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SYNOPSIS: Painted Rock Dam, a 181 ft (55.2 m) high, zoned earth embankment detention structure near Gila Bend, Arizona, retained approximately 2.3 million acre ft (2.8 x 10^9 m^3) of water for an extended period during the winter of 1978-1979.

Several areas of seepage developed over major portions of the downstream valley and in a minor area on the right abutment. These areas were monitored while instrumentation was installed and investigations made to determine the nature of the seepage and provide the basis for evaluation of structural performance and design of necessary relief and control measures.

Construction of seepage relief and control measures and stabilizing berms was accomplished in the fall of 1979. Performance of the project has been evaluated during subsequent storage periods through 1986, with the conclusion that the control measures are functioning satisfactorily, and in general accordance with design concepts. Site conditions, original design and construction, seepage observations, instrumentation, investigations, evaluations, and design and construction of remedial measures are summarized and lessons learned are presented.

PROJECT DESCRIPTION

Painted Rock Dam is a compacted, zoned earth embankment, flood control structure located on the Gila River about 22 miles (35.4 k) northwest of the Gila Bend, Arizona. Construction of the project was initiated in 1957 and was completed in November 1959, Figure 1. The length of the dam is 4796 ft (1461.8 m), with a maximum height of 181 ft (55.2 m) (Elev 705 ft msl) above the streambed and the crest width is 20 ft (6.09 m). The 25 ft (7.6 m) diameter gated outlet structure has a discharge capacity of 30,500 cfs (863.8 m^3/s), while the 610 ft (185.9 m) wide, ungated, broadcrested weir, rock spillway has a discharge capacity of 405,000 cfs (11,469.6 m^3/s). The reservoir capacity is 2,491,700 acre ft (3.1 x 10^9 m^3) at spillway crest (Elev 661) and 4,825,000 acre ft (5.9 x 10^9 m^3) at maximum water surface (Elev 696.4).

The authorized operation plan for Painted Rock Dam was established to reduce peak flood flows by retaining water for only short durations (several months at most). During nonflood periods the reservoir was intended to be dry. Unfortunately, the unimproved downstream channel of the Gila River has a relatively small, nondamaging flow capacity, approximately 5,000 cfs (141.6 m^3/s). To prevent damages and hardship, releases have been considerably less than the original design intended.

Site Conditions

The geologic setting of the site is volcanic, with alluvial deposits in valley sections, Figure 2. The complex foundation is composed of several rock units, and complicated by faulting. Three faults were exposed in the main stream channels and two on the left abutment saddle, with shear zones existing in all units at various locations across the site.

The pink rhyolite is generally hard and massive with steeply dipping joints oriented in an intersecting pattern near north-south and east-west. Most of the joints along the dam axis are relatively tight and spaced an average of about one foot apart. However, several open, near-horizontal fractures were noted in the upper portion of the left abutment saddle. The tuff and agglomerate units vary from porous and friable to moderately hard and dense, and are generally massive with occasional vertical joints. The rhyolitic andesite is hard and dense, but deeply weathered where vertical flow bands intersect the surface, with a contorted, platey structure due to flow banding. Joints are more widely spaced than in the rhyolite, and tend to strike north-south with near vertical dips. These joints are especially prominent in the left stream channel and the adjoining steep left abutment ridge.

The alluvium consists of recent and older units with the recent deposits occurring only in the Gila River streambed, where it was 60 ft (18.3 m) deep in the right channel and 80 ft (24.4 m) deep in the left channel at the dam axis. Under the dam the alluvium consists of clean coarse material, including sand, gravel and cobbles with irregular lenses of open gravel, which become more coarse with depth.

The older alluvium is composed primarily of terrace deposits containing silts, sands, gravels and talus, and occurs only in the left abutment saddle. In the central portion of the left abutment saddle the older alluvium was sufficiently cemented with caliche to be classified as rock.

Design

In light of the short duration of planned storage, it was not felt that steady-state seepage conditions would be fully developed during project operation. Consistent with contemporary practice and the purpose of the project, the only instrumentation designed into the project were surface settlement monuments.

To accommodate the site conditions it was decided to

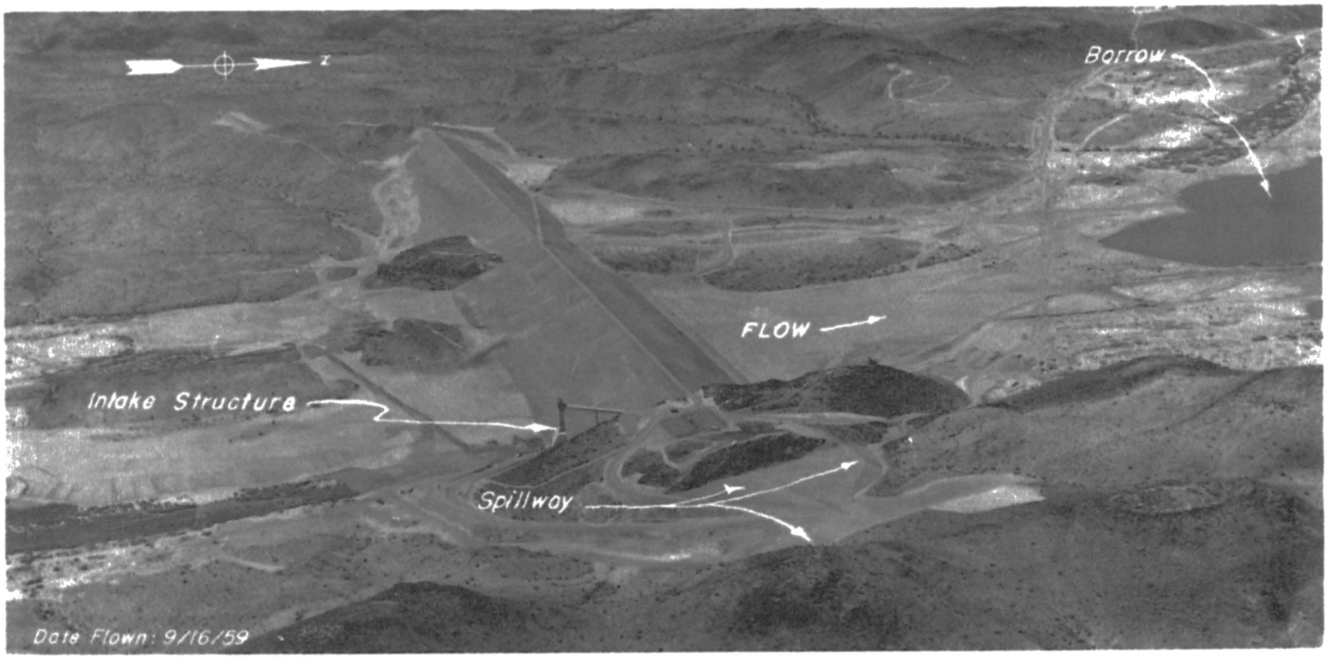


Figure 1. Aerial view of Painted Rock Dam as seen from approximately 1/2 mile (0.8 k) east of the right abutment.

carry the impervious and transition zones through the pervious alluvium to rock, Figure 3. The pervious shells were to be founded on prepared alluvium. Materials for all zones, except the impervious, were to come from a downstream borrow area, while the source for impervious materials was approximately 2 miles (3.2 k) upstream of the dam. Foundation preparation and treatment was performed in accordance with the state of practice at the time. A single grout line was used to provide for a final indication of the quality of the foundation rock, with a split-spacing process and expansion of lines planned if considered necessary during construction.

