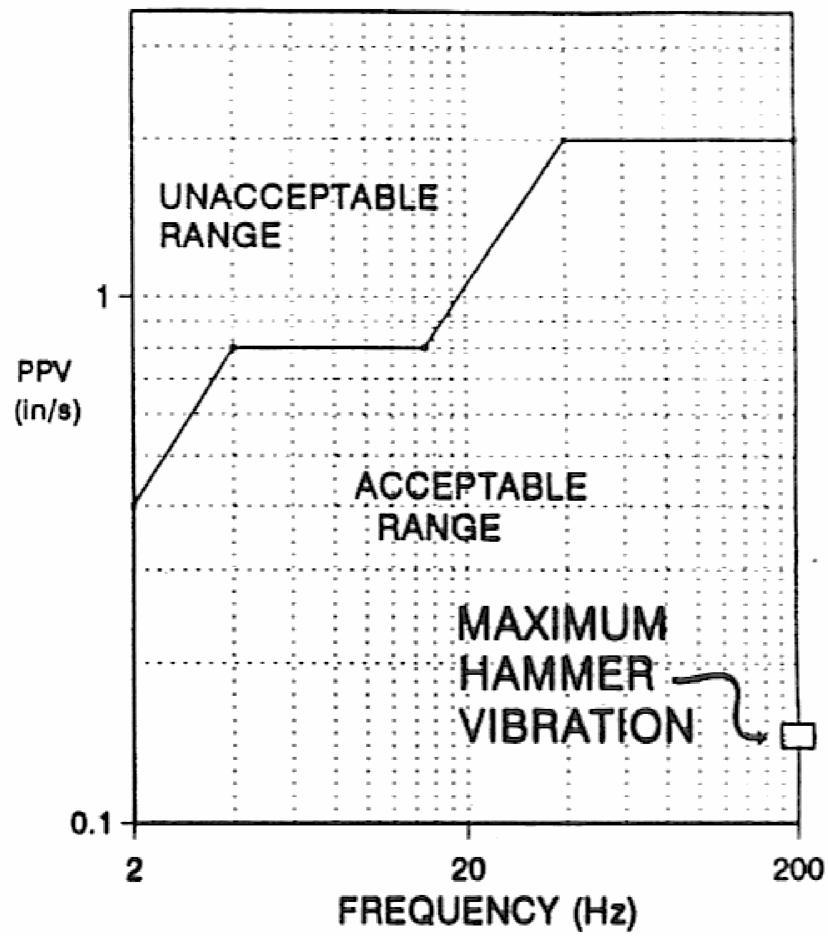

Figure 5

A total of four bits were lost down various holes; three were recovered quickly with a bit retrieval tool, while the fourth was recovered in a core barrel, after being encased in grout. Steel was regularly encountered at the 12 foot depth in many of the arch holes. Steel cuttings were usually lifted from the hole by air. Larger cuttings required magnets to remove them.

Surveying required inserting the torpedo-shaped Seeker into the rods by wireline. These surveys indicated no doglegs within any of the holes. Reclamation periodically surveyed the holes to determine if the Seeker was operating within the specified tolerance. The principal limitation of the optical survey check system developed by Reclamation was that it required a line of sight to determine the alignment. Table 3 is a select summary of holes surveyed by both methods and the difference between the two readings. Only one, Hole 4, was surveyed initially to be apparently outside of the required limit of 9 inches within a hundred feet. The Reclamation optical system confirmed the hole had deviated only 6 inches within 100 feet. As a result, realigning the hole was not required as the hole was truly within tolerance.

Another change made by Nicholson was to drill an additional 5 feet beyond that required. This additional length served as a sump to collect cuttings which were either too large or too heavy, such as steel, and that may have fallen into the hole and were not lifted from the hole as cuttings.

