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HISTORIC DAM – MODERN UPGRADE

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INTRODUCTION

Black Rock Dam, originally named Zuni Dam, is an historic embankment dam built by the Indian Irrigation Service of the Bureau of Indian Affairs (BIA) between 1904 and 1908 on the Zuni Indian Reservation in western New Mexico, on the Zuni River. Ancient basalt lava flows cover the area of the dam site, and initially impounded the Zuni River in a shallow lake. The impounded water eventually undermined the basalt, forming the gorge and dam site. The intensely jointed basalt, overlain by several feet of unconsolidated surficial deposits, form the upper 30 to 40 feet of the abutments. The basalt is underlain by erodible, discontinuous beds of silty and clayey sands and gravels, and clay. The dam is a combination embankment, consisting of a downstream rockfill section and an upstream hydraulic earthfill section. The dam is 80 feet high (structural height of 110 feet), and has a crest length of 780 feet. The original spillway was constructed by excavating a channel in the left abutment basalt, and lining the channel with concrete and masonry.

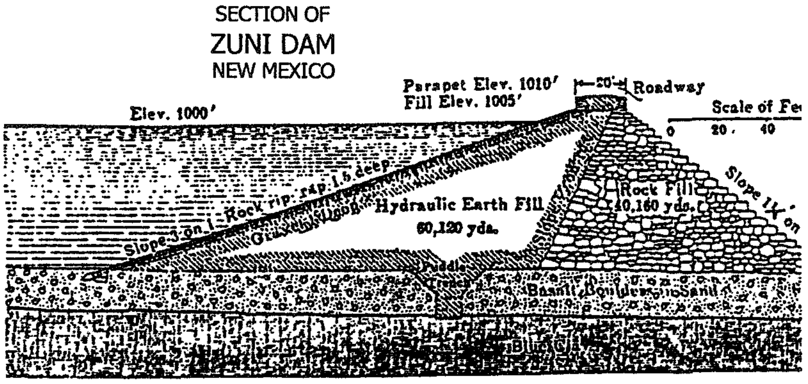

Figure 1. Black Rock Dam Maximum Section

During the first filling in 1909, reservoir water flowed through fractures in the basalt at rates up to 5000 cfs, eroding the underlying deposits, which resulted in settlements of up to 10 feet of the basalt blocks, partial collapse of the spillway structure and near failure of the left abutment of the dam. A new concrete and masonry spillway structure and a masonry and steel sheet pile cutoff wall were built in 1909-12. Exposed basalt was also blanketed with earth fill. Despite these attempts to limit flows through the basalt, the dam nearly failed from erosion in the left abutment in 1932 and again in 1936.

More recent inspections by the U. S. Bureau of Reclamation under their Safety Evaluation of Existing Dams (SEED) program classified Black Rock Dam as high

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hazard, and identified significant dam safety deficiencies of uncontrolled seepage through the abutments, inadequate spillway capacity, and poor condition of the outlet works. In addition to safety deficiencies, the reservoir has experienced heavy sedimentation, which has reduced the design storage volume of 15,000 acre-feet to about 2,600 acre-feet today.

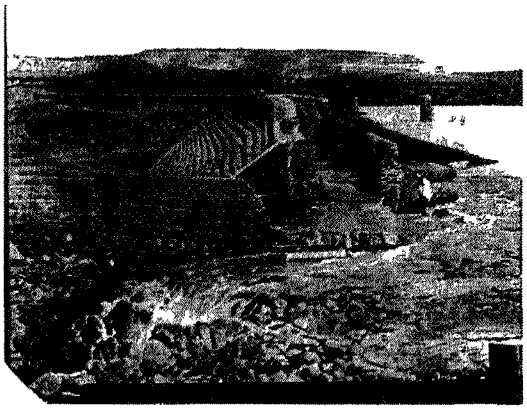
Figure 2 Black Rock Dam Near Failure 1909, Spillway is in the Foreground

The dam is operated by the Bureau of Indian Affairs (BIA). Under auspices of the "Indian Self Determination Act," Public Law 93-638, the Pueblo of Zuni contracted with the BIA to administer the design and construction of the needed safety improvements. The Pueblo of Zuni selected GEI Consultants, Inc. for design and engineering services, and Laguna Construction Company, owned by the Pueblo of Laguna, as construction contractor. Design began 1996. The first phase of construction began in early 1998 and is scheduled to be completed in August 2001.