Construction

Construction proceeded largely as envisioned during the design. Excavation of the cutoff trench was in the dry, but dewatering measures were required in the recent alluvium of the left and right stream channels, including the installation of gravel filled collection trenches in the rock immediately upstream and downstream of the impervious zone in several areas.

Several borings and exploration trenches were made during construction to confirm the condition of the foundation and the extent of treatment required in local areas. The single grout line was constructed beneath the impervious zone, approximately along the centerline of the section, with an average take of 0.3 sacks per linear ft (0.3 m) in holes ranging from 10 to 60 ft (3 to 18.3 m) in depth. Early in the program borings were made to evaluate grout penetration, which was found to be good. One grout hole in the right abutment (Sta 16+95) took 173 sacks with no surface leaks, and was bracketed with additional holes to attain closure.

Rock in the cutoff trench and the abutments was prepared for fill placement by removing all loose and objectionable material to obtain suitable foundation conditions. Vertical and overhanging rock ledges were removed from the abutments and mid-valley rock ridge. Dental concrete was required only in scour channels and pockets in the bedrock of the left stream channel cutoff trench.

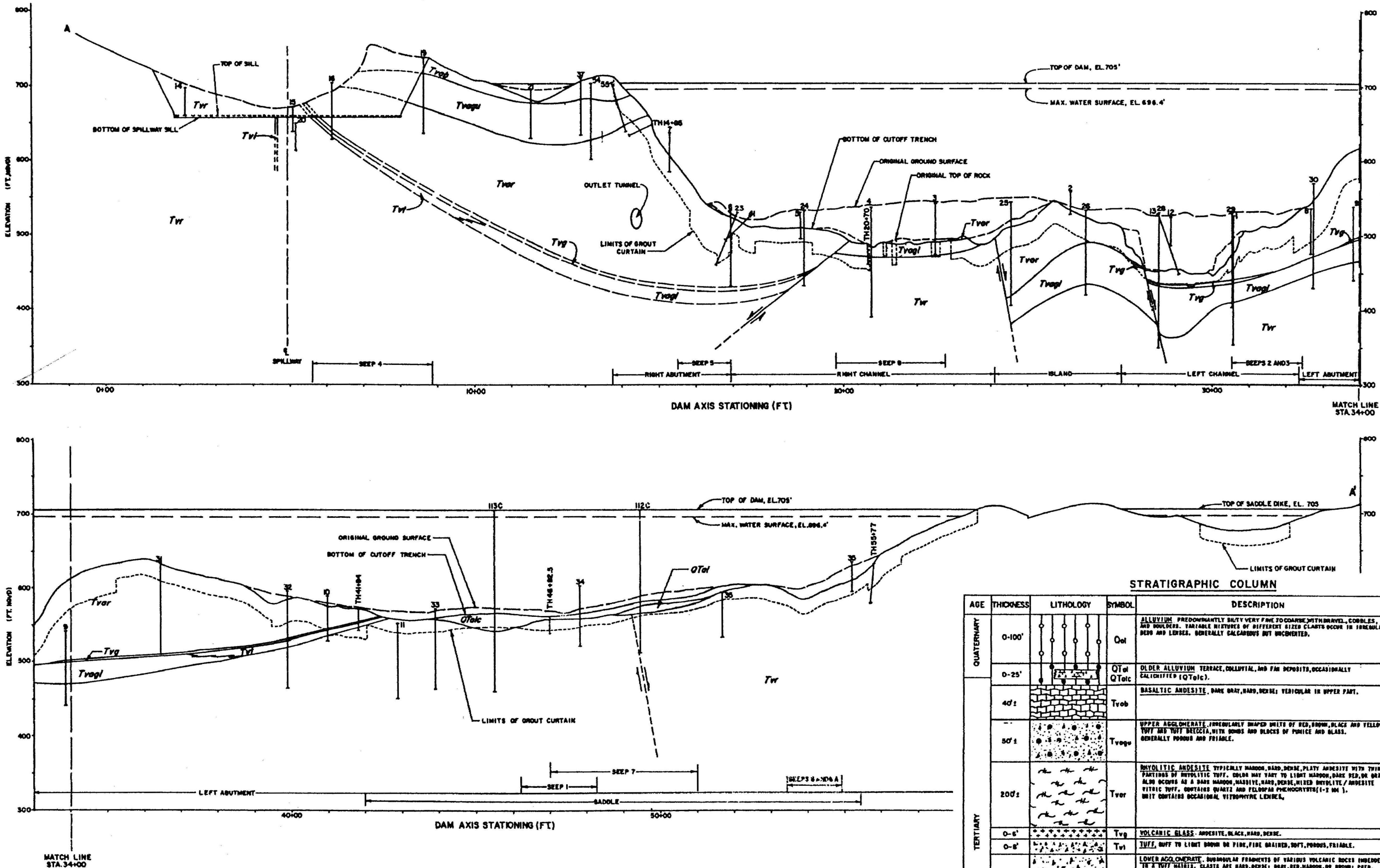
The alluvium was prepared for embankment placement by removing unsuitable materials in the sidewalls of the cutoff trench and on the surface underlying the embankment. Stripping to a depth of several feet was required, with excavation to the existing water table elevation at several locations in the stream channels. The alluvium was proof rolled prior to placing fill.

The moisture content of fill materials was adjusted by prewetting and mixing in the borrow areas. These materials were hauled and placed in accordance with the specifications, although rapid surface drying of the impervious materials due to summer sun and wind necessitated a change of procedures. The resulting construction met specification requirements with regard to both moisture content and material gradation.

