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DAM REPAIR BY ANCHORING AND GROUTING:

SOME RECENT NOTABLE CASE HISTORIES

by

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Abstract

The specialty geotechnical construction techniques of rock anchoring, and grouting, are in common use throughout the United States. Their state-of-practice has been summarized in several recent publications. There remains, nevertheless, the need to continue to publicize good case history information, particularly on projects where innovation has been introduced for special technical, commercial or logistical reasons. This paper provides such details from four recent major projects.

1. INTRODUCTION

Structural and foundation repair using the techniques of rock anchoring and grouting is a common feature of our dam safety modification and rehabilitation programs. Groups such as USCOLD, ASDSO and ASCE provide regular platforms for advances in these fields to be reviewed from a state-of-practice viewpoint (e.g. Bruce 1989 a., 1990, 1992, 1993 a., b.) In addition, however, the value of detailed case histories of especially interesting and innovative projects, remains clear.

This paper contains information on four dams, recently the subject of significant anchoring or grouting activities. They have been selected on the grounds of being extraordinary in some aspect of the work:

(1) Stewart Mountain Dam, AZ - where long, high capacity rock anchors were installed to combat seismic damage potential to the delicate double curvature arch section.

(2) Boundary Dam, WA - where high capacity rock anchors were installed upwards from within an access tunnel, through the rock abutment for rock slope stability, under very adverse access and climatic conditions.

(3) Jocassee Dam, SC - where innovative grouting featuring "Responsive Integration" concepts was conducted to reduce abutment seepage flows, and

(4) Horse Mesa Dam, AZ - where drilling and grouting techniques were used to create in-situ protection to a downstream access road, susceptible to erosion during periods of high discharge flows.

Each case history is introduced in similar format.

2. STEWART MOUNTAIN DAM, ARIZONA

2.1 Dam Description - Stewart Mountain Dam is located on the Salt River approximately 30 miles east of Phoenix, Arizona. The dam was constructed from 1928 to 1930. 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.

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 power plant and a 7-foot diameter opening serving as bypass outlet works. On the left side there is a service spillway having a capacity of about 90,000 cubic feet/second. The modified dam has a new auxiliary spillway on the right abutment added under an earlier safety modification.

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 - 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.

2.2 Introduction to the Problem - 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 concrete of relatively high water content was 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 lift 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.

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

2.3 Anchor System Design - Sixty-two tendons were designed for installation at approximately 8-foot centers along the crest. Free (stressing) 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 1). 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 averaged 630 kips (range 550 to 750), equivalent to about 50% GUTS (Guaranteed Ultimate Tensile Strength).

2.4 Construction - 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., 1991b.; Scott and Bruce, 1992).

A wealth of fundamental information was obtained from these 6 full scale test anchors, 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.

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 very 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 2) 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 particularly sensitive part of the arch, close instrumentation of the structure's downstream face was conducted. Crack meters placed

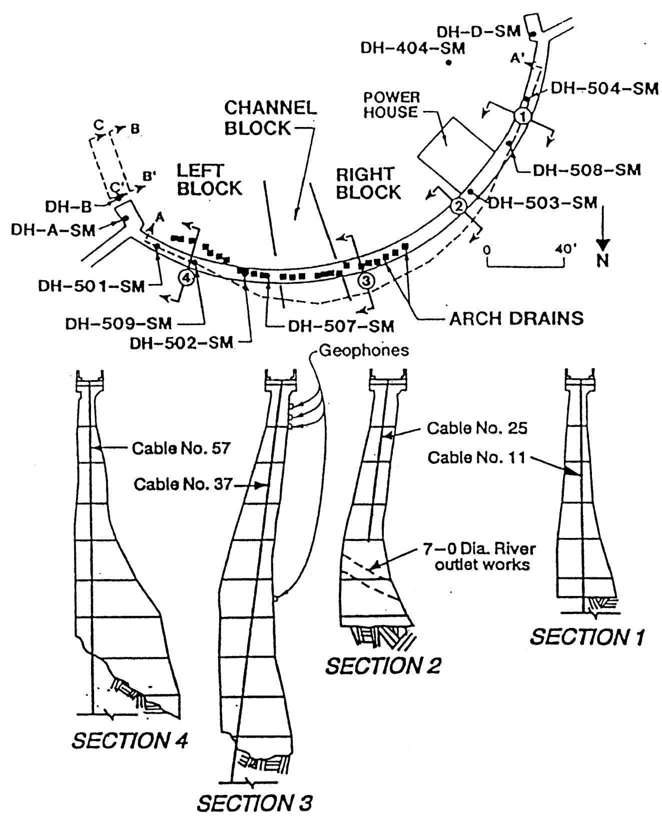

Figure 1. Typical sections through the arch, showing inclination of anchorages, positions of toe drains, and locations of geophones. Stewart Mountain Dam, AZ.

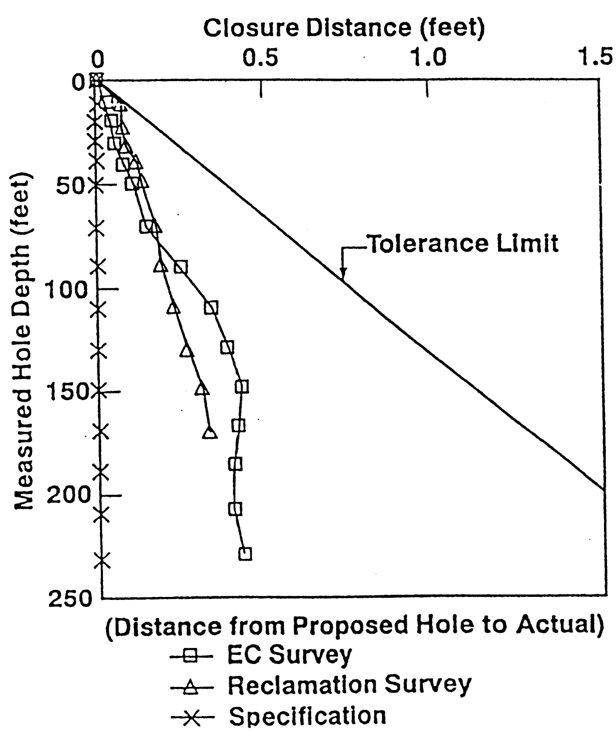

Figure 2. Typical hole deviation data, as monitored in Hole 37, Stewart Mountain Dam, AZ.

