VOL.108 NO.GT7. JULY 1982

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

PROCEEDINGS OF
THE AMERICAN SOCIETY
OF CIVIL ENGINEERS

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ISSN 0093-6405

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U.S. Army Engineers

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The Journal of the Geotechnical Engineering Division (ISSN 0093-6405) is published monthly by the American Society of Civil Engineers. Publications office is at 345 East 47th Street, New York, N.Y. 10017. Address all ASCE correspondence to the Editorial and General Offices at 345 East 47th Street, New York, N.Y. 10017. Allow six weeks for change of address to become effective. Subscription price to members is $16.00. Nonmember subscriptions available; prices obtainable on request. Second-class postage paid at New York, N.Y. and at additional mailing offices. GT.

POSTMASTER: Send address changes to American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

Remedial Treatment Exploration, Wolf Creek Dam, Ky

By Marvin D. Simmons[1]

Abstract: After 15 years of apparently satisfactory operation, Wolf Creek Dam developed underseepage and piping problems associated with its karstic foundation. When a sinkhole developed at the downstream toe of the earth embankment, a concentrated program of exploration was undertaken to determine both the source of the problem and possible remedial treatment. Various exploratory techniques were used; however, direct subsurface information obtained by drilling and grouting was the only method reliable enough to define the problem so that a remedy could be devised. After successful emergency grouting had arrested the piping, in all probability saving the dam from a major failure, a search for a permanent answer determined that a concrete diaphragm wall through the earth embankment into the limestone foundation would be the most acceptable method of correcting the problem. Construction of such a wall is costly; therefore, it was important to ascertain the minimal depth and extent required for it. The exploratory methods used for this purpose are reviewed in this paper.

Introduction

Wolf Creek Dam is located on the Cumberland River in south central Kentucky some 460 mi (741 km) above its confluence with the Ohio River (Fig. 1). The dam was designed and constructed by the U.S. Army Corps of Engineers during the period 1938–1952. Construction was interrupted from 1943–1947 due to WWII. The dam is a combination concrete gravity and earth fill structure; it impounds the 6,089,000 acre-ft (7.5 x 10^9 m^3) Lake Cumberland, making it the largest reservoir east of the Mississippi River and the ninth largest in the United States (Fig. 2). The 1,796 ft (547.8 m) long concrete section ties into the left valley wall and extends across the old stream channel toward the right abutment (Fig. 3). It has a maximum height of 258 ft (78.7 m) above founding level, and contains 10 50 × 37 ft (15.3 × 11.3 m) radial spillway gates. It also contains the intakes to the powerhouse which has six generators with a total output of 270,000 kW. From the end of the concrete gravity section, a nonzoned rolled earth fill embankment with a maximum height of 205 ft (62.5 m) above top of rock extends 3,940 ft (1,201.7 m) across the valley to the right abutment (Fig. 3). US127 traverses the top of the dam. There are no provisions for emergency drawdown of the reservoir.

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Note.—Discussion open until December 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on June 26, 1981. This paper is part of the Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 108, No. GT7, July, 1982. ISSN 0093-6405/82/0007-0966/$01.00.

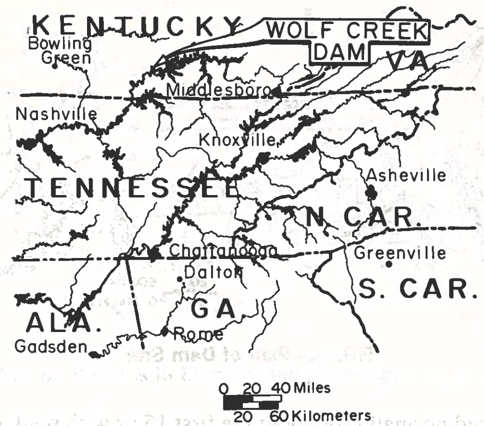

FIG. 1.—Location Map

FIG. 2.—Aerial View of Wolf Creek Dam

When an underseepage and piping problem developed in 1867, emergency grouting most likely saved Wolf Creek Dam from a major failure. Subsequently, a more permanent method of remedial treatment—a concrete diaphragm wall—was devised and installed.

The primary emphasis in this paper is on the exploratory tools and methods used to define the limits of the problem; it does not attempt to deal in detail with all aspects of the problem and its solution. Each technique used is analyzed in enough detail to inform the reader of its positive and negative points. The value of some of the procedures were in question before their use; however, due to the magnitude of the problem, few suggestions were discarded without investigation. (See Appendix I for additional details of the design and construction of the project.)