THE OCCURRENCE OF SEEPAGE

After a prolonged period of water storage at about Elev 583, on December 20, 1978, reservoir inflows of up to 70,000 cfs (1,982.4 m^3/s) caused a rapid rise of reservoir surface elevation, which reached a record high of Elev 613 on December 26, 1978. On January 2 and 8, 1979, the dam operations staff noticed isolated seeps in the valley downstream of the toe, Figure 4. As seeps occurred they were numbered sequentially and monitored for quantity and sediment. Visual monitoring of the seeps was intensified, and on about Juanuary 19, 1979, the reservoir began rising again. Seepage rates increased with almost no time lag and four additional seeps developed over the next two weeks, Table 1. During this period foot patrols and monitoring efforts were increased using additional staff temporarily assigned to the project. The reservoir reached Elev 634.6 on February 7, and subsequently dropped and then raised to the maximum peak of Elev 642.3 on April 17, 1979.

Since the early seepage carried no sediment, it was



STRATIGRAPHIC COLUMN

AGE	THICKNESS	LITHOLOGY	SYMBOL	DESCRIPTION	FDN RPT 10/80
QUATERNARY	0-100'		Qal	ALLUVIUM PREDOMINANTLY SILTY VERY FINE TO COARSE, WITH GRAVEL, COBBLES, AND BOULDERS. VARIABLE MIXTURES OF DIFFERENT SIZED CLASTS OCCUR IN IRREGULAR BEDS AND LENSES. GENERALLY CALCAREOUS BUT UNCEMENTED.	Qal RECENT ALLUVIUM
	0-25'		QTal QTalc	OLDER ALLUVIUM TERRACE, COLLUVIAL, AND FAN DEPOSITS, OCCASIONALLY CALICHIFIED (QTalc).	QT OLDER ALLUVIUM
TERTIARY	40'±		Tvob	BASALTIC ANDESITE, DARK GRAY, HARD, DENSE; VESICULAR IN UPPER PART.	QTv BASALT
	50'±		Tvagu	UPPER AGGLOMERATE, IRREGULARLY SHAPED UNITS OF RED, BROWN, BLACK AND YELLOW TUFF AND TUFF BRECCIA, WITH BOMBS AND BLOCKS OF PUMICE AND GLASS. GENERALLY POROUS AND FRIABLE.	
	200'±		Tvar	RHYOLITIC ANDESITE TYPICALLY MAROON, HARD, DENSE, PLATY ANDESITE WITH THIN PARTINGS OF RHYOLITIC TUFF. COLOR MAY VARY TO LIGHT MAROON, DARK RED, OR GRAY. ALSO OCCURS AS A DARK MAROON, MASSIVE, HARD, DENSE, MIXED RHYOLITE / ANDESITE VITRIC TUFF. CONTAINS QUARTZ AND FELDSPAR PHENOCRYSTS (1-2 MM). UNIT CONTAINS OCCASIONAL VITROPHYRE LENSES.	Tv FELSITE
	0-6'		Tvg	VOLCANIC GLASS - ANDESITE, BLACK, HARD, DENSE.	Tkl
	0-8'		Tvt	TUFF, BUFF TO LIGHT BROWN OR PINK, FINE GRAINED, SOFT, POROUS, FRIABLE.	
	0-60'		Tvagl	LOWER AGGLOMERATE. SUBANGULAR FRAGMENTS OF VARIOUS VOLCANIC ROCKS IMBEDDED IN A TUFF MATRIX. CLASTS ARE HARD, DENSE; GRAY, RED, MAROON, OR BROWN; PRED. RHYOLITE AND ANDESITE. MATRIX IS HARD TO SOFT; BUFF, GRAY, GREEN, WHITE OR YELLOW. MAY CONTAIN VITROPHYRE LENSES.	TUFF AND AGGLOMERATE
	300'+		Tvr	RHYOLITE, RED ORANGE TO RED MAROON, PORPHYRITIC, HARD, DENSE, MASSIVE WELDED TUFF. PHENOCRYSTS ARE 1-3MM, PREDOMINANTLY FELDSPAR AND HORNBLENDE. MAY EXHIBIT FLOW / COMPACTION STRUCTURE WITH ELONGATED ORANGE-BROWN TUFF OR PUMICE FRAGMENTS. CONTAINS OCCASIONAL VITROPHYRE OR UNWELDED ZONES.	Tkv RHYOLITE

ADDITIONAL UNITS

Tvi INTRUSIVES ANDESITE, DARK GRAY, HARD, DENSE, SLIGHTLY VESICULAR. OCCURS IN DIKES GENERALLY 5 FEET ± THICK. OBSERVED ONLY IN Tvr, BUT MAY PENETRATE UPPER UNITS.

FG FAULT GOUGE TYPICALLY SLICKENSIDED CLAY; COLORS MAY BE BLUE, GRAY, PURPLE, MAROON, PINK, ORANGE, OR REDDISH BROWN. CONSISTENCIES VARY FROM SOFT TO STIFF AND BRITTLE. MAY CONTAIN ROCK FRAGMENTS. FAULTS HAVE BEEN OBSERVED IN Tvar THRU Tvr UNITS.

Figure 2. Cross-valley section of the geology along the embankment centerline.

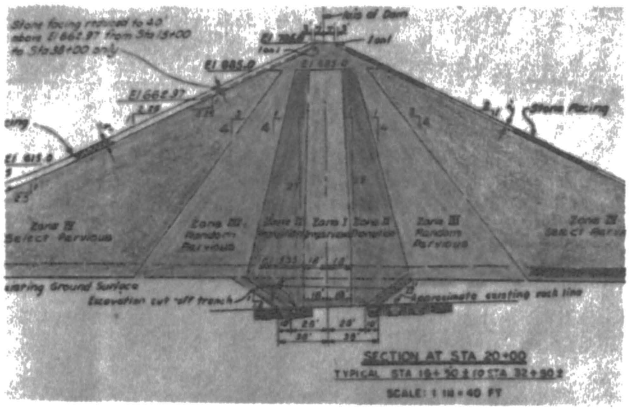


Figure 3. Cross-section of dam at Sta 20+00.

decided to leave the areas free of any temporary relief or control measures so that visual monitoring of the exit conditions could be maintained while investigations and studies were carried out to determine the physical basis for the seepage and provide a rationale for the design of permanent relief and control measures. This decision did not preclude the installation of temporary measures, and arrangements were made for quick availability of construction personnel, equipment and materials, should they have been needed. As the reservoir reached the higher levels, Seeps 1 and 2 were blanketed with gravel wrapped in filter cloth and weighted with a surcharge berm as a preventive measure.