across the lift joints indicated tiny movements at most (Table 1) while the geophones indicated equally insignificant peak particle velocities (Figure 3). 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. The impact of the drilling and water testing on local piezometric levels was evident only during the period of these activities. 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 in length. The acceptance criterion was 0.001 gallon/ft/inch diameter/minute at an excess pressure of 5 psi (in the bond length), and trice 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 at an off-site factory and transported in coils with great care. At site, each 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 was exercised to provide the exact bond length. Fluid and set grout properties were rigorously recorded as routine quality control and assurance during construction.

Stressing commenced 14 days after grouting. Nine anchorages were subjected to cyclic performance tests as per Post Tensioning Institute (PTI) Recommendations (1986) 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 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 movement 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.

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

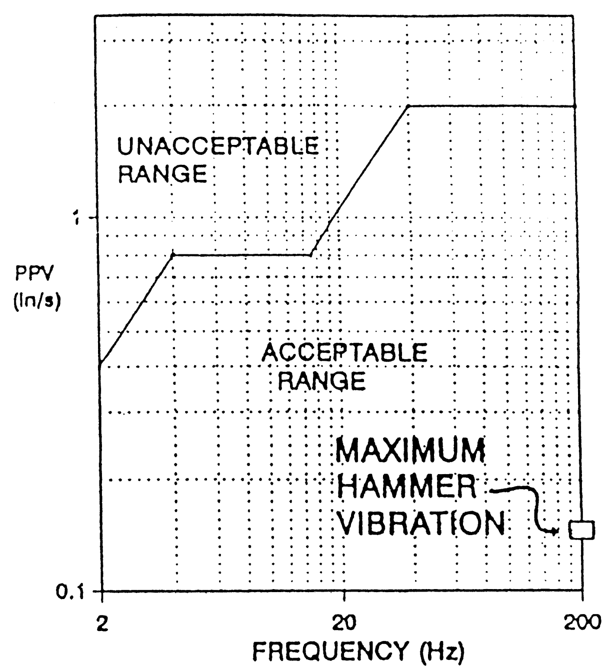

Figure 3. Data from geophone monitoring during down-the-hole drilling through concrete. Stewart Mountain Dam, AZ (Reclamation Acceptability Criterion).

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

2.5 Special Features - As detailed in Section 2.4, there were a number of special features associated with the construction aspects, namely:

- the intense pre-production test anchor program;
- the drilling methods used to ensure hole straightness and direction;
- the borehole deviation measurement techniques
- the investigation of the impact of the drilling process on the structure and its foundation;
- the use of epoxy coated strand
- the sequence of loading the arch
- the monitoring of the dam's performance under load.

However, perhaps the most significant innovative aspect which contributed to the success of this project was the procurement process.

2.6 Procurement Process - 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 budget constraints without contractual dispute or litigation.

Although not officially a "Partnering" project, the works at Stewart Mountain Dam benefited at every turn from professional partnerships developed at all levels of responsibility and participation.

3. BOUNDARY DAM, WASHINGTON

3.1 Dam Description - The Boundary Hydroelectric Project is located on the Pend Oreille River in the northeastern corner of Washington State, approximately one mile south of the U.S.-Canadian border. The project is owned by Seattle City Light, and supplies 1,000 MW of power to the Seattle metropolitan area three hundred fifty miles away.

A major feature of the Project is Boundary Dam, a variable radius double curvature concrete arch dam which was constructed between 1963 and 1967. The Dam stands 360 feet above the deepest part of the foundation, has a crest length of 508 feet (740 feet including the spillways), and has a thickness varying from 8 feet at the crest to 32 feet at the base. It is situated in a deep canyon with steeply sloping abutments on either side. The rock on which the dam is founded is interbedded limestone and dolomite, which is relatively hard but has wide variations in physical characteristics.

3.2 Introduction to Problem - A review board was formed in 1990 at the request of FERC to investigate the safety and stability of the dam. One area of concern identified by the board for further investigation was the stability of two massive wedges of rock located in the cliff at the dam's left abutment, defined by a system of joints and faults. This rock provides support for the spillway chute, as well as providing confinement for the dam foundation which carries the arch loads. It was determined that the rock wedges were potentially unstable during seismic loading, and that application of a prestress force equal to approximately ten percent of the dead weight of the wedges would be required to provide an adequate stability factor of safety.

3.3 Anchor System Design - Initial concepts for the anchor scheme considered either installing the anchors from the face of the cliff and founded into the rock below the fault lines, or installing the anchors up from the dam gallery access tunnel which runs through the cliff behind the wedges, these anchors being founded in the wedges themselves.

The final design completed by Harza Northwest, Inc. Engineers, included installation of six 1,000 kip capacity anchors and seven 500 kip capacity anchors. All of the anchors were to be installed from the gallery access tunnel (Figure 4). All of the anchors had an upward inclination from 19 to 58 degrees above horizontal. The 1,000 kip anchors were designed as double headed anchors (no bond length) due to the end of the anchors being located on or adjacent to the spillway channels, allowing relatively easy access for installation of the outside end bearing plate and anchor head. The 500 kip anchors would exit the rock on the face of the cliff, and were terminated and bonded in the rock wedges in order to avoid work on the cliff face. Total anchor length ranged from 89 feet to 207 feet. The tendons comprised 0.6" diameter strands, 31 for the 1000 kip anchors, and 16 for the 500 kip anchors.