Table 3 - Survey Deviation Summary

Tendon Hole	USBR Final Length (ft)	DEVIANCE (in) EC v/s Spec	USBR v/s Spec	E/C v\s USBR[1]
4	110	0.85	0.75	0.21
22	70	8.19	8.14	0.08
46	114	0.05	0.19	0.16
47	107	0.36	0.37	0.30
48	104	0.57	0.61	0.17
49	107	0.15	0.04	0.11
50	83	0.26	0.08	0.19
51	90	0.25	0.22	0.09
52	90	0.09	0.13	0.03
53	68	0.10	0.07	0.05
54	87	0.16	0.15	0.01
55	68	0.14	0.13	0.06
56	76	0.21	0.18	0.06
57	70	0.07	0.09	0.03
59	50	0.04	0.03	0.07

[1] Deviations are based actual location differences

Thrust block drilling. - A drilling evaluation test on tendon hole 82 was performed to determine the impact of drilling through the steel reinforcement and maintaining an alignment along a 30° incline from vertical. Upon successful completion of the performance test, production drilling commenced in early July, 1991. All 22 holes were completely drilled to full depth in late July, 1991. None of the holes required grouting to correct misalignment or caving conditions. None of the drains installed under the Stage I contract were encountered during the drilling.

Inclined drilling produced a definite curvature in an downward direction. Four tendon holes (77, 78, 79, and 80) were outside the drilling tolerance with deviations of 10, 10, 16, and 15 inches in 100 feet, respectively. However, these holes were not required to be redrilled since they were not in danger of intercepting any other tendon holes or drains in the area. This deflection was attributed to the weight of the drill rods combined with the thrust force tending to level the drill string. To counter the natural deviation, the drill mast was set at an initial inclination of 30°30' for each hole, rather than the 30° specified.

Water testing. - Water testing of arch tendon holes for the bond lengths was in accordance with the Recommendations of the Post-tensioning Institute (PTI, 1986). The maximum allowed water loss within the bond length was specified for a 10 minute interval as being 0.001 gallons per minute, per inch diameter, per foot of hole with a 5 lb/in^2 pressure at the collar. The free length requirements were relaxed to 0.002 gallons per minute, per inch diameter, per foot of hole with a 5 lb/in^2 pressure at the collar. The specified water testing performed during the drilling investigation had indicated that most of the intervals would fail even the free length criterion of water testing. The specification required water testing be performed on a minimum of two stages for holes 18 through 27: the upper 50 feet, and the remainder of the hole. Arch holes 57 through 62 were to be have at least 4 stages (2 stages within concrete and 2 stages within foundation). All the remaining holes were water tested at least in 3 stages (2 stages within concrete and the bond zone). Figure 3 shows the test intervals and shows the number of times water retesting was required.

Water testing was typically conducted in groups of 5 or 6 holes. Problems occurred if the packer test was performed too soon after drilling. Holes which had been drilled on the shift prior to water testing tended to give higher water loss readings than holes which had achieved static water level. Figure 3 displays numbers of water tests required to fulfill the requirements of the specifications.

Pressure Grouting. - Where stage water tests exceeded the criterion, pressure grouting was performed as specified. If the upper 50 foot interval was found to be permeable and the remainder of hole was found to be tight, the entire hole would require grouting and redrilling. If only an intervening interval was found permeable and the rest of hole was found tight, the permeable interval to the bottom of the hole required grouting. In nearly every case where there was an isolated interval showing a water take, the interval was retested after pressure grouting and was found tight. Figure 3 indicates the intervals requiring pressure grouting (G) and number of time pressure grouting was performed within an interval.

The specification required that after pressure grouting, the hole was to be redrilled and resurveyed. Based on the mix design, the grout was drilled out within a period of no less than 12 hours, and no longer than 48 hours after completion of grouting. If redrilling occurred after the 48 hour period, the borehole would have required resurveying. Pressure grouting and regrouting were generally only required in the bond length intervals. The tested zones in the concrete dam were very tight and met the specificiation criterion for water tightness. In Hole 50 the bond length required 4 pressure grouting operations prior to acceptance, but this was atypical.