INVESTIGATIONS AND STUDIES

Due to the dispersed seeps and the apparent complexity of the situation, it was decided to execute a comprehensive program of complementary investigations. The possible urgency of the situation dictated that the most expedient methods be used as quickly as possible, to be followed by programs requiring borings or more time consuming installations. Table 2 summarizes the types of investigations, while Figure 5 shows locations of instruments and surveys. Several of the investigations occurred concurrently in the January-February time period, and some were completed prior to the high water conditions in early February. These results contributed to decisions governing operations made during that period.

Seepage Collection and Measurement

As soon as seeps developed the water was collected at points where monitoring of quantity and sediment could occur. An attempt was made to maintain this capability throughout the study period, although later activities necessitated combining several flows. Temperature readings were taken and, later, samples for chemical analysis obtained. This effort contributed to the early decision process and provided continuous information on response times to reservoir changes and piping potential.

Observations Wells

Corps of Engineer staff quickly installed a total of 22 well points in the valley alluvium and fill downstream of the toe. Eleven of these were placed in the general area of Seep 1, between Sta 43+75 and Sta 49+80. The balance were placed in the vicinity of Seeps 3, 4 and 5, between Sta 6+00 and Sta 30+00. In addition to observing water levels, these wells were also pumped to gain an indication of permeability.

Electrical Resistivity

During the period of January 19 to 26, 1979, electrical resistivity surveys were performed in the vicinity of Seeps 1, 2 and 3. The use of Wenner and Bristow arrays resulted in interpreted seepage paths under the dam associated with the rock ridge near Seeps 2 and 3, and in the foundation near Seep 1, where several shallow seepage paths appear to converge.

Geothermal Monitoring

Thermistors were installed in 10 ft (3 m) deep holes as follows: (1) 20 along the crest of the dam at 200 ft (61 m) intervals, (2) 13 along the lower access road at 100 ft (30.5 m) intervals, and (3) 12 along the access ramp at 100 ft (30.5 m) intervals. They were allowed to stabilize, and readings were initially taken on January 23, 1979. Three other sets of readings were made between January 26 and February 4, 1979. Interpretation of the temperature anomolies resulted in identification of six zones of potential seepage, Figure 5. Temperature surveys, accomplished in the deep borings made for piezometer installations, indicated the seepage paths to be in the foundation rock.

Aerial Imagery

On January 28, 1979, two flights were made, at 0755 and 1040 hours, to obtain color, color infrared and thermal infrared images. The intent was to take advantage of thermal differentials to confirm the location of seeps already discovered, possibly locate seeps as yet undiscovered by the extensive foot patrols, and to extend the area of coverage beyond that possible by foot. Temperatures of Seeps 1 and 2 were determined to be 6 to 8° F higher than the reservoir, while the balance of the detected areas were nearly the same temperature as the reservoir. No seeps were discovered outside of the immediate region of the dam. Existing seeps were largely confirmed and two seeps, Seeps 7 and 8, were discovered before surface seepage was observed.

Refraction Seismic

Refraction seismic surveys were performed during the period of February 2 to 8, 1979. The objective was the expedient profiling of subsurface conditions to use in the evaluation of seepage and the selection and design of relief and control measures. In particular, preliminary design of the relief well systems was based on these results, confirmed by some boring data.

Chemical Analyses

On April 20, 1979, a long term program of measuring reservoir temperature and water chemistry (dissolved solids) was initiated for comparison with concurrrent evaluations of seepage water temperature and chemistry. In the seeps a broad range of total dissolved solids, 2,090 to 31,770 ppm, was measured compared with 400 to 650 ppm for the water in the reservoir. The higher values are associated with the cooler water in Seep 6. By August 14, the concentrations had dropped in all areas, ranging from 1,500 to 7,740 ppm, the higher values remaining associated with Seep 6. The interpretation of shorter seepage paths resulting in less dissolution of salts from the geologic units was consistent with the physical conditions at the site, and the reduction overall indicated that the development of solution channels was improbable.