Foundation Problem (1968–1972)

Lake Cumberland was impounded in December, 1950; the first of six generators was placed on-line in October, 1951 and the last in August, 1952. The

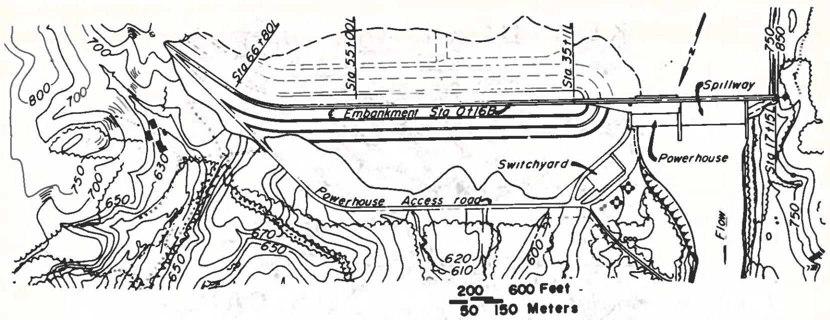

FIG. 3.—Plan of Dam Site

project was operated normally for about the first 15 yr with pool levels fluctuating between approximately El. 692 (211 m) in late fall, to El. 722 (220.2 m) in early spring. In August, 1967, four wet areas were noted toward the right abutment near the downstream toe of the embankment, some 2,000 ft (610 m) away from the tailrace (Fig. 4). Further investigation determined that these wet areas had developed over an extended period of time and had been considered by operations personnel only as a nuisance to be dealt with when mowing the grass. The potential problem that this condition represented was recognized, and plans for instrumentation of the area were undertaken. In October, 1967, muddy water was observed exiting into the tailrace near the retaining wall below the powerhouse at a time when the generators were shut down and the tailwater was at a minimum. In mid-January, 1968, muddy flows could be seen emerging from the rock floor of the tailrace at a point about 50 ft (15.3 m) away from the wall. As investigations continued as to the source of the muddy water, there was still no real alarm relative to the safety of the dam.

Sinkhole.—On March 13, 1968, a sinkhole developed near the downstream toe of the embankment where it wrapped around the concrete section of the dam (Fig. 4). At first, the sinkhole was only about 2 ft (0.6 m) in diameter at the surface, and 6 ft (1.8 m) deep, but in continued to develop until it was 13 ft (4 m) in diameter and 10 ft (3 m) deep (6).

Immediately following the development of this sinkhole, all available data on the foundation were studied. Drill crews were mobilized, and a large number of exploratory borings were made through the embankment and into the foundation. In the beginning of this program, a few piezometers were set at the top of rock; eventually, a network of more than 300 instruments would be employed. One of the first piezometers installed was in an area indicated as a low on the design top of rock contour map. Upon reaching the elevation of general top of rock, a stratified clayey silt was encountered. As the borehole continued, it became apparent that a solution channel, open to the top of rock, was being penetrated. When rock was finally encountered at El. 528 (161 m), some 40 ft (12.2 m) below general top of rock, free water immediately rose to El. 640 (195.2 m). Water later rose to within 2 ft (1.6 m) of the surface (El. 658 (200.7 m)) after the piezometer was installed.

A second sinkhole occurred on April 22, 1968, about 40 ft (12.2 m) away

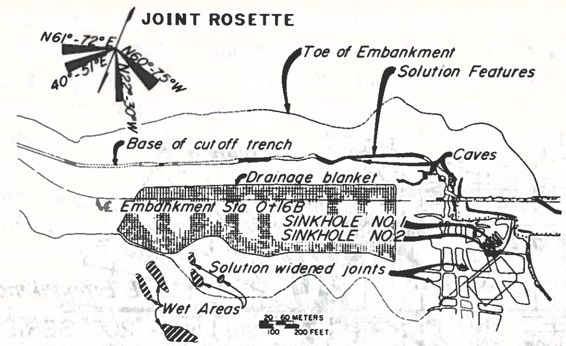

FIG. 4.—Plan of Features in Embankment Foundation and Wet Areas

from the first (Fig. 4). There was no warning of the occurrence of this second sink. In fact, a drill rig had sampled the soil to the top of rock at this location the day before and there had been no indication of disturbance of the fill or natural overburden materials. At about the same time as the occurrence of the second sink, muddy water could be seen in the tailrace with the generators operating.

After assembling available data, an interpretation of the bedrock conditions indicated that an extensive interconnected network of solution channels and caves was present in the limestone foundation of Wolf Creek Dam. As would be expected, these features tended to follow the alignment of the area's joint pattern. It was generally concluded that seepage was passing either through or under the cutoff trench or both, then probably under the area where the sinkholes developed, and finally through a system of solution features in bedrock to the tailrace. Piping of filling material from these rock channels had cleaned them to the point that the overburden and the dam could no longer support themselves. They collapsed into the rock, thus forming the sinkhole at the surface. In addition, a 20 ft (6.1 m) fluctuation of tailwater results from power operations, and this rapid rise and fall causes a surging and flushing action. This opening up of the foundation had allowed dangerously high hydraulic pressures to migrate downstream of the center line of the embankment, and to be present near the surface around the downstream toe in the area of the wrap-around. Some method of safely reducing this high pressure had to be devised.

Emergency Grouting.—It was decided that the most expeditious way of treating the high hydraulic pressure was by grouting (7). The plan devised and ultimately performed was to construct three grout lines (1, 2, and 3 in Fig. 5) along the crest of the dam, tying into the concrete section and extending toward the right abutment about 200 ft (61 m). About 150 ft (45.8 m) to the right of the concrete, another three lines (4, 5, and 6) tied into the first three and ran downstream normal to the axis of the dam, thus isolating the area of the wrap-around. The lines were 5 ft (1.5 m) apart, and the primary holes were drilled full depth to El. 500 ft (152.5 m) on 20 ft (6.1 m) centers. These holes were subsequently split to 2.5 ft (0.76 m) centers, regardless of grout take.