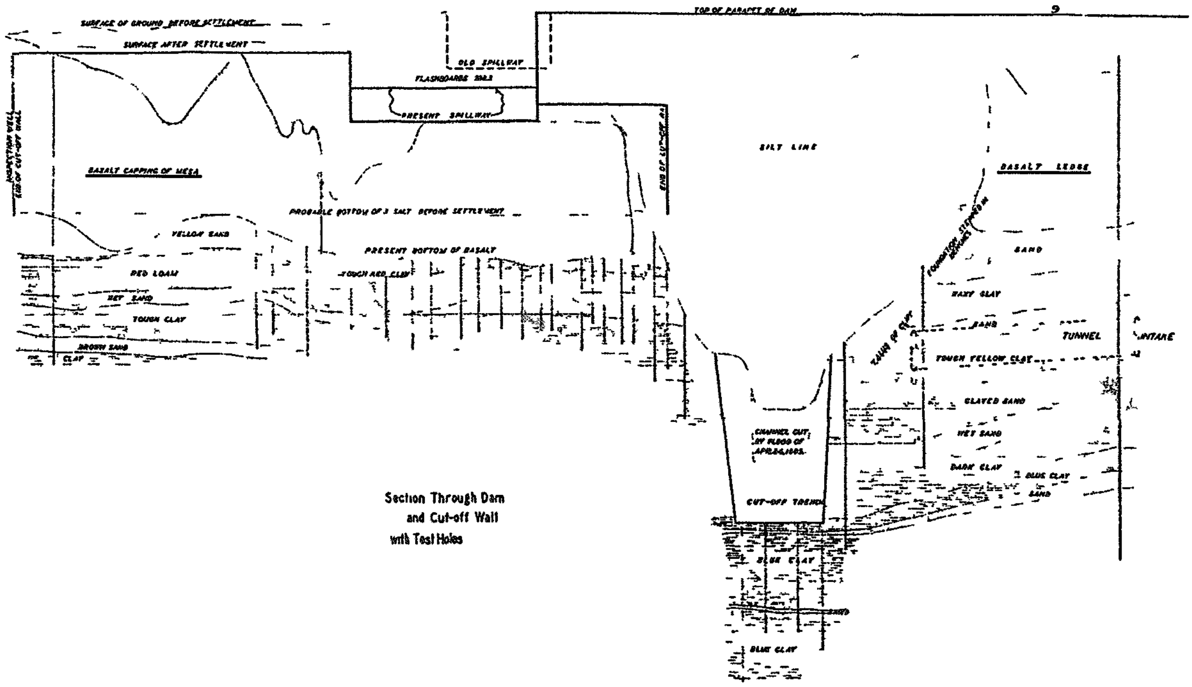

Figure 3 Geologic Profile Through the Axis of the Dam, Looking Downstream

OUTLET WORKS

The original outlet works consists of an upstream, pile-supported intake tower in the reservoir, a concrete lined tunnel through the right abutment and a terminal structure, which delivers flows to an irrigation canal or to the Zuni River. Reservoir releases were controlled by a single, low-level gate at the base of the intake tower. The gate had no trash rack. This low level gate was damaged and became inoperable in 1971, and a new gate was installed at a higher level, approximately the level of sediment in the reservoir. The low-level gate opening was sealed with concrete on the inside of the tower.