The drilling, water testing, and installation of the grout and tendons in an upwardly inclined anchor presented many construction difficulties. It was proposed by Nicholson Construction Company, the successful low bidder, to install all of the anchors with double headed tendons. Their use improved anchor quality as well as constructability, eliminating the need to provide adequate bond of the anchor into the rock wedge with a mechanical connection.

3.4 Construction - Drilling of the anchor holes was completed using a ten inch diameter down-the-hole hammer advanced with a Davey Kent rotary drill rig. To maintain proper alignment of the drill hole, the drill mast was removed from the drill chassis and mounted on a rigid frame, with the upper end of the mast bolted to the wall of the 24-foot high tunnel at the surveyed anchor point of entry. The alignment criterion for the 1,000 kip anchors exiting in the spillway was 1/2 inch per 10 feet of anchor length. Results of all of the alignment tests, performed after initial drilling, and again after consolidation grouting and redrilling, fell within the specified criteria. A 26-foot long composite rigid stabilizer pipe on which the percussive hammer was mounted allowed further control of the hole alignment.

The length of the completed drill hole and the location of the rock fault line were inspected using a video camera. The holes were then tested for water tightness. The testing showed high water loss in all holes. The required procedure was to plug the bottom of the hole with a packer, fill the hole with water, seal the top of the hole with a packer, then pump water at a pressure of 5 psi while measuring the flow rate for a ten minute period. The specified maximum flow rate was 0.01 gallons per linear foot of hole per inch of hole diameter. This initial water test was completed on only one of the holes, which did not meet the criterion. The remaining holes would not fill up with water, which was pumped in at a rate exceeding 50 gallons per minute. In order to provide a grout tight hole, which allows more reliable grouting and proper corrosion protection of the tendons, the holes were consolidation grouted and redrilled. Water testing performed after redrilling showed results less than the specified maximum flow rate.

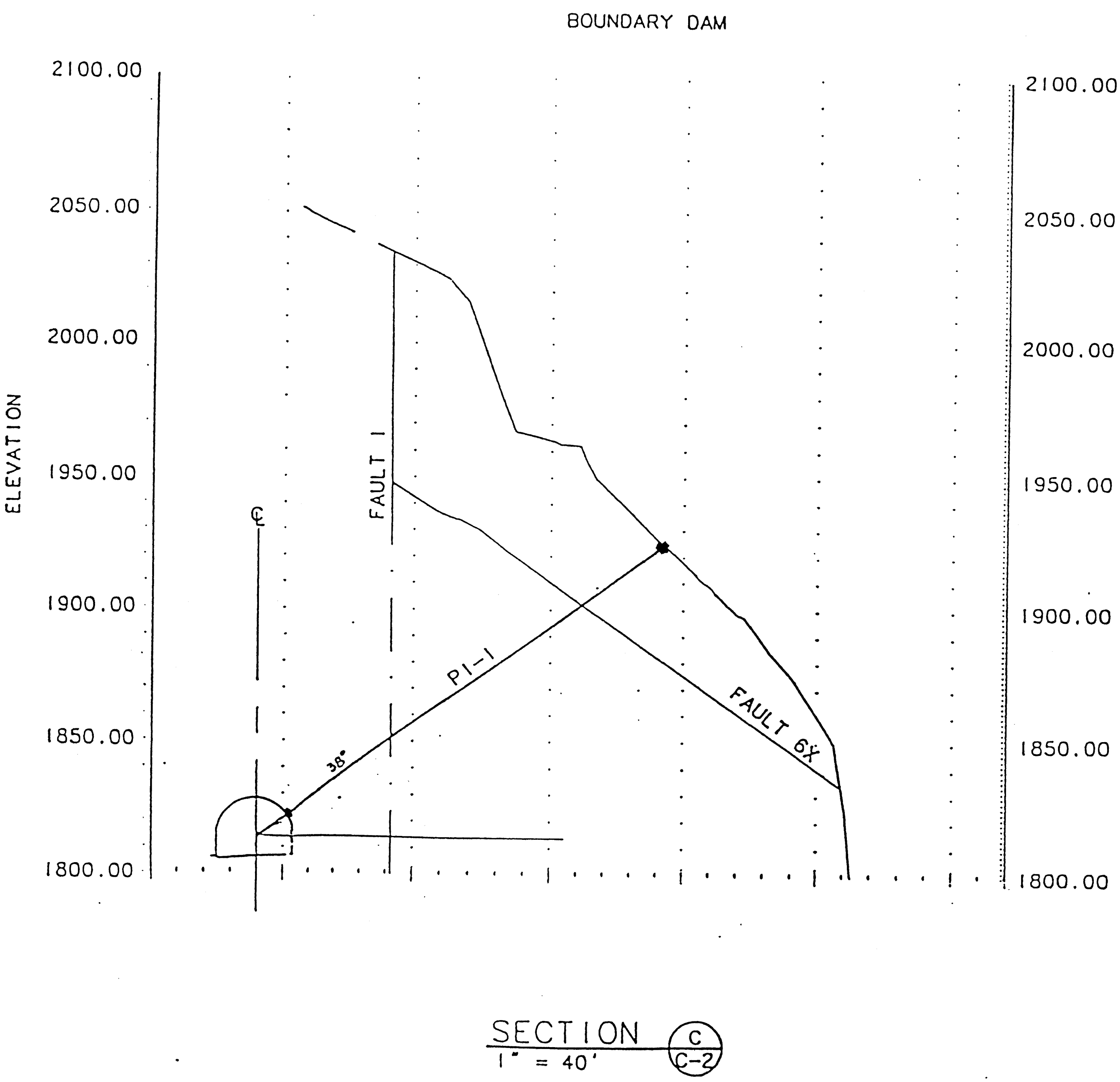

Figure 4. Schematic representation of prestressed anchors. Boundary Dam, WA.

The rock at each end of the anchor was chiseled level to provide a bearing surface for the 18 to 30 inch diameter bearing plates. Bolts were installed on which the plates were mounted and aligned. The void under the plate was filled with non shrink grout. The tendons were inserted into the tunnel end of the anchors and hoisted up with a pulley and cable system hooked to an air powered winch located on the spillway apron above. The full lengths of the tendon strands were individually covered with a grease filled plastic sheath.