Completion of tendon installation. - Fabrication of the tendons began shortly after determination of the bond lengths from the pull out test. These tendons were fabricated and wound onto reels at the Dywidag plant in Illinois, and delivered to site. Fabrication consisted of cutting strands to the required lengths and installing centralizers, stabilizers, and two 1-1/2 inch grout lines within the bundle of strands. One grout line was intended to grout the bond length; the second was for grouting the free length.

Prior to insertion of the tendons, the bearing plate was installed, leveled, and grouted. Installation of the first tendon began in June 1991.

Upon the satisfactory completion of each hole, the tendon was carefully unrolled from the reel into the hole to ensure that the epoxy-coated strands and grout pipe were not damaged during the installation. The tendon was suspended within the hole and the bond length was grouted. Specification required that the mix be capable of achieving a minimum compressive strength of 3,500 lb/in^2 in 7 days and that the mix be capable of flowing and fully encapsulating the tendons. By the end of August , 1991, all the stressing of the tendons on the arch was completed. This began the 100-day evaluation period. By mid September, 1991, all the thrust block tendons were stressed.

Upon completion of the stressing of thrust block tendons, grouting of the free length was conducted. The strand "tails" were then cut and the blockouts were backfilled with reinforced concrete to protect the head assembly from physical damage and corrosion.

During the 100-day evaluation period, Reclamation monitored the movements of the arch after the application of the load. If during the period any adverse defections or long-term trends were observed, Nicholson had the equipment onsite to detension the tendons and relieve the load.

The 100-day evaluation period for the stressed tendons within the arch was completed in early December, 1991. Instrumentation and survey data indicated that there was no significant movement of the dam as a result of tendon loads. It had been decided that all tendons with final lift-off readings of less than 108.5% of design load (DL) would be restressed. Any tendons which had relaxed below 108.5% of DL would be restressed to at least 108.5% DL. None of the tendons required restressing.

The free length was grouted without significant problems using the same mix design of 0.45 water/cement ratio and with 0.5% of superplasticizer. The grout tube on tendon 32 was initially plugged but was eventually cleared and grouted. Due to the possibility of hydrofracturing the dam concrete, the grouting procedure for the free lengths in Holes 37 through 47 was modified. Following placing and setting of the first stage of free length grout, the tremie was perforated, allowing the rest of the hole to be filled. Upon satisfactory completion of grouting, the strands were cut and blockouts backfilled with reinforced concrete.

Lessons Learned.

Negotiated Contract - The negotiated type of contract was particularly beneficial in this situation since the work involves a highly specialized, rapidly developing and complex operation. With a negotiated contract, the Contractor provided from his experiences a technical proposal that provided the assurance and solutions that the equipment and personnel were capable of performing the work and that there were sufficient contingencies to correct potential problems should any occur. Also, a negotiated contract allowed for written and/or oral dialogue with the contractor, and ensured that only the more qualified contractors remained eligible during the best and final offer process.

Pullout test - The test program helped determine the actual bond lengths and verified the design assumptions, but also served as a training opportunity for designers, inspectors and contractor. The equipment and techniques were adjusted to the conditions at the site eliminating the majority of the procedural problems prior to beginning actual production drilling. Most of the inspection staff and designers were provided an opportunity to observe the installation and tensioning of tendons. Also fundamental data on rock anchor performance were provided for the international community (Scott and Bruce, 1992).

Borehole survey tools - Seeker or similar gyroscopic survey tools are best suited for inclined boreholes where tight drilling tolerances are required. The uncertainty of the survey needs to be considered in the selection of the survey tool. For vertical holes, magnetic survey tools maintain comparable tolerances and uncertainty, but bearing readings for magnetic devices on holes inclined from vertical at 30° have uncertainities of approximately 0.9° (18 in/100 ft) [Wolf and deWardt, 1981]. For conditions where magnetic influences can impact readings, the gyroscopic tool is ideally suited. The principal limitation of the device is that it is expensive to run and maintain, and as are all bore hole survey instruments, they are sensitive to vibrations. The best advantage of the gyroscopic tool is that it can be used within the drill rods, and so minimizes delays.