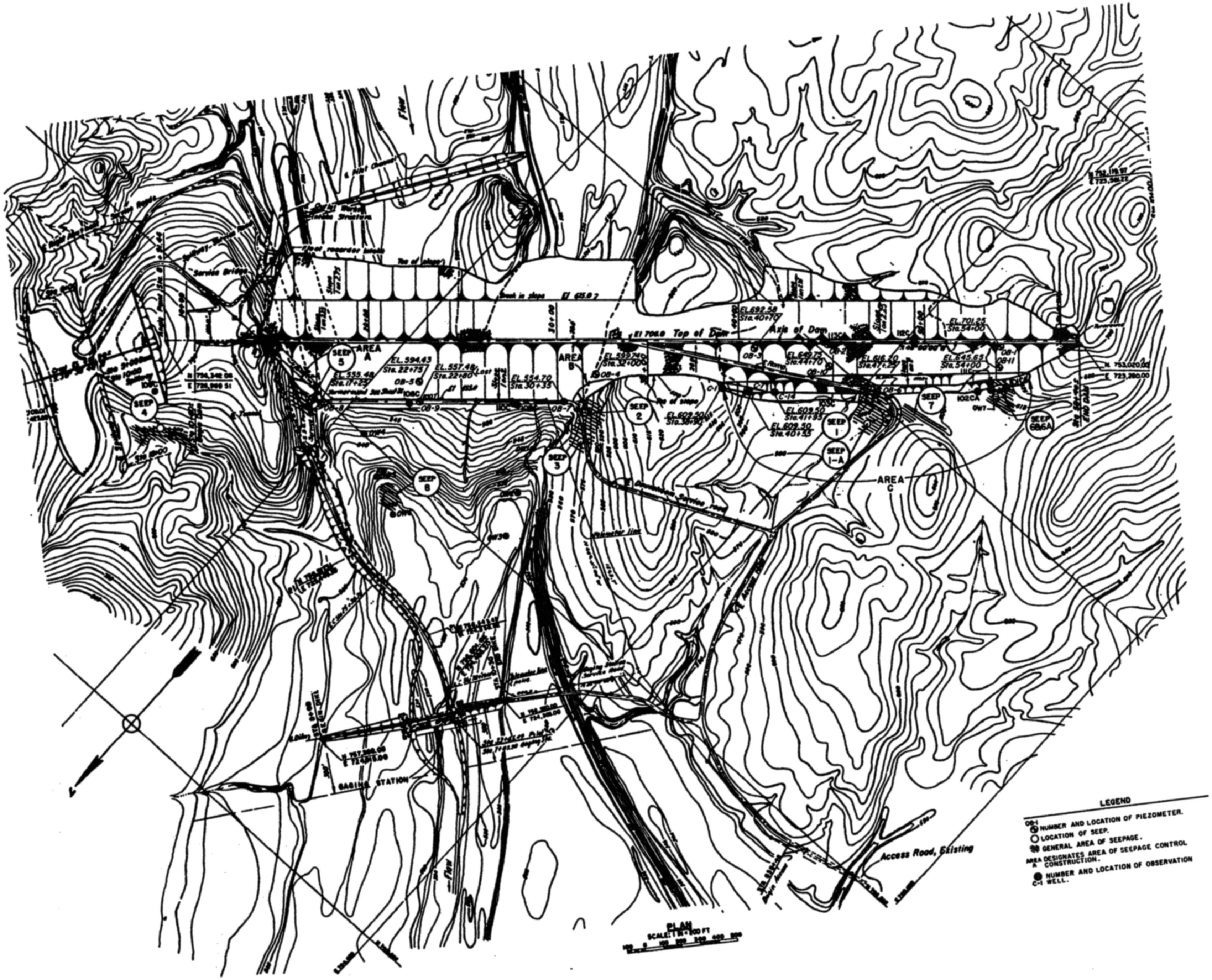


Figure 4. Plan view of Painted Rock Dam showing sequentially numbered seeps in their order of occurrence.

TABLE 1

Summary of Seepage Areas

Number	Location	Surface Elev, msl	Depth to Rock	Remarks
1A	Sta 47+00 at base of downstream access ramp.	600	15-20 ft (4.6-6 m)	Natural topographic low with rock outcrop as one boundary.
1B	Sta 47+00, 200 ft (61 m) north of Area 1A.	600	15-20 ft (4.6-6 m)	Natural topographic low with rock outcrop as one boundary.
2	Sta 31+00 at downstream of lower access road.	560	0	On face of mid-valley rock ridge; north-south joints.
3	200 ft (61 m) north of Area 2.	565±	0	On face of mid-valley rock ridge; north-south joints.
4	In remote spillway, 200 ft (61 m) downstream of crest.	600	0	Fractured rhyolitic andesite.
5	Sta 17+00 at right abutment and lower access road.	555±	0	Rhyolitic andesite with joints.
6	Sta 54+00 at downstream toe.	615±	6 ft (1.8 m)	Thin alluvium.
6A	Sta 54+00 at downstream toe.	615±	6 ft (1.8 m)	Thin alluvium.
7	Sta 47+00 to 51+00 at about 200-1000 ft (61-305 m) downstream of toe	600	4 ft (1.2 m)	Rock ridge; exposed contact of tuff-agglomerate and rhyolitic andesite.
8	Sta 20+00 at 600 ft (183 m) downstream of toe.	531	60-80 ft (18.3-24.4 m)	Main channel; foundation rock faulted.

Piezometers

Multipoint piezometers, a Casagrande and two pneumatic, were installed in each of nine borings ranging from 41.4 to 150 ft (12.6 to 45.7 m) in depth during the period February through June 1979. The Casagrande device was generally installed in or near rock, with the two pneumatics above. In three of the borings all three devices were placed in rock. Borings made from the crest of the dam were angled downstream to avoid the impervious zone and surveyed to determine location. Initial readings indicated that all piezometers were operational, except for two of the pneumatic devices.

EVALUATION OF SEEPAGE CONDITIONS AND STABILITY

Examination of the design exploration data, geologic information collected during construction, and results of the various investigations confirmed that the complex site conditions gave rise to erratic seepage paths. There was general agreement between the data and investigations regarding the location of seepage paths, correlating to the joint systems in the foundation rock. No evidence was found to indicate seepage through the embankment, although quasi-steady state conditions could possibly develop during long storage durations.

In addition to evaluation of seepage and stability at each defined seepage zone by Corps staff, an independent evaluation was commissioned. Both evaluations concluded that seepage pressures developed under the dam, in areas of shallow alluvium, would give rise to conditions of marginal stability, i.e., a safety factor near 1.0 for pool elevations of about 650 and long term storage, while in areas of deeper alluvium pressures would be dissipated in the foundation alluvium without endangering the embankment. This conclusion was based on flownets adjusted to piezometer and well point data and stability analyses. The decision was made to construct relief, control and collection systems and stability berms in the downstream seepage areas.

DESIGN AND CONSTRUCTION OF CONTROL MEASURES

Based on the determination that the seepage was underseepage in the rock foundation rather than seepage through the embankment, the decision was made to incorporate three distinct components into the downstream relief and control system. (1) A series of relief wells drilled into rock and angled to intercept the maximum number of joints in the foundation rock. A separate collector and discharge system, designed to accommodate over 200 gpm (0.1 m^3/s) per well while only flowing one-half full, was provided for each group of wells. (2) Graded toe drains with drainage blankets over the seep areas where exit gradients were calculated to exceed 2, to prevent the possibility of fines being piped and provide for controlled discharges. (3) A drained surcharge berm over the toe/blanket drains and embankment toe to provide additional stability at high pool levels.

The required capacity of relief wells was evaluated by installing three wells in seep areas 1 and 1A, and pumping to observe drawdown and flow conditions. A capacity of at least 200 gpm (0.1 m^3/s) was required to effect the desired amount of relief. Due to construction considerations, the design configuration of the installed

TABLE 2

Summary of Investigations

Name	Objective	Date	Results
Observation Wells (wellpoints)	Early and continued monitoring of conditions in the vicinity of seeps and in the valley section.	1/79	Provided indication of subsurface flow conditions, time lags, and permeability (when pumped).
Electrical Resistivity	Early identification and location of seepage paths, Areas 1, 2 and 3 only. Wenner & Bristow arrays.	1/19/79-1/26/79	Identified possible seepage paths contributing to Areas 1, 2 and 3.
Geothermal: - 10 ft deep - deep profiles	Early identification of seepage paths from crest and access roads by thermal anomolies. Vertical surveys in deep piezometer borings indicated elevations.	1/20/79-2/4/79, and continuing into 1980	Identified possible seepage paths for Areas 1, 2, 3, 5 and 8.
Aerial Imagery: - Thermal IR - Color Photo - Color IR	Early identification or verification of seepage locations over a broad area, taking advantage of wide daily thermal variations and possible water temperature differentials.	1/28/79	Verified seeps 1, 2, 3 and 4. Detected seeps 7 & 8 before visual detection. Did not detect seeps 5 & 6. Indicated seeps 1 & 2 to be 6-8° F warmer than reservoir.
Refraction Seismic	Locate depth to rock across broad area to aid in design of piezometer and relief well systems.	2/2/79-2/8/79	Provided top of rock data. Partially saturated flow conditions negated ability to locate paths.
Thermal & Chemical	Early and continued evaluation of seepage and reservior temperature and quality to aid in identification of seepage conditions.	4/79 into 1980	Provided comparison of temperature and dissolved solids for incorporation into seepage evaluations.
Piezometers	Long-term monitoring of seepage conditions in the embankment and foundation. Installation based on visual observations, electrical resistivity and geothermal surveys.	4/79 to present	Monitored pressure response and lag time for original seepage condition and to evaluate remedial construction effectiveness.

drainage blankets was calculated to have 100 to 300 percent of the capacity required for satisfactory performance. Design of the stability berms conservatively assumed that relief systems were ineffective, a desirable and relatively inexpensive approach in light of the complex seepage conditions.