Drilling Grout Holes.—The grout holes, rotary-drilled through the embank-

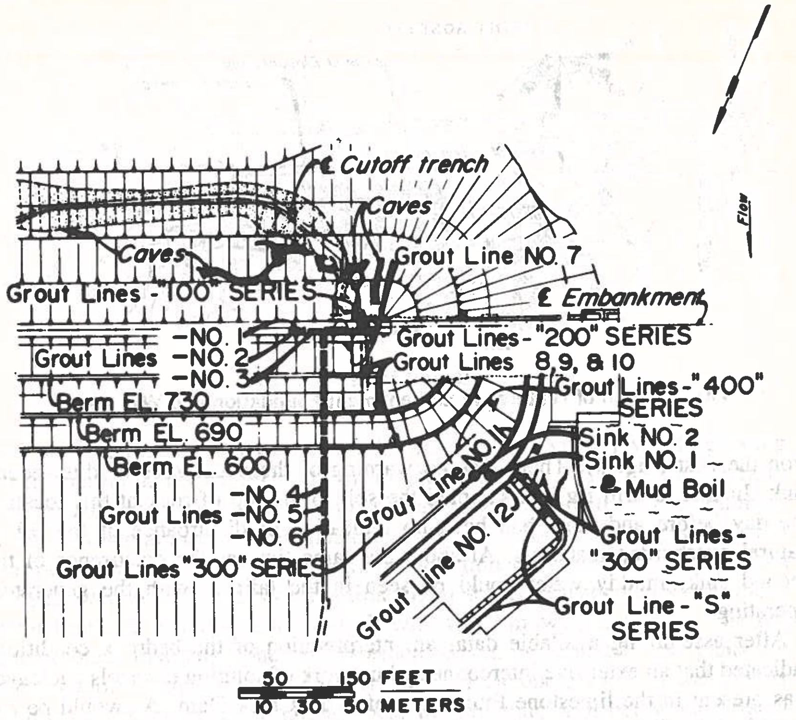

FIG. 5.—Plan of Emergency Grouting Lines

ment fill and alluvial overburden, were 6.75 in. (171.4 mm) in diameter. After continuous flight augers and air-drilling methods failed to advance holes through the fill, drilling fluids were used. As much as 10,000 gal (37,900 L) of bentonite mud was pumped into a single borehole while it was being advanced to the top of the rock. The remedy to this problem was to reduce the penetration rate of the drill bit to not more than 0.6 ft (0.18 m) per minute; to use only additives such as shredded newspaper, mica flakes, and polyethelene flakes in the natural mud mix; and to keep the weight of the mud below 72.5 lb/sq ft (353.1 kg/m^2). After a stable borehole was made to the top of the rock, a 4 in. (101.6 mm) casing was inserted and a 3 in. (76.2 mm) borehole was drilled with a tricone roller rock bit to El. 500 (152.5 m). No pressure testing was performed during this phase of the program.

Grouting.—A 1.5 in. (232.6 mm) diameter pipe was inserted to the bottom of the borehole in rock, and the grout was allowed to flow under gravity head. A pressure gage at the surface indicated if a blockage was created between the injection pipe and the walls of the borehole. When the grout was injected, the grout pipe was withdrawn as the rock section refused to accept grout. While continuing to introduce grout, the 4 in. (101.6 mm) casing and the grout pipe were slowly removed from the borehole. No attempt was made to limit grout travel, and, indeed, all boreholes were grouted to refusal generally without interruption. The unit take for grout line 1, the first line to be constructed, ranged

between 16 cu ft/lin ft (1.47 m^3/m) for primary holes (20 ft cc (6.1 m)) to 1 cu ft/lin ft (0.09 m^3/m) for quaternary holes (2.5 ft cc (0.76 m)). In general, the grout was composed of one part cement and one part water; one part sand was added to the mix when large takes were experienced.

After construction of the first six lines, additional grouting (Fig. 5) was performed along the berms and at the contact of the embankment with the concrete dam. The total program, which required about two years to complete, included 174,666 ft (53,273 m) of overburden drilling, 97,032 ft (29,595 m) of rock drilling, and 290,087 cu ft (8,214.3 m^3) of injected grout solids. Early in this program, the piezometric heads were reduced; in fact, upon grouting the second hole, which took 1,771 cu ft (49.6 m^3) of grout, the high pressures at the downstream toe of the embankment began to dissipate. The muddy flows also ceased.

In all probability, this emergency grouting program saved the dam from a major piping failure. However, the grouting had at best only filled the voids that were intercepted in the overburden and bedrock solution features; it could not be considered a long-term solution to the problem. The solution network was still partially filled with erodible soil that the grouting operations had not been able to treat. The seepage forces caused by the reservoir were still acting and could again result in an unsafe condition beneath the dam. Realizing the temporary nature of the solution provided by the grouting, the Corps of Engineers sought a more permanent solution, along with an evaluation of the embankment's foundation.