The use of the optical theodolite by Reclamation is considerably less expensive and apparently more accurate. The principal drawback to the system is that it is a line of sight instrument and therefore cannot be used in curved holes to any great depth and cannot be used with water or other fluid in the hole.

Down-the-hole hammer - The use of the down-the-hole hammer when operated by qualified personnel and proper equipment can maintain a 10-inch borehole within a 9-inch per hundred foot alignment to depths of over 250 feet for subvertical holes to 30° off vertical. This hammer, considered in some circles as a potential source of vibration and cause of cracking and widening cracks, was not a problem at Stewart Mountain Dam. There were no indications of doglegs within the drilled holes. The hammer was capable of advancing through steel without significant problems on drilling rates or deflections. However, it should be pointed out that even with qualified personnel and the best equipment, four tendon holes (77, 78, 79, and 80), failed to fall within the tolerance specified until adjustments were made to the rig's setup.

Water testing of tendon holes - The recommended water testing requirement of 0.001 gallons per minute, per inch diameter of hole, and per foot of interval is a very tight requirement. Packer tests within the foundation indicated takes in the range from 0 to 23 lugeons. Regrouts within the bedrock were required up to 4 times in one hole before acceptance. Water testing recommended by PTI does not take into account the surrounding sidewall conditions, and multipressure water testing criteria should be considered. Although not tested except during the pullout testings, the best criterion would be 0.05 gal/min/lb/in^2 over 10 feet length or 10 lugeons (Littlejohn, 1975). The only other problem was that water testing should not be performed prior to the hole reaching static equilibrium in order to prevent unnecessary grouting.

Appendix 1. References

Bianchi, R.H., (1989). *Technical Memorandum No. 3610-89-28.* Division of Geotechnical Engineering and Geology, Reclamation, Denver.

Bruce, D.A., (1987). *Technical Note B5006.* Nicholson Construction.

Casataneda, P., (in progress). *Technical Report of Construction, Stewart Mountain Dam, Safety of Dam, Post-tensioned Cables,* Arizona Projects Office, Phoenix.

Department of Army, (1972). *Engineering Manual EM 1110-2-3800, Engineering and Design, Systematic Drilling and Blasting for Surface Excavations.* Office of Chief of Engineers, Washington.

Littlejohn, G.S., (1975). Acceptable Water Flows for Rock Anchor Grouting. *Ground Engineering.* vol 8 (2) pp 46-48.

Houlsby, A.C., (1976). Routine Interpretation of the Lugeon Water Test. *Quarterly Journal of Engineering Geology,* Vol 9 (3), pp 303-313.

Nuss, L., (1987). *Technical Memorandum No. SM-220-01-87.* Civil Engineering Division, Reclamation, Denver.

_____. (in progress). *Technical Memorandum No. SMC-3110-01-90.* Civil Engineering Division, Reclamation, Denver.

Post Tensioning Institute (PTI), (1986). *Recommendations for prestressed rock and soil anchors.* Phoenix.

Scott, G.A. & Bruce, D.A., (1992). *Full scale field tests of high capacity rock anchors.* Presented at 33rd U.S. Rock Mechanic Symposium, Santa Fe.

U.S. Bureau of Reclamation, (1980). *Blasting Review Team Report.* Dams Branch, Division of Design, Denver Federal Center, Denver.

________________, (1984). *Salt River Project Dams, Salt River Project, Arizona, Safety of Dams Program, Modification Report.* Department of Interior, Denver Federal Center, Denver p 78.

Wellendorf, C.S. (1985). *Geologic design data for Stewart Mountain Dam modification.* USBR, Arizona Project Office, Vol 1.

Wolf, C.J.M., & deWardt, J.P., 1981. "Borehole Position Uncertainty - Analysis of Measuring Methods and Derivation of Systematic Error Model. *Journal of Petroleum Technology.* pp 2339-2350.