The History of Grouting at Wolf Creek Dam, Kentucky

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ABSTRACT

USACE's Wolf Creek Dam comprises a 1,796-foot-long concrete gravity section, and a 3,940-foot-long embankment section, founded mainly on karstic limestone. The initial foundation grouting of the dam was conducted between 1943 and 1949. Dam construction was completed in 1952, but by 1967 the safety of the dam was severely threatened by erosion of materials from the massive karstic features. This prompted a huge emergency grouting project, from 1968 to 1974, which arguably saved the dam prior to a concrete cutoff wall being installed in 1975-1979. By 2005, however, the conditions of the foundation had again deteriorated, and that year Wolf Creek Dam was rated in the highest-risk category by USACE. A third grouting campaign was conducted in the limestone in 2007 and 2008 along the length of the dam, using modern drilling and grouting methods. As part of the definitive remediation of 2009-2013 involving the construction of a longer, deeper, concrete cutoff wall, further extensive grouting, targeted in specific areas, was conducted to facilitate safe construction of the cutoff wall, and to extend its influence. A fifth grouting campaign, performed in 2011 and 2012, was executed along the entire grouting gallery in the concrete gravity section, and also along part of the downstream toe of the embankment, near the hydroelectrical powerhouse.

This paper provides a summary of each of these five remarkable phases of work, and highlights the practices followed at each time.

INTRODUCTION

Wolf Creek Dam is located in south-central Kentucky with the nearest town being Jamestown, KY (Figure 1). This U.S. Army Corps of Engineers (USACE) dam comprises a 1,796-foot-long concrete gravity section, and a 3,940-foot-long embankment section (Figure 2), founded, in part, on karstic limestone. Wolf Creek Dam retains 6-million-acre-feet of water at full reservoir level. This makes it the largest reservoir, by volume, in the eastern United States, and is the ninth largest pool, by volume, in the entire United States (Simmons, 1982). The dam was built during two periods, from 1941-1943 and from 1946 -1952, interrupted by World War II. Clearly the design was based on principles developed in the 1930's.

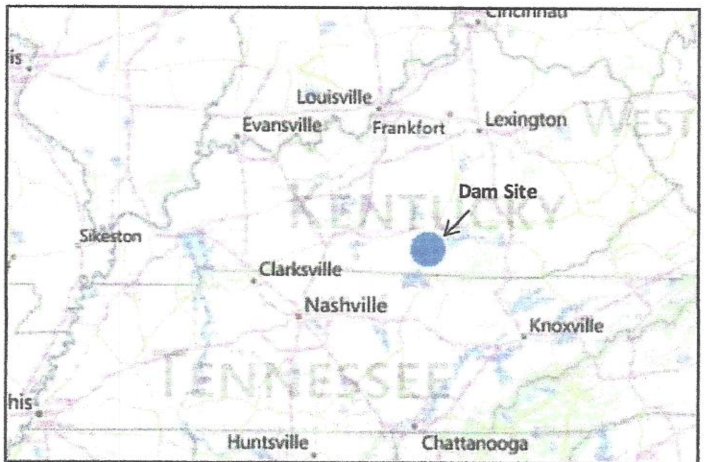

FIGURE 1. Project Location Map (Source: Bing Maps)

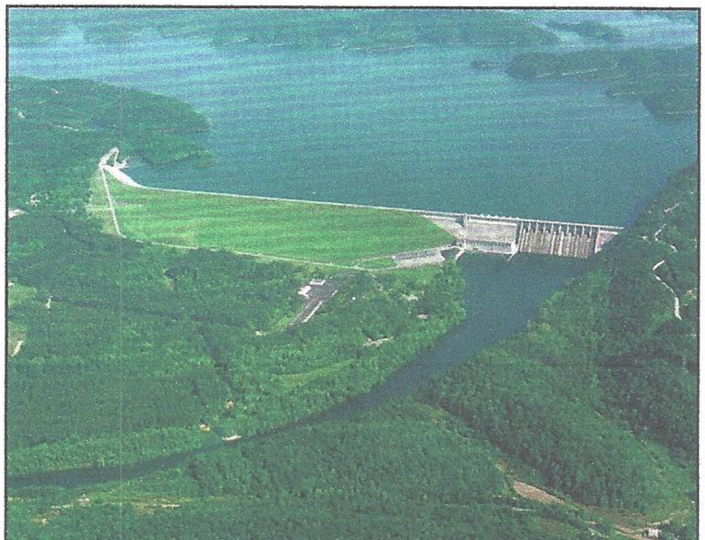

FIGURE 2. Aerial View of Wolf Creek Dam showing the concrete gravity section (including powerhouse) on the left abutment and the embankment section to its right (Source: USACE)

The dam is founded on Ordovician-age limestones (Kellberg and Simmons, 1977). The abutments include younger rocks of Silurian to Mississippian age, but they make up a very small portion of the foundation. Figure 3 summarizes the stratigraphy at the dam site. The vast majority of the grouting onsite has been in the Liepers Formation, with many holes terminating in the upper Catheys Formation.