Diaphragm Wall (1972–1979)

In early 1972, a consulting board of non-Corps engineers and geologists was retained to review the problem and to make recommendations for treatment. A number of alternatives were considered including the following: a new dam and powerhouse at a downstream site; a new rockfill dam immediately downstream of present dam using existing powerhouse; a new concrete dam immediately downstream of present dam using existing powerhouse; a concrete diaphragm wall constructed from the downstream El. 660 (201.3 m) berm; a concrete diaphragm wall constructed from the upstream slope at el. 740 (225 m); a concrete diaphragm wall constructed along existing core trench; a tunnel at the top of the rock with a cóncrete diaphragm wall constructed into foundation; an upstream impervious blanket; and the selected scheme, a concrete diaphragm wall constructed from the upstream crest of the embankment to the base of all solution features encountered during exploration and construction.

Such a wall had never before been constructed to the depths necessary—in similar geologic conditions that were present (Appendix I), and with an existing pool. Diaphragm walls to depths greater than the 278 ft (84.8 m) maximum depth at Wolf Creek have been constructed, but never through 100 ft (30.5 m) of rock subjected to solution activity.

Originally, the wall was to be constructed to a depth of 5 ft (1.5 m) (a depth that appeared to be below all solution activity) into the Catheys Formation (Fig. 6), and from the concrete section toward the right abutment, a distance of over 3,000 ft (915 m). Ultimately, the depth of the bottom of the wall varied in elevation depending on conditions encountered during explorations. Linear ele-

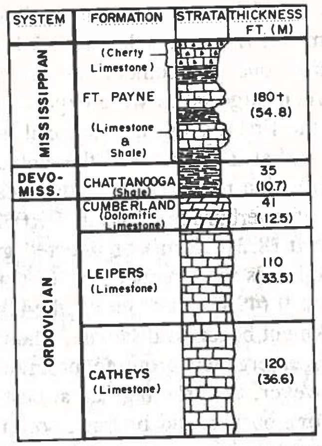

FIG. 6.—Generalized Geologic Column at Wolf Creek Dam

ments or panels could not be used to install the wall because of possible problems with instability of long, open excavations in the overburden and fill material. Such instability could result from the loss of stabilizing fluid should a large cavity be encountered during excavation. The hardness of the rock also discouraged the use of panel construction. It was concluded that the wall should have a minimum thickness of 2 ft (0.61 m), and that the single element method was the most feasible type of construction for Wolf Creek. Since this type of construction is expensive, and exploratory program was undertaken to determine if anything could reduce the amount of wall needed or the possible problems that could be associated with its installation.

Exploratory Method and Techniques

Before reviewing the specific exploration program performed to define the limits of the diaphragm wall and the conditions along its alignment, some of the more general methods used to determine the overall condition of the embankment should be examined. Some of these methods were used in the early periods of investigation, beginning in March, 1968.

Piezometers.—The single instrument most used in determining the condition of Wolf Creek Dam has been the piezometer. Over 325 of these devices have been installed since 1968; they are read on a regular basis. By far the largest number installed have been the open tube Casagrande type and the wellpoint type (11). For the most part, these instruments were installed with a cable tool or churn drill through the embankment and overburden sections of the dam to the top of rock. The next largest number were installed in discontinuities or cavities in the rock, and, finally, a smaller number were installed in the embankment. The piezometers indicated the originally high piezometric pressure downstream of the center line of the embankment and later confirmed that this pressure had been reduced by the grouting. In addition, further details about

subsurface conditions were gained as the piezometers were being installed.

In addition to the open-tube piezometers, 10 pneumatic and 10 electrical transducer piezometers (11) were installed in 1975 to measure any transient pressures that might be produced by constructing the wall. All of these instruments were set in rock, and one of each type was set at the same elevation in 10 different holes. The depth of installation ranged from 150–197 ft (45.7–60.1 m) below the dam's surface, and from 568 ft (173 m) to 586 ft (178 m) in elevation. Only seven of the 10 transducers were functioning after installation, and problems experienced with the 10 channel recorders used with the transducers never were completely resolved. There was a wide range of variation in indicated water levels between the two types of instruments, even though they were set at the same elevation. It appears that the pneumatic type performed more responsively than the electrical transducers. But, compared with the open-tube piezometers, both types were much more expensive to install and their reliability was questionable.

Temperature Survey of Subsurface Water.—After the occurrence of the sinkhole in March, 1968, temperatures were taken in all drill holes and all piezometers on a regular basis. A probe, 1/8 in. (3.2 mm) in diameter, was lowered to the bottom of the hole or piezometer to record the temperature to the nearest degree Fahrenheit (0.55° C). The normal temperature of the ground water for the area outside the influence of the reservoir was determined to be 59° F (15° C), and temperature profiles were run on the reservoir for various times of the year. The temperature of the lake was always below 55° F (12.8° C) at a depth of about 80 ft (24.4 m). When temperatures lower than 59° F (15° C) were measured in the piezometers and drill holes, it was assumed either that colder water had seeped in from the reservoir or perhaps that tailwater had intruded into the solution network. Temperature contours were drawn based on readings taken from holes drilled into the rock and from top of rock piezometers, and a zone of low termperature emerged around the end of the concrete section of the dam, down toward the sinkholes and into the tailrace. In general, this zone of low temperatures correlated with the alignment of solution features in the foundation rock.