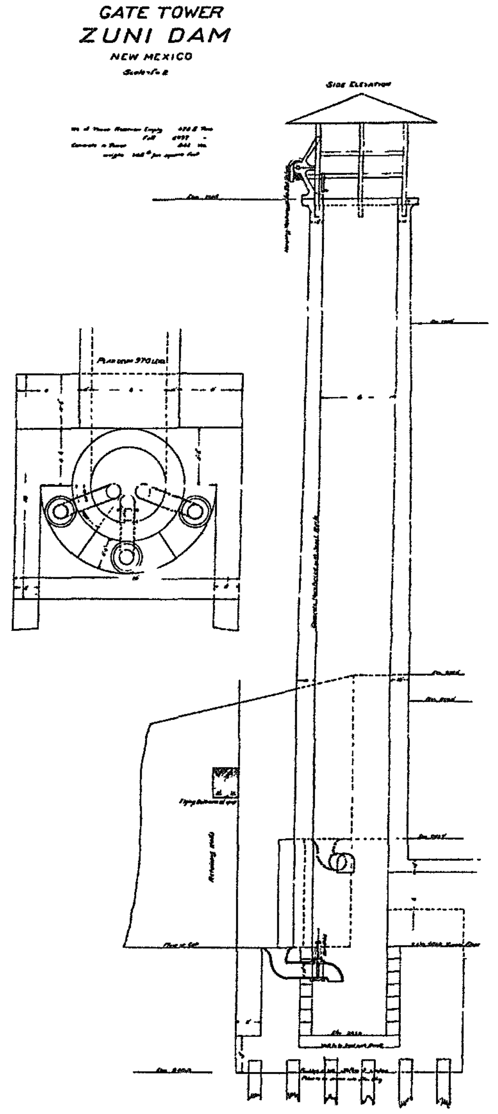

Figure 4. Gate Tower

The 6-foot inside diameter tower was constructed of concrete reinforced with small railroad tracks and then-conventional, square reinforcing bars. The concrete was severely deteriorated due to freeze-thaw action, and there were several circumferential cracks in the structure.

The upper portion of the tower was removed and a circular steel liner was grouted into the remaining portion of the structure. The new steel liner extends up to the elevation of the top of the original tower, and a new reinforced concrete structure was built around the new liner. The new structure is designed to resist a pseudo-static seismic force of 0.33g, as well as water and sediment loads.

The original low level gate and concrete seal were removed and a new gate was installed. A trash rack structure with provisions for stop logs was incorporated with the gate replacement. The new, upper portion of the tower includes a second gate at the reservoir sediment level. Both gates are cast iron sluice gates, operated by mechanical lifts on a deck on the top of the tower.

The outlet works conduit is a six-foot diameter, concrete lined, tunnel through the right abutment, below the overlying basalt. The concrete lining had longitudinal and circumfrential cracks of up 3/8-inch wide and settlements of up to four inches in several places. In addition to the cracks, the joints between the lining segments had separated, up to a half-inch, allowing water to enter the conduit. No reports were found on how the concrete lining was constructed. Observation of the tunnel and construction debris at the site indicate that it was cast in place, in annular segments about two feet long and about one foot thick. Form supports for the concrete segments were steel angles bent into a circular shape, and the forms were two-foot long boards, spanning between adjacent circular steel angle sets. There are no records or reports of high seepage at or near downstream end of the conduit. The small amount of seepage that was observed could as easily have been from springs in the right abutment as from reservoir seepage along the conduit. Whatever efforts had been made during the original construction to prevent seepage along the conduit appeared to be successful.