As the anchor connections were purely mechanical, conventional anchor performance testing was not required. The required lockoff load was applied individually to each strand using a single strand jack. Elastic elongations of the strands were measured and compared to the theoretical value, and liftoff testing was performed to verify accurate loading of the tendons.

After anchor loading, the holes were backfilled with a stable cement based grout, pumped from the tunnel end of the anchor. A form was placed across the outside of the recess chiseled into the rock and the grout was allowed to flow up out of the anchor to backfill the recess and protect the anchor head. The anchor heads in the tunnel were protected with a grease-filled cap.

Five drain holes were then installed up from the tunnel through the fault to facilitate drainage.

3.5 Special Features - Three full time rock climbers were used for completion of the work on the cliff face two hundred feet above the valley floor. Equipment for this work was limited to hand held tools. The difficulty of the climber's work was greatly increased by the extreme winter weather. The project commenced October 1992 and was completed at the beginning of April 1993. Near record snow fall and wind chill temperatures as low as 20 degrees below zero were experienced. Ice buildup on the cliff face from water flow off the spillway reached thicknesses of 4 feet. Warm air flowing up the anchor shafts helped keep the ice back from the exterior recesses. Despite all the difficulties of access and weather, the projected was completed without experiencing a lost time accident.

3.6 Procurement Process - No special processes were used during procurement. The initial investigation for the safety review board was completed by Morrison Knudsen Engineers. Harza Northwest was the successful respondent to Seattle City Light's Request for proposals on the Project Design. A standard request for proposals was issued for project construction. After submittal of the proposals, bidders under consideration were required to submit a qualification package, which included personnel resumés, list of previously completed projects, equipment and material specifications, and a project schedule, as the final step in the contractor selection process.

4. JOCASSEE DAM, SC

4.1 Dam Description - Duke Power Company's Jocassee Dam, impounds the 7500-acre Lake Jocassee in the northwest corner of South Carolina on the Keowee River, 7 miles north of Salem. The 408-foot high structure is composed of an earth core, with rockfill shells, and can sustain a 30-foot drawdown when utilized in the pumped storage scheme for power generation. The crest length is 1787 feet, and normal pool elevation is at about 1005 feet.

The site is underlain by a variety of gneisses with pegmatite intrusions. The dominant joint set strikes northwest and dips 85° to the northeast.

4.2 Introduction to Problem - Following first reservoir filling in late 1973, a number of springs and seeps were observed on both downstream abutments. By 1976, flows, carrying silt and sand were observed at the C3 weir (Figure 5) about one third of the way down the left abutment. The volume of flow from C3 (at El 986) was directly dependent on the reservoir level in the range between El 1080 and 1110 feet and varied from 0 to 50 gpm. After some interim measures, Duke undertook in October 1991 a full program of action to concentrate on spring C3 in particular. Additional data are provided in Bruce et al., (1993).

4.3 Curtain Design - A straight line projection from C3 towards the dam crest helped locate the area to be grouted and this was confirmed by further coring and dye flow testing. Study of the major joint orientation yielded the most effective orientation for the grout holes (Bearing southwest; dip 30° below vertical). This single row of new holes was designed to intersect the original three-row curtain, but not to intrude into the core material. Their depth was determined as equivalent to El 1050, bearing in mind the relationship between reservoir level and seepage volume.

Investigations had indicated a maximum apparent flow rate of 1 cm/sec, demanding that the grout mix design and placement method had to overcome the possibility of dilution or washout prior to setting.

4.4 Construction - The work totaled 10 Primary holes, spaced at 10 foot centers, and 6 Secondary holes concentrated near the embankment. Since the seepage was felt to be restricted to well defined open channels in the weathered bedrock, (as opposed to a myriad of fine fissures) a 4 1/2 inch diameter down-the-hole hammer with air flush was used to drill each hole to dull depth. Permeability tests were carried out in stages of 27 to 33 feet length in each hole, using the multipressure Houlsby (1976) method to a maximum excess head of 38 psi.

The variation of the resultant Lugeon values over the course of each stage test provided insight into the nature of the fissure geometries and characteristics, and the initial grout mix for each stage was selected on the basis of this information. Certain stages could not be water tested due to the difficulty of packer seating in very soft/weathered horizons.

A suite of grout mixes was derived from an on-site test program which assessed stability, flowability, specific gravity, and set time (Table 2). The Type I cement grouts were judged most appropriate for the Primaries, while the finer grained Type III, and microfine grouts were used in the Secondaries. Applied grouting pressures were typically limited to 1 psi/ft of depth for the Primaries, and 1 1/2 psi/ft for Secondaries.

4.5 Analysis and Performance of the Grouting - The decision to terminate the work after installing the six Secondaries in the most critical area was made with respect to grout take characteristics (including Reduction Ratio comparisons) and hydrogeological impacts. Water tests in Primary holes confirmed high Lugeon values with Turbulent Flow characteristics (large fissures). A total of 20,449 gallons of grout was injected at an average take of 196 pounds/foot. Secondary permeabilities were substantially lower and many exhibited Laminar Flow characteristics, indicative of small fissures. Their average take was 63 pounds/foot, therefore yielding a very satisfactory Reduction Ration