Dye Tracer Tests.—In order to establish continuity and to obtain an estimate of the velocity of the subsurface flow, fluorescein dye was poured into holes drilled into rock in the area of the sinkhole. The solution contained about 1/2 pt (0.24 L) of dye powder mixed with 5 gal (19 L) of water. They dye was easily recognizable as it exited into the tailrace at low tailwater. The greatest velocity calculated in several tests was about 18 ft (5.5 m) per minute. After the initial emergency grouting, dye was again poured into holes in the area of the sinkhole, but it was never observed in the tailrace.

Saline Tracer Tests.—An attempt was made to establish continuity among the drill holes in rock by pouring a salt water solution into one hole and attempting to measure an increase in electrical conductivity in adjacent holes with the downhole single-point resistance probe of a geophysical logger. These tests were judged unsuccessful and were discontinued after only a few tries since high saline concentrations were measured in all holes. In retrospect, it is believed that the probe became contaminated after the first injection and that all subsequent readings were false.

Surface Resistivity.—The electrical resistivity method of shallow subsurface exploration was attempted during the early stages of exploration using the Wenner array. While others have reported some success in detecting subsurface solution features with this method (2), at Wolf Creek the results were judged to be negative, probably in part because the mass of the embankment over the area of interest was too great.

Echo Depth Sounding Survey.—An underwater survey, extending approximately 800 ft (244 m) beyond the toe, was conducted along the upstream face of the dam. A Model DE 719B Fathometer (manufactured by the Raytheon Marine Corporation) was used to profile the bottom on 5 ft (1.52 m) spacings normal to the center line. Contours were drawn and were compared with preconstruction and as-built data. Although the results of a survey such as this are not highly precise, any large depressions or holes can be detected since the accuracy of the equipment is +1 in. (25.4 mm). Several of these surveys, run over the years of investigation and treatment, indicated six areas of depressions that required further investigation.

Inspection by Divers.—In connection with the echo depth surveys, scuba divers investigated any feature of the bottom that appeared unusual. Visibility was limited to less than 1 ft (0.305 m); therefore, a great deal of the inspection was conducted by touch. This type of inspection was sufficient to identify all the depressions indicated by the echo depth surveys, as well as one area of possible inflow (where the divers said they could feel the water moving). A large quantity of fluorescein dye was placed in the area; however, the dye was never observed downstream.

Underwater Television Survey.—A Model 125 television system was leased from Hydro Products of San Diego, California, for use in an underwater survey. The camera was reputed to be able to "see" better than the human eye because its light source was a 250 W thallium iodide lamp, which emits light energy in the region of maximum transmission in water and also in the region of maximum response of the television camera's vidicon. The equipment functioned very well; however, because of the murkiness of the reservoir's water, the distance at which it could clearly see an object was only about 6 in. (152 mm). Obviously, an overall view of an area of the bottom was not possible, rendering this survey of little or no value.

Remote Sensing.—Panchromatic and color infrared air photography and thermal infrared imagery were also used in the studies at Wolf Creek. The imagery was obtained from an altitude of 60,300 ft (18,392 m). A Zeiss mapping camera with a 12 in. (0.305 m) focal length was used for the color infrared photography, while the thermal imagery was in the 10–12 μ range and was printed as a 35 mm thermogram. Although these studies did reveal some overall aspects of the geology of the area (such as the jointing and drainage pattern), and the wet areas near the downstream toe toward the right abutment did stand out very well, no new information was gained from any of the imagery.

Test Excavation.—Just beyond the downstream toe of the embankment, and toward the right abutment in an area where the bedrock was known to be less than 5 ft (1.52 m) deep, an excavation was constructed to top of rock. This backhoe and hand labor excavation was made to determine the solution feature orientation and condition that might be present. Top of rock was found to be

cut by solution channels 1 ft (0.305 m) or so wide and about the same depth. The measurements of five different joint orientations coincided, in general, with what was expected from studies of the area and from the construction of the cutoff trench (Fig. 4).

Thermonic Survey.—A reconnaissance thermonic survey was conducted as a demonstration by Hydrotechnics of Albuquerque, New Mexico. The survey was made by accurately determining the thermal gradients in 140 piezometers that were in place at the project in early 1971. Most of these piezometers were set at the top of the rock, although a few were in the embankment and foundation rock. From this and other information, it was claimed that the zones of major seepage could be defined. The flow rate was reported to be as much as 10 times greater in those major seepage zones than in the rest of the foundation, even though an absolute flow rate could not be established. The zones of seepage, as defined by the survey, were consistent with the slope of the potentiometric surface and coincided very well with the known orientation of solution features in the bedrock.

Exploration of Wall Alignment

In 1974, the decision was made to construct a diaphragm wall, and explorations were also undertaken to define its limits. It was important to determine whether solution features—even of limited lateral extent—crossed the alignment of the wall. If some method could be found capable of indicating conditions between boreholes on reasonably wide centers, large amounts of exploratory funds could be saved.