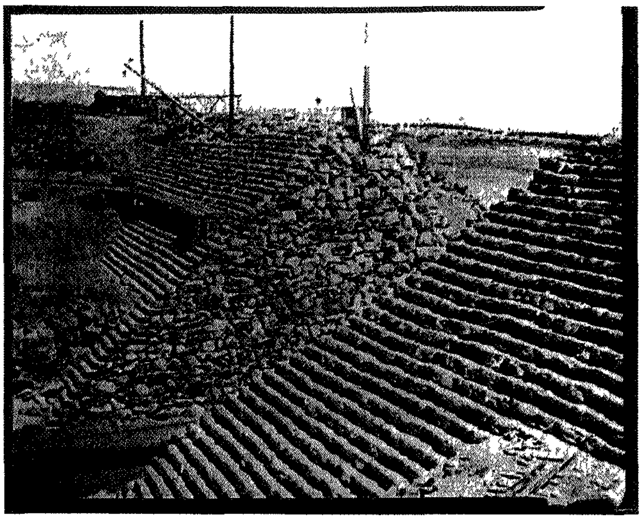

Figure 5. Original Outlet Works

A four-foot diameter steel liner, sized to meet reservoir drawdown requirements of 250 cfs and designed to resist all external loads without relying on the original concrete lining, was grouted in place and welded to the vertical steel liner in the intake tower. The liner extends 40 feet beyond the downstream end of the tunnel to a new terminal structure. A sand filter was placed around the steel pipe liner at the end of the tunnel to control any seepage that might occur along the new steel liner and the original concrete tunnel lining.

The new terminal structure was replaced with a modified impact basin. This impact basin has a wall at the downstream end, which serves as a check, raising the water level in the basin so that it flows by gravity into the nearby irrigation canal. A Parshall flume in the canal measures irrigations flows.

High flows through the outlet works overtop the check wall in the impact basin, and continue downstream to the Zuni River.

SPILLWAY

The spillway, rebuilt in 1909-12, and modified several times afterwards to increase reservoir storage, was generally in good condition. Its discharge capacity of 6,000 cfs was far less than the 63,500 cfs required to meet dam safety standards.

Inflow Design Flood

The drainage area above Black Rock Dam is 800 square miles, of which 200 square miles is a closed basin, and considered non-tributary. The basin lies west of the continental divide, with a mix of mountains and steep, narrow valleys, mesas and broad, flat canyons, and plains. Elevations range from 6247 at the dam to 8922 in the Zuni Mountains, which form the continental divide. Vegetation ranges from sagebrush, and range grass, pinion and juniper in the lower elevations to ponderosa pines at higher elevations. Soils are shallow in the mountains, valleys and on the mesas, but very deep in the canyons formed by the mesas. The soils have a high clay content, and most are Hydrologic Soil Group C.

With the Pueblo of Zuni about three miles downstream, the dam is classified as high hazard - loss of life would be expected should the dam fail - and the inflow design flood is the probable maximum flood (PMF). The Zuni River is a tributary to the Little Colorado River, which flows into the Colorado River just upstream of the Grand Canyon. Procedures for estimating the probable maximum precipitation (PMP) for the area are prescribed in Hydrometeorological Report 49, "Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages," published by the National Weather Service.

The Zuni River is formed by the confluence of the Rio Nutria and the Rio Pescado, about two miles upstream of the dam. Each of these tributaries has several on-stream reservoirs: three on the Rio Nutria, two on the Rio Pescado. Of these five reservoirs, two are large enough to affect flood flows. The sub-basin for each of these tributaries was modeled separately, and tributary sub-basins were further sub-divided at the two large upstream reservoirs. The overall hydrologic model includes the attenuating effects of the upstream reservoirs, and the failure of the those dam when their spillway capacity is exceed. The flood hydrology was modeled using the HEC 1 computer program.

Using the PMP from HMR 49 and the HEC 1 hydrologic computer model, the PMF is estimated to have a peak inflow of 65,400 cfs, and the routed outflow (based on the final spillway configuration) of 63,500 cfs. The PMF volume is estimated to be 62,200 acre-feet.

A risk analysis which might reduce the inflow design flood (IDF) from the PMF was considered unlikely to be effective due to the close proximity of the Pueblo of Zuni three miles downstream of the dam. A site-specific probable maximum precipitation study was undertaken for the area, but resulted in little difference in the estimated PMP for the basin based on HMR 49.

Spillway Alternatives

Several spillway alternatives for passing the PMF were considered, including lowering normal storage levels to increase flood storage, overtopping protection on the downstream face, a replacement dam upstream, auxiliary spillways on one or both of the abutments, and increasing the capacity of the existing spillway.