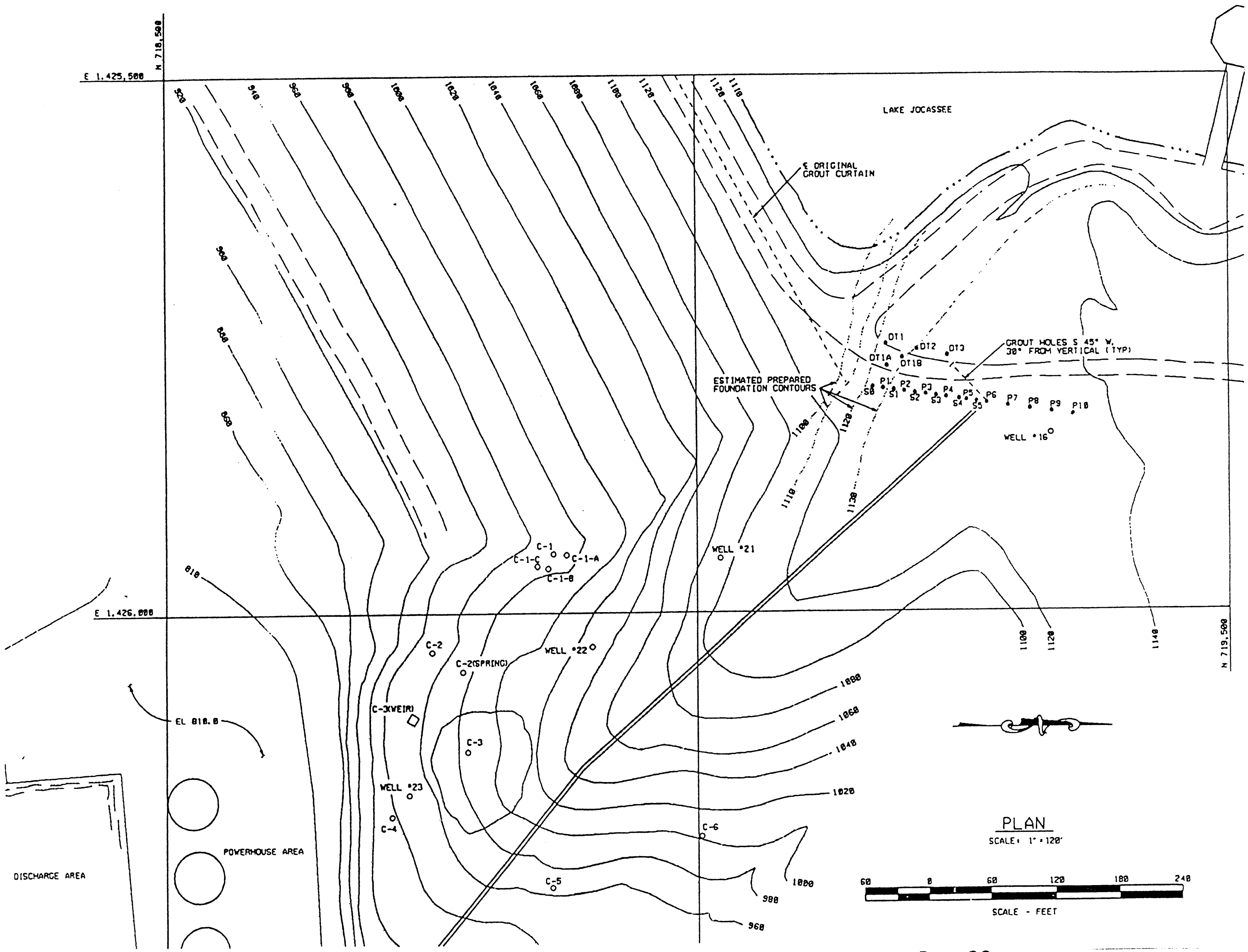

Figure 5. Location of grout holes, Dye Test holes, and springs, Jocassee Dam, SC.

Mix #	Water (Pounds)	Bentonite (Pounds)	Cement (Pounds)	Stiffening (Hours)	Hardening (Hours)	S.g (Meas.)	Marsh Cone (Secs)	Final Bleed (%)
B	166	2 1/2	94	6	8+	1.31	40	23
C	166	4	94	5 1/2	7 1/2+	1.34	43	17
D	166	8	94	5	7+	1.35	49	6
H	166	10	94	5	7+	1.36	54	4
E	166	4	188	3-4	4-5	1.56	50	3
F	166	8	188	3-4	4-5	1.57	65	2
G	166	4	282	3	4	1.73	75+	1

Table 2. Summary of Type I Cement Grout Mixes, Jocassee Dam, SC.

Note:
(1) Water and bentonite mixed for 30 seconds prior to adding cement; samples taken 60 seconds late
(2) For microfine grout, the mix design was 26 gallons/44 lb bag (W/C = 5.0 by weight), + 7 oz. dispersant
(3) Air temp 55°F during testing.

of 63/196 i.e. 32%. Regarding the hydraulic indicators of grouting effectiveness upon completion:

- the nearest piezometer showed a significant increase in head;
- spring flows from all sources in the vicinity were reduced from 107 gpm to just over 30 gpm. (Figure 6). No transported sediment has been noted;
- dye testing confirmed far less direct hydraulic connections across the curtain.

These results were judged sufficiently satisfactory to avoid further deepening, lengthening or intensifying of the curtain.

4.6 Special Features - "Responsive Integration" - The use of grouting techniques for existing dam remediation has not always met with the expected degree of success (Weaver, 1991; Bruce, 1992). Reasons have included:

- Unsuitability of methods: including drilling technique, type of grout staging, grout injection method, geometry of holes, grout pressures, and the use, location and types of packers.
- Unsuitability of materials: physical and chemical properties of components (principally water, cement, fillers, additives and bentonite for cement based grouts, and water, base, reagent and accelerator for chemical grouts), proportioning, rheology, stability, and resistance to physical and chemical attack.
- Lack of critical expertise from the contractor: such as adaptability/flexibility to changing conditions, prior experience, local knowledge, ability to partner with the owner, technical competence, quality of documentation, and understanding of requirements necessary for administering the work.
- Lack of Quality Control and progressive monitoring ability: methods, means and resources to monitor, correct documentation, and timeliness and quality of result evaluation.