Seeps were grouped to form three areas, identified as Areas A, B, and C in Figure 6. Typical cross-sections for the systems constructed in each area are shown in Figures 7 and 8.

Area A, encompassing the right abutment contact and extending to Sta 17+35, contained Seep 5 and had eight wells drilled at least 40 ft (12.2 m) into rock on 20 ft (6 m) centers along the contact. The contact was blanketed with a drain extending over the abutment and embankment 15 ft (4.6 m) in each direction, and a 15 ft (4.6 m) thick stability berm constructed over the area. Drainage discharge was routed through a headwall located on the downstream side of the access road at the toe, Figure 9.

Area B included the embankment-rock ridge contact at the left side of the main valley, extending from Sta 30+72 to 32+76 and encompassing Seeps 2 and 3. The system is the same as described for Area A, with nine wells installed along the contact.

Area C extends from Sta 40+00 to 54+70 and contains Seeps 1 and 6. It was considered the most critical area due to the shallowness of the foundation alluvium and a contained basin in the rock surface at depth. Four gravel packed relief well systems were installed in this area, totaling 40 wells drilled 40 ft (12.2 m) into rock at 20 ft (6 m) spacing. Wells were deleted when the alluvium exceeded the 15 to 20 ft (4.6 to 6 m) depth. Discharge from each group of wells was routed into a manhole which contains silt basins and provisions for measuring flow quantities. From the manholes the flow is routed to a headwall for discharge. The area was covered with a drainage blanket and stability berm extending onto the embankment, Figure 8.

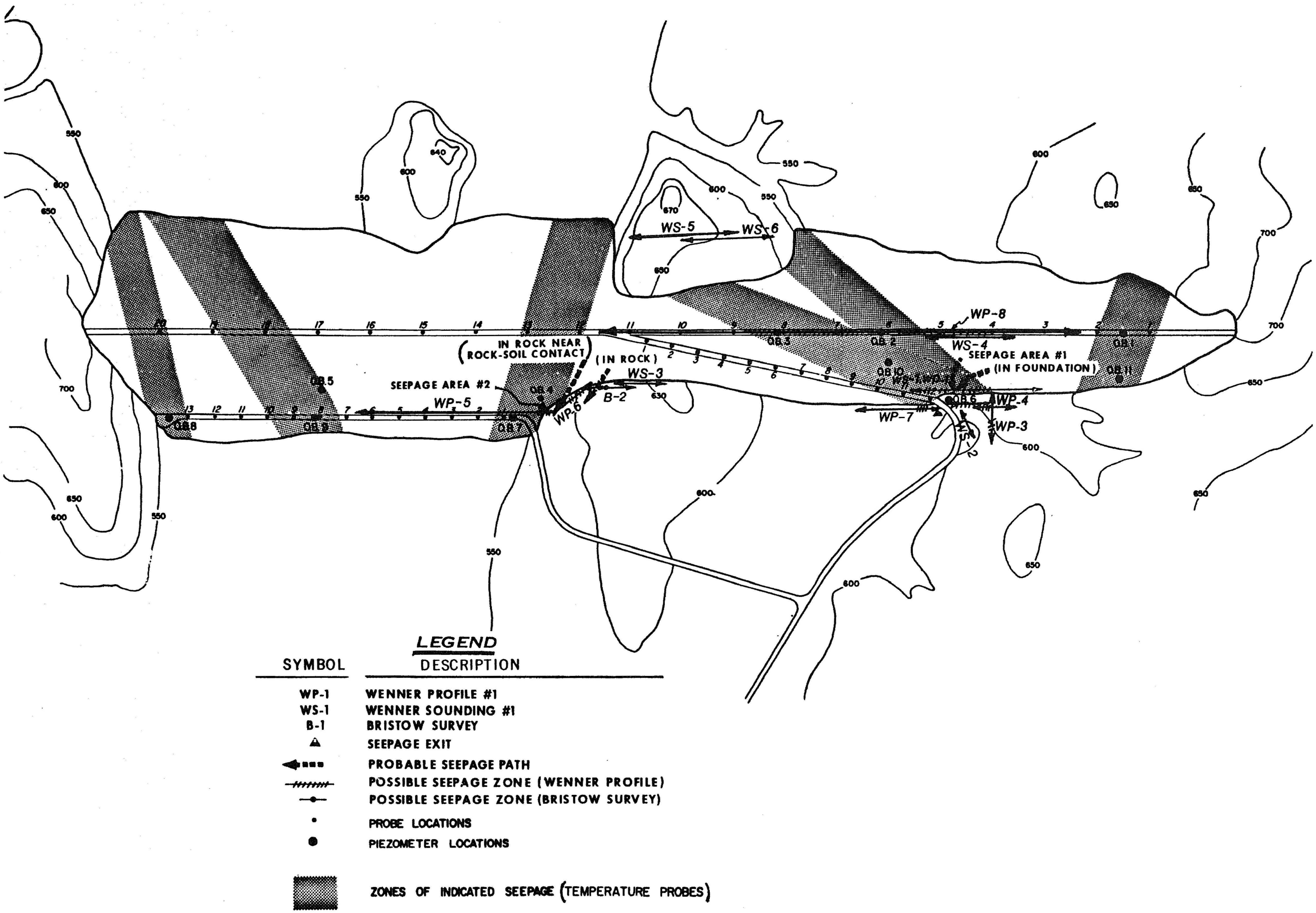


Figure 5. Plan view showing locations of instruments and surveys used to study seepage. Also shown are the probable seepage paths inferred from the various methods.