Lowering the normal storage level in the reservoir, already reduced by sedimentation, was considered undesirable. The basalt blocks on the downstream face are considered historically important and the Pueblo of Zuni was reluctant to make major modifications to the appearance of the dam. Overtopping protection would have significantly modified the downstream face of the dam, making it undesirable.

A site for a new embankment dam, about two miles upstream of Black Rock Dam, had been identified in earlier studies by others. A replacement dam at this site, and the removal of Black Rock Dam, was considered. The new dam site was in a broad valley flanked by mesas. The foundation and abutments would have required extensive treatment to control seepage. The dam would rise 120 feet above the valley floor, and have a crest length of 2300 feet, much larger than Black Rock Dam. The visual impact of this new dam, the impacts of constructing a dam at a new site, and the loss of the existing, historic dam ruled out a replacement dam.

The existing spillway was originally constructed with masonry walls and a concrete floor. At the entrance to the spillway, a seven-foot high, thin ogee crest had been constructed. Replacement of this fixed crest with a moveable crest would maintain normal reservoir water storage, and allow greatly increased spillway flows when moved or lowered during large floods.

Several types of moveable crests were evaluated, including inflatable bladders, inflatable bladders with upstream steel facing, earth embankment fuse plugs, and Fuse Gates, a proprietary product. Fuse Gates were selected due to their low maintenance requirements, and their ability to pass normal flows as a service spillway while tipping at prescribed reservoir elevations to pass large floods without depending on mechanical equipment or human intervention.

Selected Alternative

The selected spillway configuration consists of modifications to the existing spillway, the installation of Fuse Gates in the existing spillway, and an auxiliary spillway on the left abutment.

The Fuse Gate installation consists of fifteen steel-fabricated units, each 7.9 feet high. During normal operations, flows pass over the labyrinth crest formed by Fuse Gates. During large floods the Fuse Gates are designed to tip three at a time, at five predetermined reservoir elevations. The first set of Fuse Gates will tip at flows near the 100-year flood, and the last set will tip with the reservoir elevation at the crest of the auxiliary spillway.

The walls of the existing spillway converge. The left wall is skewed relative to the crest, and the right wall is curved, convex to the spillway. The convergence produces a backwater in the spillway at high flows. Hydraulic control transfers from the spillway entrance at low flows to a point downstream within the spillway at high flows. To maintain hydraulic control at the upstream end of the spillway, maximizing its discharge capacity, a portion of the right spillway wall was reconstructed parallel to the left spillway wall, which widened the lower end of the spillway.

With the Fuse Gates in place, the modified spillway can pass the 100-year flood without using the auxiliary spillway. With the Fuse Gates removed, the modified spillway can pass 30% of the PMF without using the auxiliary spillway and 50% of the PMF, with the reservoir elevation at the crest of the dam. The auxiliary spillway passes the other 50% of the PMF.

The existing spillway structure extends downstream of the dam, but ends 60 feet above the Zuni River. Discharges from the spillway flow down a steep chute formed by the collapse of the original spillway in 1909, which left the chute lined with random blocks of basalt. The chute had functioned satisfactorily for the relatively small flows, which had occurred since 1909, but was vulnerable to erosion and undercutting of the spillway if design flows were to occur. To protect the chute from erosion, an RCC chute and stilling basin structure was constructed to carry flows from the existing spillway to the Zuni River.

Since floods of up to the 100-year flood flow through the modified existing spillway, the auxiliary spillway discharges only during larger, less frequent, floods. Some erosion damage to the auxiliary spillway during these infrequent floods is acceptable, as long as the sudden, uncontrolled release of the reservoir does not result.

With this criteria, the auxiliary spillway was designed as an open cut on the left abutment, just left of the existing spillway. The auxiliary spillway consists of a channel excavated in earth and basalt, with a control section at the upstream end. The auxiliary spillway can pass 31,500 cfs with the reservoir elevation at the crest of the dam. The combined capacity of both the service spillway and auxiliary spillway is 63,500 cfs.