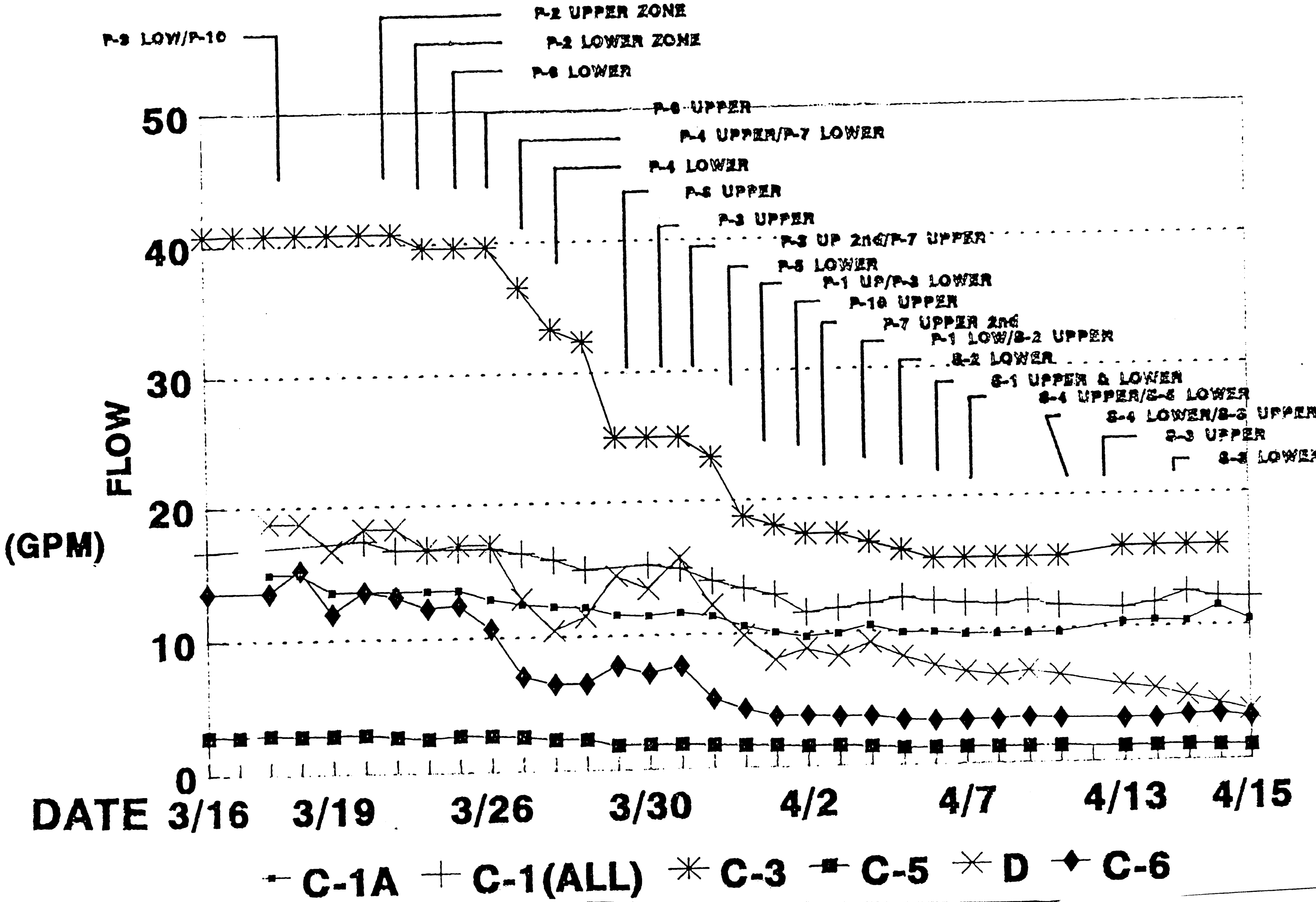

Figure 6. Spring flows measured during grouting (Reservoir level relatively constant between 1107.4 and 1109.0 feet. Elevations of flow points: C1 = 980'; C3 = 986'; C5 = 956'; C6 = 1000'; D = 1000'), Jocassee Dam, SC.

- Overly restrictive, outmoded specifications: which give little opportunity for modifying methods or materials during the course of the work, in response to what is progressively revealed.
- Procurement practices which foster contractual disagreements and which incite claims and litigation: a clear understanding of both the owner's and the contractor's needs is essential, but rare, whereas it is most common to find a) unit pricing based on estimated quantities which may not be accurate, and b) factors which force the contractor to build in large overhead charges or to inflate unit prices for unrealistic "what if" circumstances feared by the owner.

As outlined previously, this project was not extensive in scope, but was technically challenging. It was perfectly suited to the concept of "Responsive Integration"sm recently crystallized by Nicholson Construction. The essence of the concept is for all parties, in active partnership, to evaluate data from all relevant sources, both historical and contemporary. The conclusions of this repetitive integration are then used to direct the progress of the work in the field, in the way which is most responsive to the actual conditions, to secure the most cost effective technical benefits.

Now, whereas this may seem a logical, simple and wholly desirable process, many of the negative factors listed above have usually acted to frustrate the efficacy of the individual processes and have therefore adversely impacted the quality of the resulting work.

In order to effect this success, several conditions within the owner-contractor relationship must exist:

- There must be mutual respect for each other's abilities, and the attributes which each party brings to the job.
- Specifications should be structured so as to be non-adversarial.
- There must be mutual desire for success, in terms of objectives, cost, environment, and so on.
- The parties must have the ability to respond quickly to changes in conditions or directions.
- There must be harmonious coordination and optimization of each process towards attaining a quality production.

The contractor should know and understand the owner's objectives, know the owner's abilities, understand the owner's share and limits of risk, and understand the owner's desire to control costs. Likewise, the owner should understand the contractor's abilities, expertise, and degree of flexibility, and should acknowledge the contractor's share of the risk, and his desire to make a reasonable profit.

The possibility that the benefits of "Responsive Integration" could be used at Jocassee Dam was initially promoted by the owner's attitude towards the method of bid solicitation and contract award, as detailed in Section 4.7.