Acoustical Surveying.—Holosonics Incorporated of Richland, Washington, was contracted to survey 21 holes using two different methods: pulse-echo and through-transmission. The pulse-echo method used a probe consisting of one transmitting and one receiving element placed in a single borehole and operated on a narrow band-width at 22 KHz. Reflected signals from interfaces within the 100 ft (30.5 m) range of the system were picked up and displayed as an oscilloscope trace, giving the elapsed time and amplitude of each signal. There was difficulty, however, in distinguishing signals reflected from cavities and the signals reflected from vertical joints and bedding planes.

The through-transmission method used probes in two adjacent drill holes spaced between 20 and 30 ft (6 and 9 m) apart. The 22 KHz probe was used to transmit the signal, while both probes were raised at 1 ft (0.305 m) intervals as the hole was surveyed. The p-wave velocity was 14,000–16,000 ft/sec (4,270–4,880 m/sec) in the shaley limestone, and 18,000–21,000 ft/sec (5,490–6,405 m/sec) in the harder limestone beds. The reliable limit of through-transmission was established at 30 ft (9.15 m).

After surveying the 21 holes, it was determined that this method had potential only in rock and not in embankment or overburden. Cavities were difficult to distinguish from joints, bedding planes, and low velocity shale beds. Although these methods had some potential for detecting cavities, they were not definitive enough nor reliable enough to establish the limits of the diaphragm wall at Wolf Creek.

Downhole Logging.—Several downhole logging methods were used, including spontaneous potential, single-point resistance, natural gamma, gamma-gamma, neutron, temperature, short and long normal and lateral resistivity, caliper, televiewer, and 3-D acoustic velocity (4). Correlation between the geophysical data and the core logs was very good. Cavities intersecting the borings were depicted by the caliper, gamma-gamma, neutron, 3-D acoustic velocity, and televiewer tools. The stratigraphy was readily correlated from hole to hole; however, clues as to the presence of cavities beyond the walls of the borings did not materialize even with the wide variety of methods used.

Deviation Surveys.—In order to determine if the borings being drilled to explore the alignment of the diaphragm wall were actually sampling the interval desired, and in order to determine the distance to use between holes in the cross-hole seismic surveys, deviation surveys were conducted in 66 of the holes. The equipment used was obtained from Eastman Whipstock of Houston, Texas, and consisted of both multishot and single-shot tools. The gyroscopic multishot surveys were run by Eastman personnel in five holes and were presented as a continuous log of direction and amount of deviation. The single-shot tool was leased, and the surveys were conducted by government personnel in 61 holes. This type of survey is time-consuming, since the tool has to be removed from the hole and reloaded for each measurement made. The single-shot tool also used a magnetic compass to measure direction of diviation. The compass was affected by the presence of the powerhouse and did not give accurate readings of direction; however, the amount of deviation with depth could be measured.

The surveys indicated that the 3 in. (76.2 mm) diameter, NW wireline holes averaged about 2 ft (0.61 m) of deviation for every 100 ft (30.5 m) of hole, with most in the fill and overburden above the top of the rock. The maximum deviation measured was 20 ft (6.1 m) in one 200 ft (61 m) hole. There was no apparent pattern to the deviation.

Drilling and Grouting.—By far the most effective method of exploration, and the one that was most depended upon for establishing the limits of the diaphragm wall, was the drilling and grouting program (8–10). This program, carried out over a two-year period beginning in June, 1973, consisted of exploratory drilling and grouting along the alignment of the proposed wall. Primary holes were on 25 ft (7.6 m) centers carried about 50 ft (15.2 m) into the Catheys

TABLE 1.—Test Grout Mixes

Mix Proportions: Cement (1)	Mix Proportions: Clay (2)	Mix Proportions: Sand (3)	Mix Proportions: Water (4)	Marsh funnel viscosity, in seconds (5)	Unit weight, in pounds per cubic foot (6)	Shrinkage, as a percentage (7)	U_c, in T per square foot 7 day (8)
1	1	6	2	35	108.3	15.8	6.0
1	1-1/2	8	3	51	113.7	4.9	8.9
1	1-1/2	8	4	42	113.6	5.4	5.4
1	1-1/2	10	4	39	115.8	8.6	4.6

Formation (Appendix I) to El. 475 (144.9 m), approximately 300 ft (91.5 m) deep. The holes were split-spaced down to 3-1/8 ft (0.95 m) and gravity-grouted (as described earlier for the emergency grouting program of 1968). Since the wall was to be constructed along the same alignment as the grouting, it was decided to grout above top of rock with a mix that was comparable in strength to the embankment fill. Test mixes were prepared, cured, and tested in a compression machine to determine their unconfined strengths. The mixes and strengths determined from the tests are given in Table 1.

The third mix described in Table 1 was chosen for grouting above the top of the rock. The clay used was a CL with a liquid limit of about 45 and was processed from formations in the Paris, Tennessee, area by H. C. Spinks Co. The sand was a medium-fine sand with about 55 % passing the No. 40 sieve. It was processed by crushing a natural rock formation, the Sewannee Conglomerate. The mix containing 10 parts sand could have been used, but the selected mix had a little over 3% less shrinkage, and it was believed that it could also be batched and pumped with less difficulty. A mixture of one part cement, one part sand, and one part water was used in the rock section of the holes.