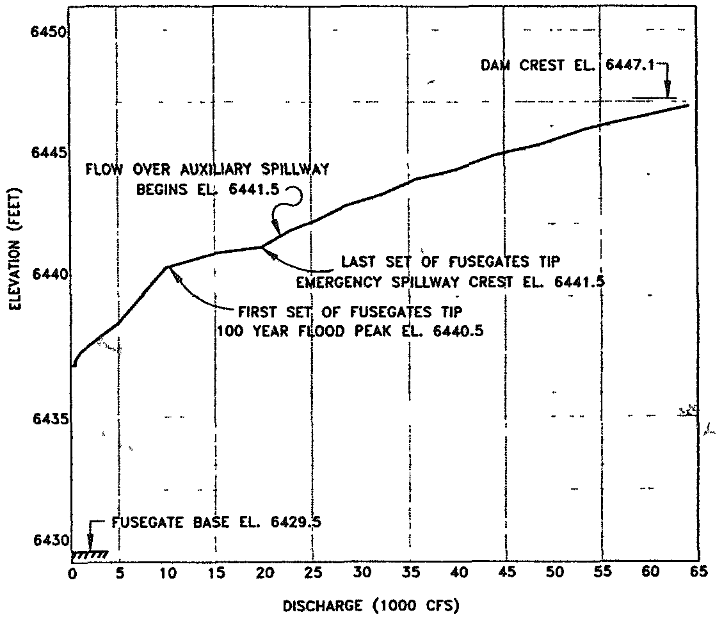

Figure 6. Elevation-Discharge Curve for Combined Service and Auxiliary Spillway

SEEPAGE

Seepage of reservoir water through fractures in the basalt in the upper portions of the abutments, and the subsequent erosion of underlying soil, had caused near-failure of the dam on at least two occasions. Preventing water from entering the basalt is a key consideration for improving the safety of the dam. The left and right abutment stratigraphy are similar and consist of several feet of eolian silty sand deposits over 15 to 30 feet of fractured basalt, over interbedded alluvial sands and gravels, and clays. At normal storage levels, most of the basalt is exposed to reservoir water. The valley between the abutments was eroded into the material below the basalt.

Any seepage control measure would have to extend horizontally into each abutment, and either tie into the hydraulic fill of the dam or extend across the upstream face of the dam.

Alternatives

Several alternatives for reducing and controlling seepage were considered. These included covering the basalt with impervious material, grouting of the basalt blocks, a horizontal imperious blanket, and a vertical cutoff trench.

Covering the upstream faces of the basalt with impervious material or grouting the fractures in the basalt would seem an obvious first step in reducing flows through the basalt. However, water could then flow through the permeable soils beneath the basalt and exit in an upward direction into the basalt fractures downstream of the impervious layer or grout curtain, where erosion of soils and settlement of the abutment would continue. An impervious liner extending several hundred feet horizontally into the reservoir could lengthen the seepage path, reducing gradients to safer levels. Construction of this liner would be very difficult due to the sediments in the reservoir. The sediments are very soft, and construction the liner on the sediments would be difficult and costly. Removal and disposal of sediments beneath a horizontal liner would also be difficult and costly.

A vertical cut-off trench could extend into the soils underlying the basalt, reducing seepage flows and gradients to safe levels, and could be constructed with minimum excavation of reservoir sediments. A soil-bentonite filled cutoff constructed by slurry trench methods was selected.

Design

The depth and horizontal extent of the cutoff trench was selected after seepage modeling of alternatives using the SEEP/W computer program. The silts below the basalt were categorized as either sand or clay, based on descriptions of split spoon samples obtained from borings. Six soil types were modeled as shown in Table 1. The cutoff trench was modeled as soil 7.