4.7 Procurement Process - Duke Power wished to allow bidders maximum flexibility in proposing and utilizing processes they felt most effective and economical, while still meeting the owner's criteria for stage grouting and controlled grout travel, stability and penetrability. The owner did not therefore specify the drilling technique, drilling equipment, grout, grout equipment or injection method. The contract award was determined by the owner's evaluation of anticipated effectiveness and economy, and with regard to employees' health and safety.

A "menu type" bid form was accompanied by a detailed questionnaire regarding the bidders' method statement, program, experience, workforce references, and environmental protection plan. As a basis for financially comparing the different bids, the foreseen major quantities of work items were listed.

Afterward, and prior to commencement of work, the owner and contractor met to discuss specific details of the contractor's processes, equipment schedule, contract administration, facilities, requirements and so on, and to make final revisions to the specification and procedure, based upon the selected drilling and grouting process. The agreed specification and procedure were then "Released for Construction."

Nicholson Construction was awarded the contract largely on the basis of their technical expertise and especially their proposal to use relatively inexpensive cement-bentonite grouts. The advantages of using these mixes were that they were easy to mix and pumpable at low water-cement ratios, they comprised readily available materials, were controllable, could penetrate finer cracks, possessed moderate strength, and were relatively stable and dilution resistant. Provision was made for the addition of sodium-silicate, if required for flowing water or for ultra fast set times, and for the use of Type III and microfine cements, to encourage penetration of finer fissures, as described above.

5. HORSE MESA DAM, ARIZONA

5.1 Dam Description - Horse Mesa Dam is a concrete arch structure on the Salt River, located about 60 miles northeast of Phoenix, Arizona. It was constructed during the years 1924 to 1927. A 30-foot diameter tunnel exiting the canyon wall at the dam's right abutment was constructed in 1937, increasing the dam's discharge capacity from 103,500 to 150,000 cfs.

Two access roads were constructed on the left downstream side of the dam to facilitate operation, maintenance, and repairs. The upper access road is graded into the canyon slope, and provides access to the top of the dam. The lower roadway is constructed on alluvial and talus deposits on the river bank at elevations ranging from a few feet to 30 feet above river level. The lower access road was judged to be incapable of withstanding significant tunnel discharge volumes from the dam, and it was considered more cost effective to rebuild than to reinforce the roadway.

5.2 Introduction to the Problem - During the period from 1968 to 1983 the roadways were damaged to varying degrees six times from discharge volumes ranging from 3,000 to 70,000 cfs. Typical repair measures consisted of cleaning of the river channel, rebuilding the lower road with large rock material, and regrading the upper roadway. A large retaining wall consisting of a lower tieback anchored concrete wall with a Hilfiker welded wire reinforced wall on top was constructed in 1984 on the upslope side of the lower road, which allowed regrading and realignment of the upper road. The lower road was also rebuilt at this time. With construction of this large wall, the importance of the lower road stability was greatly increased.

5.3 Initial Test Grouting Program - Efforts to safeguard the lower roadway from damage caused by future discharges commenced in late 1990 with the issuance of a request for proposals (RFP) for a test grouting program. The intent of the program was to test the feasibility of binding the boulder and fragmented rock roadway base material into a stable mass by using cement based grouts. The main difficulties initially considered were the potential for easy flow of the grout through the voids between the rocks into the river and/or away from the area intended for grouting, and the difficulty of guaranteeing adequate grout penetration into zones where compacted rock debris, sand, and silt sized particles were present. The RFP announced the basic program concept, and required the bidding contractors to propose the specific details of the grouting program.

Nicholson Construction was the successful bidder and executed the test grouting program. The program consisted of grouting a "containment curtain" around a 15 foot by 25 foot area and then grouting the volume inside the curtain. A sand-cement grout was to be used initially. In the case of excess grout quantities, an increased sand content and/or the use of plastic fibers and dosium silicate reagent was to be used to limit travel of the grout.

However, the results of the test program showed that the voids present between the rocks were not as uniformly large or interconnected as initially expected, and so it was not possible to economically place a sufficient quantity of grout to uniformly solidify the roadway bed.

5.4 Pile Wall Design - An in-situ wall concept to provide a positive mechanical method to protect the road was thereafter proposed by Nicholson Construction. The design concept consisted of small diameter drilled and grouted piles (Bruce, 1989 b.) which were embedded into the bedrock and reinforced with a small wide flange steel beam. The design was finalized by the Owner's Consultant, Woodward Clyde & Associates, and ultimately consisted of one or two rows of vertical 12 inch diameter piles spaced at 12 inches on center, with every other pile staggered back 7.5 inches (Figure 7). The piles were embedded a minimum of 5 feet into the bedrock layer, and were backfilled with a sand/cement grout which was "reinforced" with steel fibers. W6 x 25 steel beams were used for the pile reinforcement. An 18 inch thick reinforced concrete pile cap was installed to tie the piles together. The wall was a total of 425 feet in length, and protected the roadway in front of the anchored wall upstream towards the dam. A total of 610 piles totaling over 14,000 linear feet was installed.

5.5 Pile Wall Construction - The piles were drilled with track mounted, diesel powered hydraulic rotary drill rigs of both Nicholson and Casagrande manufacture. The piles were installed by first drilling down to the bedrock layer with a 12 inch diameter down-the-hole hammer. The hole was stabilized by the insertion of a temporary steel casing. The pile embedment socket was then drilled to full depth with a 9 inch diameter down-the-hole hammer. The steel wide flange pile was then inserted, the hole backfilled with grout, and the temporary casing extracted.

The grout was batched on site by first placing sand aggregate and steel fibers into the drum of the concrete transit truck. The cement and water were mixed separately in a grout batch plant and added to the fibers and aggregate in the drum for final mixing. Concrete materials for the pile cap was delivered dry to the site from a redimix plant. Water was added and the concrete mixed on site. A target unconfined compressive strength of 4000 psi was set.

5.6 Special Features - Several features contributed greatly to the difficulty of the job, with the most important being the very difficult drilling conditions. The shifting of the boulders created problems such as "binding" of the down-the-hole hammer, and difficulty with insertion and extraction of the temporary casing.