A total of 852 holes were drilled along the alignment; most of them were cored with NQ wireline tools at the election of the drilling contractor. Fifty-one holes took over 1,000 cu ft (28 m^3) of grout before refusal; the maximum grout injected into any hole in rock was 6,971 cu ft (195.2 m^3). The total footage drilled was 239,460 ft (73,035 m), and the total grout injected was 146,461 cu ft (4,100 m^3), for a unit take of 0.61 cu ft/lin ft (0.056 m^3/m) of hole. Although these quantities indicated the overall condition of the embankment and its foundation, when the details of the program were studied, the different conditions at specific reaches could be identified. An example of some of the more serious solution features encountered in rock along the alignment are shown on the geologic profile (Fig. 7).

Exploration Costs.—The costs incurred from various exploration programs associated with the Wolf Creek diaphragm wall are given in Table 2. It readily can be seen that large sums of money have been expended. Indeed, the question

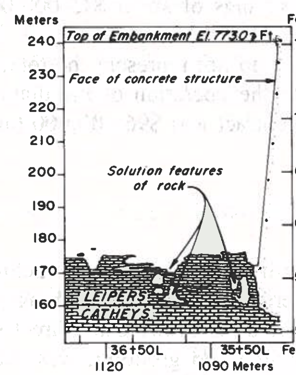

FIG. 7.—Section along Diaphragm Wall Alignment Showing Solution Features of Bedrock

TABLE 2.—Cost of Exploration

(1)	Date (2)	Dollars (3)
Government subsurface exploration	1973–1975	252,520
Downhole geophysical logging	1973–1975	50,080
Acoustical survey	1973	20,000
Deviation survey	1973	3,756
Television survey	1972	3,723
Echo depth sounding and diver survey	1974	4,085
Thermonic survey (demonstration)	1971	0
Remote sensing (other gov't agency)	1972	0
Drilling and grouting (contract)	1973–1975	1,620,796
Subtotal		1,954,960
Government inspection and evaluation	1972–1975	1,666,926
Total		$3,621,886

TABLE 3.—Comparability of Wall Costs with and without Exploration

(1)	(2)
Wall as proposed before exploration (bid price)	$69,253,000
Wall as constructed after exploration (bid price)	50,715,000
Difference	$18,538,000
Cost of all exploration	−3,621,886
Net difference	$14,916,114

might well be asked, "Has all this expensive exploration been worthwhile?" A comparison of the estimated costs of the wall before exploration and the costs as bid is given in Table 3. The difference in costs is the result of raising the bottom of the wall to a higher elevation since the cost both before and after exploration is computed at the actual bid price per square foot of wall. This comparison indicates that a savings of about $15,000,000 was realized by the extensive exploration.

The costs shown in Table 3 do not represent the total costs of the wall since many items were common to the operation of installation, notwithstanding the depth. The total cost of the contract was $96,400,000 (all costs given are actual for the dates performed).

Conclusions

The problems associated with designing and constructing a concrete diaphragm wall at Wolf Creek Dam were unique in that nothing of this nature had been attempted in a dam with a filled reservoir. Only direct subsurface information, such as that gained from drilling and grouting, was acceptable in establishing the limits of the wall and the conditions along the alignment. Several advantages of this type of exploration were realized. As the holes were drilled and systematically backfilled, any voids in the embankment or foundation were immediately filled. The program also resulted in fewer problems during construction since, as the voids along the alignment were filled, the area was consolidated. Since subsurface information was available on such close centers, there were no disputes as a result of changed condition claims. The wall was successfully completed without major problems about nine months ahead of schedule and $517,000 under bid price for that portion of the contract directly related to constructing the wall. Since completion in September 1979, the wall has performed satisfactorily.

Appendix I.—Site Geology

Wolf Creek Dam is situated in the Highland Rim section of the Interior Low Plateau. The area is underlain by flat lying beds ranging in age from Ordovician to Mississippian. The axis of the northeast-southwest trending Cincinnati arch is located about 20 miles (32 km) west of the dam, giving rise to a slight dip of the beds to the southeast. The Fort Payne formation, a shaley limestone, caps both abutments at the damsite (Fig. 6). Underlying the Fort Payne is the Chattanooga shale which rests unconformably on the Cumberland dolomitic limestone.

These formations do not presently contribute to the underseepage problems at Wolf Creek Dam, but they played an important role in the karstic history of the site. The lower reaches of the valley at the dam site are underlain by 230 ft (40.2 m) of the Leipers and Catheys Formations. These Ordovician beds are made up primarily of argillaceous limestones ranging from 49–82% calcite, which renders them highly susceptible to the development of solution features that may act as paths for ground water movement along joints and bedding planes. Below these formations is the Cannon limestone, which is also susceptible to solution activity; however, exploration indicates that most of the karstic activity terminated at contact with the upper Catheys, about El. 525 (160 m) which is 10 ft (3 m) below stream bed.