Soil Description	**Depth (ft)**	**Hydraulic Conductivity (cm/Sec)**
1. Silty Sand	0 – 12	5 x $10^{\wedge -4}$
2. Basalt	12 – 35	5 x $10^{\wedge -3}$
3. Sand	35 – 45	5 x $10^{\wedge -3}$
4. Clay	45 – 52	5 x $10^{\wedge -8}$
5. Silty Sand	52 – 84	5 x $10^{\wedge -3}$
6. Clay	84 – 150	5 x $10^{\wedge -8}$
7. Cutoff Trench	Varies	5 x $10^{\wedge -7}$

Borings had been made in both abutments. Boring logs indicated that water loss in the basalt was significantly higher in the left abutment than in the right abutment. This correlated well with incidents at the dam where relatively small amounts of seepage and settlement observed on the right abutment compared to the high seepage and settlements observed on the left abutment. The model computed the phreatic surface, equipotential lines and the total seepage flow rate (for comparison) through the profile.

Existing conditions were modeled without a cutoff trench and the calculated phreatic surface was compared to ground water levels observed during drilling. The comparison showed a satisfactory correlation between the calculated and measured values. The effectiveness of various cutoff wall configurations was evaluated by comparing the reduced seepage rate reduction to the results of this initial model.

A five-foot wide cutoff trench was selected, extending 500 feet into the right abutment, 575 feet into the left abutment and 65 feet below the modeled bottom of the basalt. Continuous construction of a single trench was more economical and effective than tying two trenches into the hydraulic fill of the dam. The trench therefore extends across the upstream face of the dam, about 50 feet from the crest, adding about 200 feet to the length of the trench.

In order to construct the cutoff trench, trenches in each abutment had to be excavated through the basalt to expose the underlying soil. The trenches were designed to be as narrow as possible. The trenches were excavated by blasting, with side slopes of 0.5 to 1, and a nominal bottom width of 10 feet. Open fractures in the exposed downstream face of the basalt were manually filled with grout. The trenches were then backfilled with compacted clay, with filter material against the downstream face of the basalt, to a constant elevation, providing a work platform 20 feet wide for the slurry trench construction. The slurry trench was constructed to 75 feet below the work platform

Once the cutoff trench was constructed, the remainder of the trenches in basalt were backfilled with clay and filter material. Because of expected settlement of the soil-bentonite material in the cutoff trench, which could potentially form a void below the clay backfill, the backfill was not placed until several months after completion of the slurry trench. Settlement measurements were taken of the cutoff trench backfill at seven locations along the trench during this period. Settlements were generally 0.8 to 1.2 feet at 180 days.