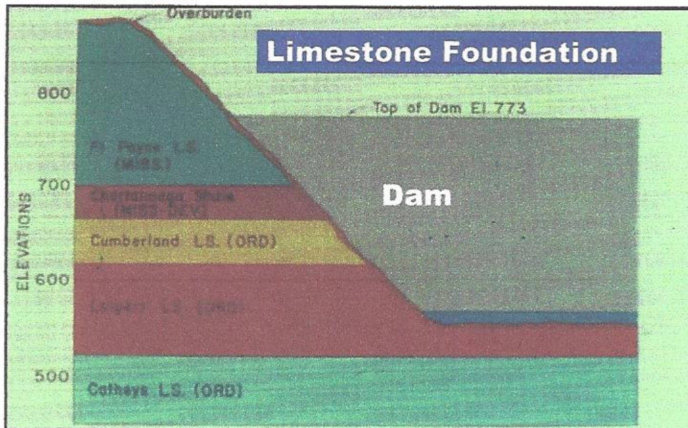

FIGURE 3. Summary of site geology. View of the site stratigraphy at the left abutment / concrete dam interface, looking downstream (Source: USACE)

There is extremely well developed karstification restricted to the Leipers Formation. A deeply incised collapsed cavity sits along the upstream heel of the dam, which was utilized to create the core trench, and which ties into the concrete dam. A number of cavities intersect the core trench alignment, one of the largest being shown in Figure 4. During foundation preparation, the contemporary standard of care in the 1940's was to remove the cavity infill material, but to then replace it with core-like silty clay. The compaction of this backfill would have been of dubious quality under the overhangs created in the karstic features, and it was subject to erosion and removal under service conditions. Figure 5 shows the location of cavities detected by grout holes prior to 2006.

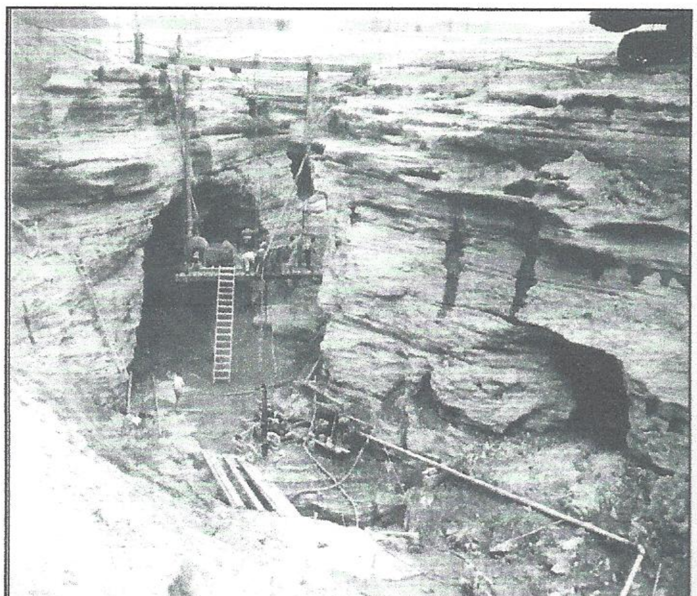

FIGURE 4. Karstic cavities intersecting the core trench alignment at Wolf Creek Dam as revealed during original construction (Courtesy of USACE).

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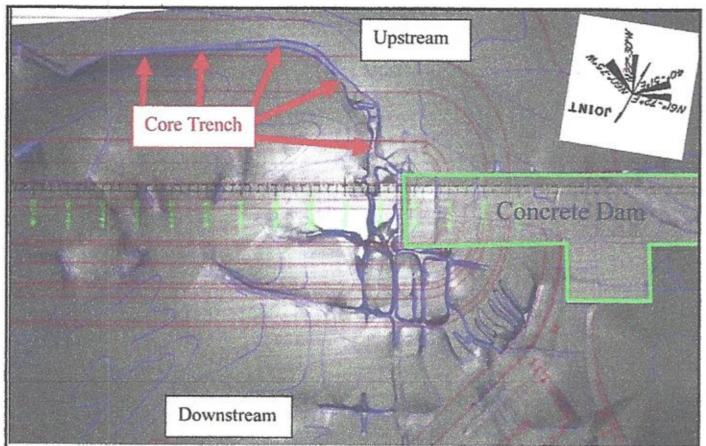

FIGURE 5. Karstic development trends. This shaded relief map is based upon data from pre-2006 grout holes. A joint rosette shows the orientation of joint systems. (Source: USACE)

Rock mass grouting at Wolf Creek Dam was conducted in five main phases: 1942-1943 and 1948-1949; 1968-1971 and 1973-1975; 2007-2008; 2009-2011; and 2011-2012.