The original river channel was approximately 300 ft (91.5 m) wide at the dam site and flowed essentially on the Leipers formation at the base of the steep left abutment. On the right side of the river as much as 50 ft (15.3 m) of alluvial valley fill was present near the stream, but, in general, the irregular top of rock was covered with 10–20 ft (3–6 m) of fine silty sand for about the first 3,000 ft (915 m) away from the river. Much of the fine silty sand was blanketed with a lean clay from 10–30 ft (3–9 m) in thickness, which extended all the way to the right valley wall. For a more complete treatment of the geology, see Kellberg and Simmons (5).

Design and Construction of Embankment (1938–1951)

Design of the Wolf Creek Project began in 1938, and construction commenced in August, 1941. Because of World War II, construction was interrupted in August 1943, but not before the completion of nearly all foundation work for the embankment, including the cutoff trench and grouting, but excepting the tie-in to the concrete section of the dam. Work resumed in 1946, and the embankment was completed in June, 1951.

The embankment was designed as a homogeneous impervious section and was constructed from the stratified alluvial deposits in the flood plain both upstream and downstream of the dam site. The more than 10,000,000 cu yd (7.65×10^6 m^3) embankment has a top elevation of 773 (235.8 m); side slopes range from 1V on 2H, to 1V on 3 − 1/2H, with 1V on 10H random fill berms at both the upstream and downstream toes (Fig. 8). The embankment is composed primarily of a low to medium-plasticity clay; it contains lesser amounts of silty sand and high-plasticity clay.

Cutoff Trench.—Design exploration made clear that an interconnected network of solution features existed in the Leipers Formation that comprises the embankment foundation. Cavities in the Leipers Formation, ranging from just below top of rock to as deep as 75 ft (22.9 m) (El. 500 (152.5 m)), had been encountered in the borings, many of which extended to depths 100 ft (30.5 m) below the solution activity. These cavities ranged from small solution features along bedding planes and high angle joints, to features as much as 12 ft (3.7 m) in height and of unknown lateral extent. Because of these solution features and the stratified alluvial overburden, a cutoff trench was designed to be constructed at about three-fifths of the distance from the center line to the upstream toe of the embankment. This cutoff trench was to be carried to sound rock with a single-line grout curtain extending below to a depth of 50 ft (15.3 m) for the entire length of the embankment. Soon after excavation began, large solution features in the Leipers Formation were encountered that roughly followed the alignment of the designed cutoff. These features were excavated to a general sound rock floor and were used as the cutoff. Although this procedure resulted in a somewhat zigzag alignment, most of the solution features would be cut off by constructing the trench in this manner (Fig. 4).

The trench had a minimum bottom width of 10 ft (3 m) and was backfilled with lean clay, the same type of material as comprised the embankment. As a result of constructing the cutoff within a system of solution features, very irregular and sometimes vertical and overhanging side slopes resulted and were left in place. At some locations, the bottom of the trench was as much as 60 ft (18.3 m) below top of rock. In general however, the depth of the trench was about 45 ft (13.7 m) below top of rock for the first 1,500 ft (457.5 m) from its tie-in with the concrete section of the dam. A large number of stratified silt-filled solution channels and caves intersected the trench on both upstream and downstream sides. These features followed the orientation of the joint sets present in the area (Fig. 4) and received no special treatment during construction.

Drilling and Grouting.—As designed, a single-line grout curtain was constructed from the bottom of the cutoff trench to a general depth of about 50 ft

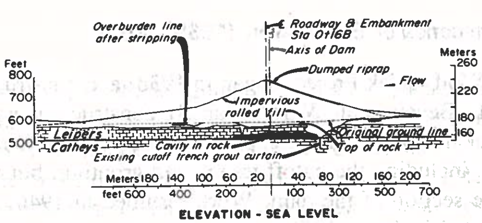

FIG. 8.—Cross Section of Embankment near Concrete Section

(15.3 m), which resulted in the bottom of the grout curtain being very irregular and not penetrating to any constant elevation or stratigraphic horizon. Since the grouting was performed from the bottom of the cutoff trench, most of the solution features that would have accepted large quantities of grout were never encountered—they were above this level. All grout holes were cored 2 in. (50.8 mm) in diameter and angled toward the right abutment at about 12 degrees from vertical. Primary holes were drilled and grouted on 10 ft (3 m) centers and split-spaced to 5 ft (1.5 m) centers, if the adjacent hole took more than 10 sacks of cement.

Six hundred holes were drilled along the curtain for a total of 32,671 lin ft (9,964.7 m). A total of 20,387 cu ft (570.8 m^3) of cement was injected into the curtain for an average grout take of 0.6 cu ft/lin ft (0.056 m^3/m) of hole. Some areas along the grout line took quantities very much in excess of the average, however. In one of these areas, where the trench bottom stepped up to near the top of rock and continued at this elevation for about 300 ft (91.5 m), 39 holes took 12,005 sacks of cement for an average take of more than 4 cu ft/lin ft (0.37 m^3/m) of hole. This high grout take no doubt resulted from these holes' penetration of the upper 50 ft (15.3 m) of rock, whereas most of the rest of the curtain was constructed from the bottom of the cutoff trench, which was below the upper section of rock that contained most of the solution activity. (For additional analyses of the project, Couch (1) and Fetzer (3) are recommended.)

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