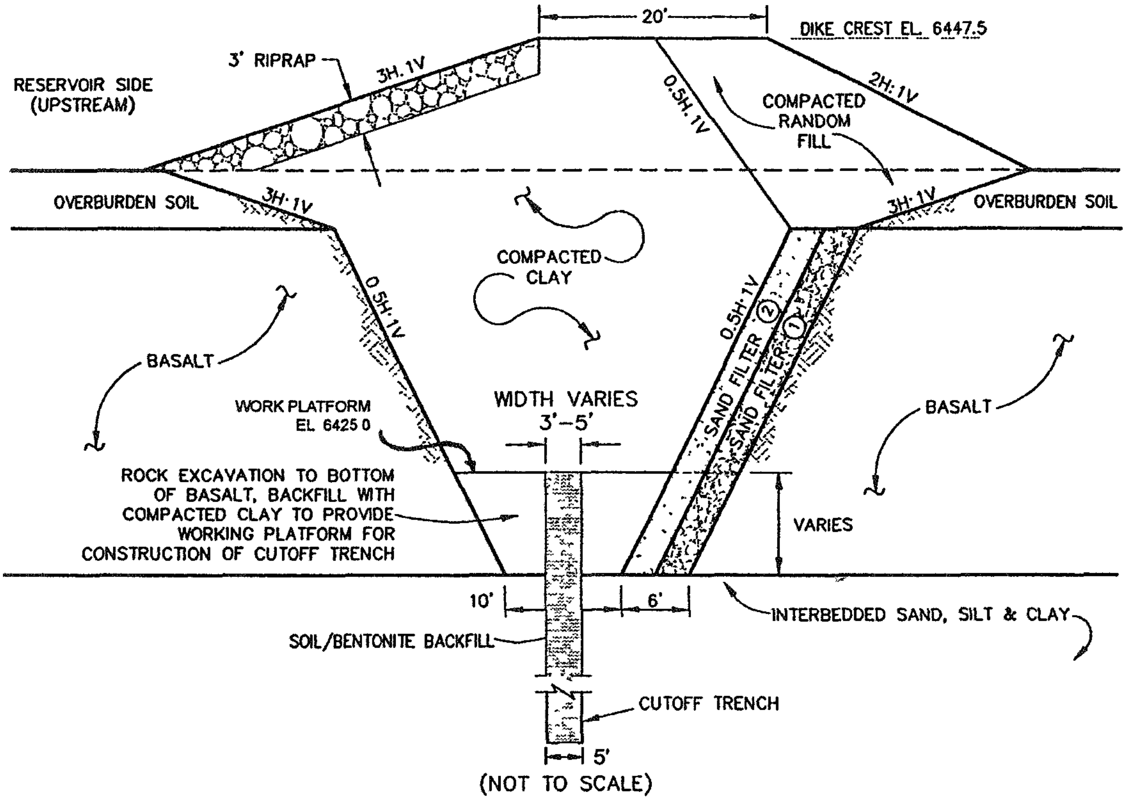

Figure 7. Cutoff Trench Typical Section

Because the alignment of the cutoff trench is upstream of the dam, the top of the clay backfill is below the elevation of the dam crest. Dikes were constructed on the clay backfill, above the cutoff trench, with a crest elevation equal to the dam crest, and rip rap was placed on the upstream face of the dikes.

TYING IT ALL TOGETHER

The cutoff trench and dikes greatly reduces reservoir water flowing through the basalt abutments of the dam. The auxiliary spillway on the left abutment is formed by lowering a section of the left dike, and excavating a discharge channel downstream. Auxiliary spillway discharges would flow over the cutoff trench, and downward into the basalt. To prevent this, the auxiliary spillway is lined with a HDPE liner. The liner is tied into cutoff trench, and extends to a point near the downstream end of the auxiliary. To complete the seal, the liner also extends across the service spillway and is tied into the hydraulic fill of the dam.

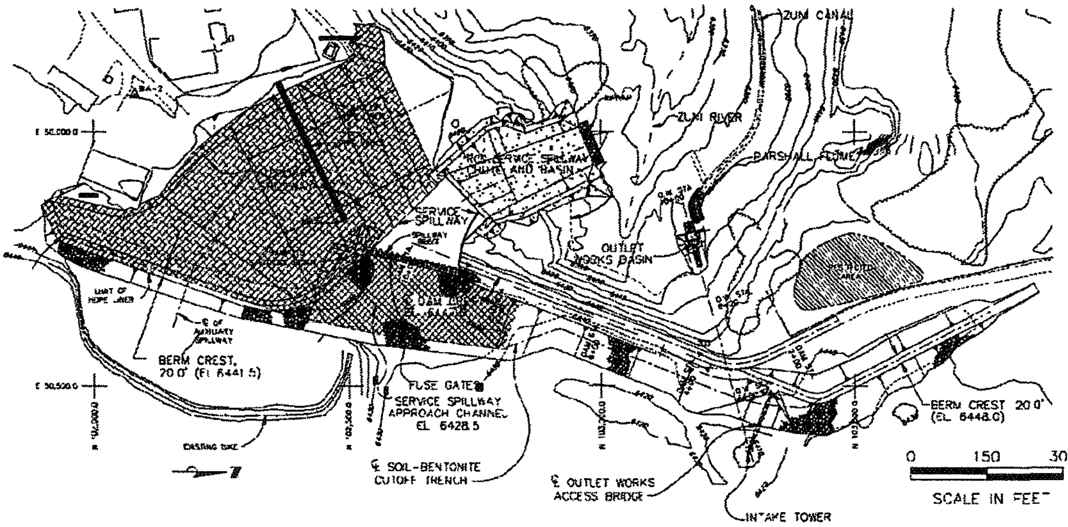

FIGURE 7 - BLACK ROCK DAM PLAN

Figure 7. Black Rock Dam Plan