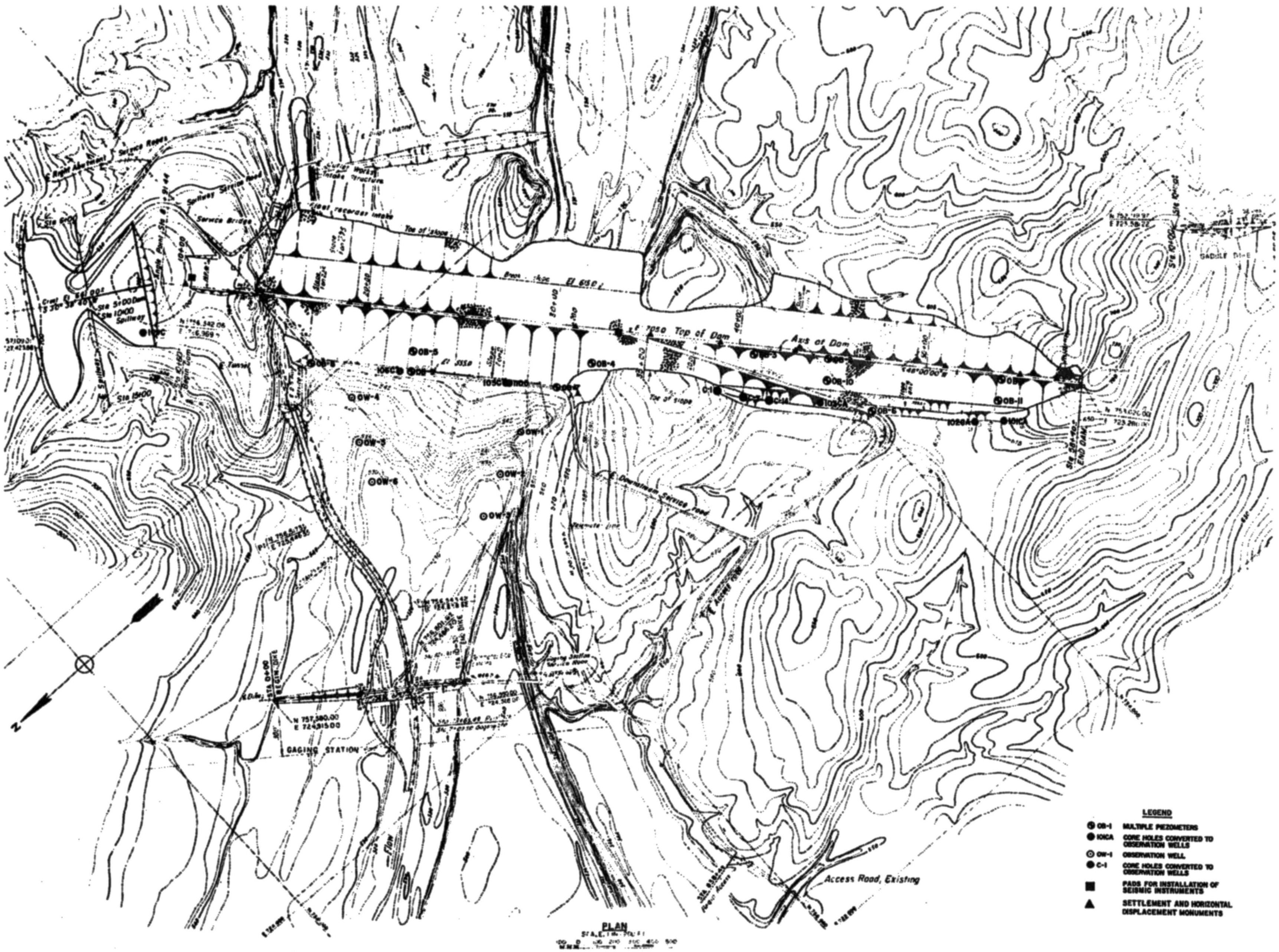


Figure 6. Plan view showing areas of remedial measures and all instrumentation which is presently installed and operating.

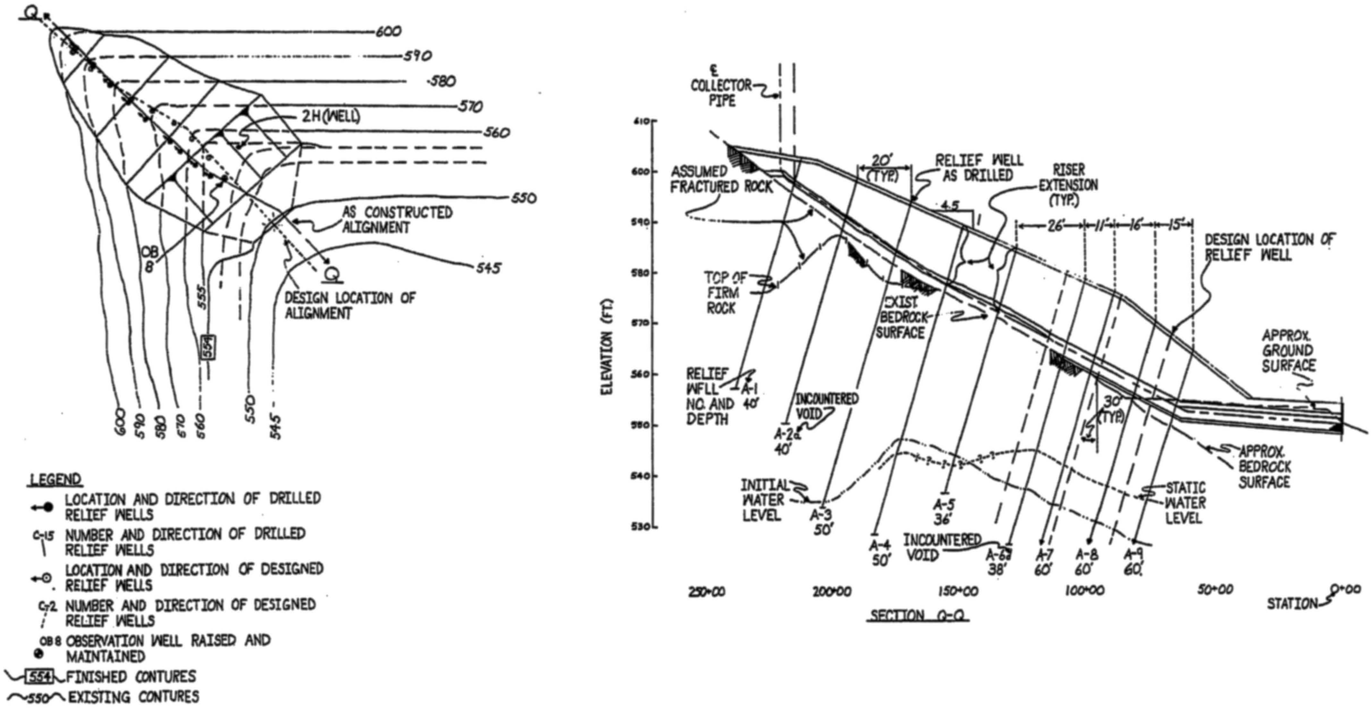


Figure 7. Plan and section of seepage control systems and stability berm constructed in Areas A and B.