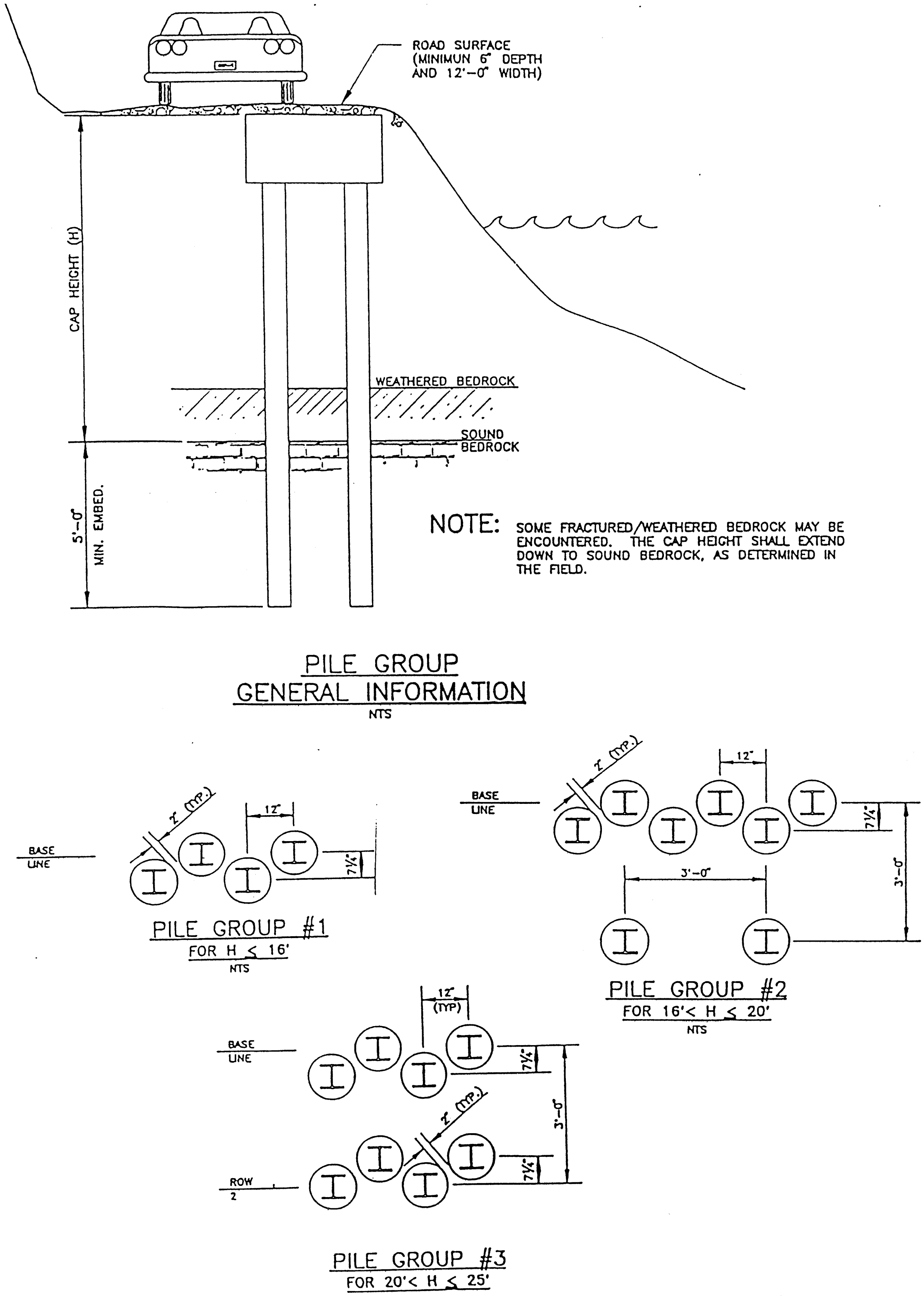

Figure 7. Pile Group: General Information, Horse Mesa Dam, AZ.

A second feature was the remoteness of the project site. Travel time to the nearest town was approximately 1.5 hours. The closest a semi truck could come to the dam for delivery of materials and equipment was a "half hour up the hill." Materials, fuel, and small equipment had to be shuttled in with smaller trucks as it was needed.

The allowed time duration for completion of the job also added to the construction difficulties. Construction was not allowed during the nesting period of a pair of bald eagles which lived near the dam. The work was done on a three shift per day basis to complete a majority of the wall installation prior to this "eagle shutdown" period.

The wall design concept and construction quality were tested during the very wet spring of 1993, during which extremely high dam discharge volumes were experienced. the heavy rains and high water discharge caused sloughing on the slopes above, washout of the lower access road upstream from the pile wall, and washout of the material in front of the pile wall. However, the pile wall and the adjacent anchored and reinforced earth wall remained intact.

5.7 Procurement Process - Completion of the project design and determination of the methods of construction featured a partnership between owner, consultant and contractor. This approach was established initially by issuance of the RFP for the test grout program as a design/build proposal. Cost was not the only or even the most important award determinant.

Thereafter, a full and open review of the effectiveness of the test grouting program was conducted by the partners. When it was concluded that other solutions were more likely to satisfy the Owner's requirements, the contractor was permitted to investigate and price various options. The small diameter pile wall proved to be the most cost effective technically acceptable choice, and a contract was negotiated on that basis.

6. FINAL REMARKS

The four case histories described in this paper illustrate innovative solutions to a wide range of technical and logistical problems. However a common thread is the way in which innovative contractual and procurement vehicles have been used by the Owners. These vehicles have encouraged full and open partnership and cooperation between the Owner and the Contractor so that the highest possible quality has been achieved in a non-litigious, cost-effective manner. As the needs of the dam remediation market become more demanding, so specialty contractors will have to continue to research and develop innovative solutions. The successful application of these solutions will depend largely on parallel advances in bid procurement and contract administration processes.

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