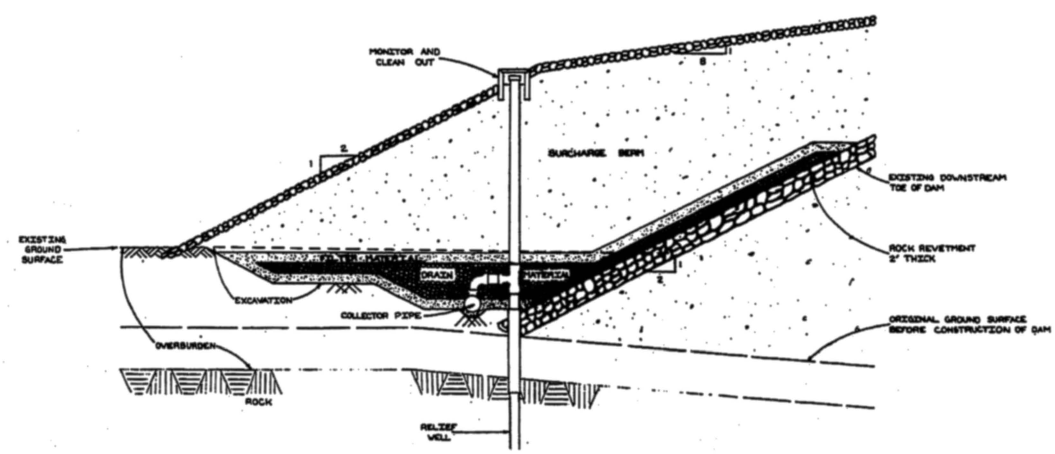
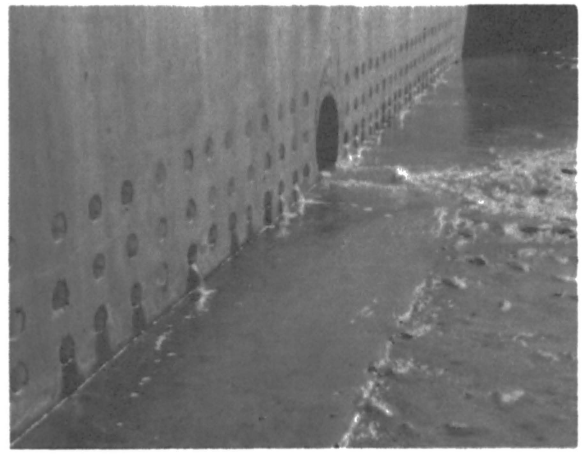


Figure 8. Section of seepage control systems and stability berm construction in Area C.

Area B



Area C

Figure 9. Completed headwalls in Areas B and C. The weephole discharge is from the gravel drainage blankets and the discharge from the larger pipes is from the relief wells.

POST CONSTRUCTION PERFORMANCE

Large inflows in February and March of 1980 resulted in a new record high pool at Elev 647.7 on March 6, 1980. Relief wells in Area C started flowing on February 25, followed by well flows in Areas A and B on February 27. Depth to the gravel pack was measured in Area C wells, and three wells were observed to have had some consolidation of the pack.

Seeps appeared on the right abutment downstream of Area A, on the rock ridge downstream of Area B, and downstream of the headwall in Area C on March 3, 1980. Additional seepage appeared in Area C on March 4, expanding downstream in a linear fashion a distance of about 35 ft (10.7 m), see Figures 10, 11 and 12.

Piezometer readings indicated general relief of pressures in rock, but one was artesian, demonstrating that flow in the complex foundation warranted the conservatism exercised in the design of relief and control measures. Overall performance of the relief, control and stability measures was satisfactory to assure safe performance of the structure at design pool elevations, but the relief wells were not as effective as anticipated, due largely to flow occuring in foundation joints which were not intercepted by wells.

LESSONS LEARNED

1. Continued awareness must be maintained for changes which may compromise the original design intent and, hence, safe performance of a project.

2. As many investigative methods as needed and reasonably possible should be used to obtain information in complex situations so that confidence can be developed regarding existing conditions and proposed solutions.

3. Geothermal monitoring can be an effective diagnostic tool for locating complex seepage paths.

4. A measure of conservativeness should be maintained in the design of remedial measures to accommodate unknowns and inaccuracies.

ACKNOWLEDGEMENTS

The work covered herein was the result of a high level of cooperation between several organizations and the dedicated effort of many individuals. Coordination and direction of the work was done by Hugo Kraythoff, project engineer. The investigation, design and construction of these measures within essentially one year attests to the success of their efforts.

REFERENCES

Design Memorandum No. 3, Dam and Appurtenances for Painted Rock Reservoir, Gila River, Arizona, March 1955. U.S. Army Corps of Engineers, Los Angeles District.

Design Memorandum No. 6, Dam and Appurtenances for Painted Rock Reservoir, Gila River, Arizona, November 1956. U.S. Army Corps of Engineers, Los Angeles District.

Report on Construction Control and Embankment Construction, Painted Rock Reservoir, Dam and Appurtenances, Gila River, Arizona, U.S. Army Corps of Engineers, Los Angeles District.

Painted Rock Dam, Arizona, Study of Seepage Conditions - 1979 Filling, H.R. Cedergren, January - July 1979.

Thermal Monitoring of Painted Rock Dam, Maricopa County, Arizona. Geothermal Surveys, Inc., February 1979.

Remote Sensing Study, Painted Rock Dam, Arizona. Fugro, Inc., March 1979.

Piezometer Installations, Painted Rock Dam, Arizona, Fugro, Inc., March 1979.

Painted Rock Dam, Part I: Surface Resistivity Studies, and Part II: Refraction Seismic Surveys. U.S. Army Engineer Waterways Experiment Station, February 1979.

Seepage Control Measures, Painted Rock Dam, Gila River Basin, Arizona, October 1981. U.S. Army Corps of Engineers, Los Angeles District.

Figure 10. Area A completed. Flow along toe of surcharge berm is from observation well OB-8 which became artesian with high reservoir elevation and a small seep in the abutment rock.

Figure 11. Area B completed. Damp spot in center is from seepage exiting through fractures in the bedrock.

Figure 12. Area C completed. Nuisance surface flow exists along what could be a fault zone. Seepage expanded downstream in a linear fashion along this line.