Detailed information about the dam, its foundation, and its construction is provided in excellent papers by Kellberg and Simmons (1977), Fetzer (1979), Simmons (1982), and Mackey and Haskins (2012).

PHASE 1: INITIAL CONSTRUCTION GROUTING (1940's)

Core trench grouting began on January 12, 1942 and was completed on August 10, 1943 (Mackey and Haskins, 2012). Six hundred grout holes were installed in a single line along the 4,380 feet long core trench (located on the upstream heel) generally on 10-foot centers for a total linear footage of 32,671 feet. Holes were typically 50 feet deep, but 112 of the borings were deepened to accommodate geometry. Figure 6 shows a cross-section of the cutoff trench and grout curtain. The majority of the holes were angled 12° right: the remainder were drilled vertically.

Grouting was conducted with the downstage method. Neat cement grouts were used, composed of Portland cement and water. Most of the grout was placed using a mix of 1 bag of cement (94 lbs.)

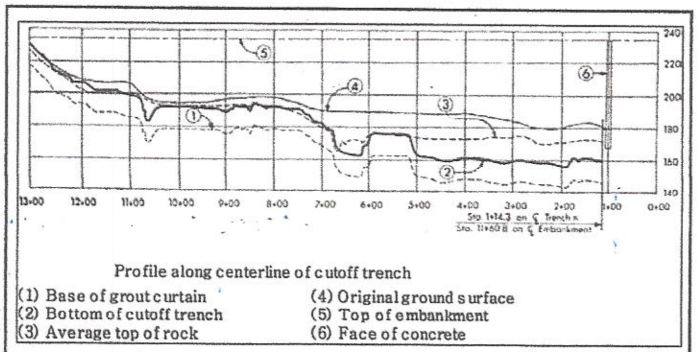

FIGURE 6. Original embankment grout curtain, looking upstream (Fetzer, 1979)

to 7.5 gallons of water (i.e., water:cement ratio of 0.66 by weight although the range was 0.88 to 0.44). When especially high takes were encountered, round silica sand was added at the direction of the USACE inspectors. Grouts were mixed by air-driven paddle mixers and injected by air-powered, double action piston pumps at pressures of 1 psi per foot depth. Permeability testing was conducted at similar pressures, but curtain closure was based on grout takes, not Lugeon values. Holes consuming over 940 lbs. of cement were split-spaced by the addition of two adjacent holes. In total, 20,387 bags of cement were used to inject the 600 holes. Approximately 30% of the grout line required split-spacing to less than 10-foot centers.

The grout holes were 2 inches in diameter and were cored. The core was logged but not saved. No overburden was drilled.

Gallery drilling and grouting was conducted in 1948 and 1949. A gallery tunnel within the concrete monoliths was built to allow grout holes and relief wells to be installed. Minimum hole spacing was 10 feet along the length of the grouting gallery, but actual hole spacing varied due to split-spacing. Steel casings for both grout holes and drain holes were drilled a few feet into sound rock and installed in the concrete as the monoliths were placed. Casings for grout holes were 3-inch diameter, and for drain holes were 5-inch in diameter.

Initial gallery grout holes were oriented towards the west (left abutment) at a 7° angle and were drilled near the upstream side of the gallery. Grout holes were cored at least 2 inches in diameter. All grout holes were drilled to a fixed depth of 75 feet, and were stage drilled in three successive 25-foot-long downstages.

Gallery drain holes were drilled after gallery grouting was completed. These drain holes were oriented 12° downstream, from the downstream side of the gallery. Drain holes were 4 inches in diameter below the metal standpipe.

Neat cement grouts were used as for the earlier core trench work, and it appears from some field books that up to 1 psi per foot of pressure was used.

Holes were grouted using the split-spacing method, based upon primary holes at 20-foot spacing. Particular holes were further split-spaced up to quaternary level (2.5 feet between holes). A total of 3,860 bags of cement (362,840 pounds) was used in 20,025 linear feet of grout holes. Detailed information about the original gallery grouting is limited to internal reports.

PHASE 2: EMERGENCY GROUTING (1968-1971) AND PRE-CUTOFF GROUTING (1973-1975)

The emergency grouting conducted by the USACE was performed in response to rapidly deteriorating foundation conditions beneath the embankment. Muddy water was observed in the river adjacent to the powerhouse, suggesting the presence of piping conditions. Two sinkholes appeared at the downstream toe of the dam, just upstream of the electrical switchyard, one in 1967 and one in 1968 (Figure 7). The sinkholes were only about 100 feet from each other. Water could reportedly be heard running in the bottom of the sinkholes.

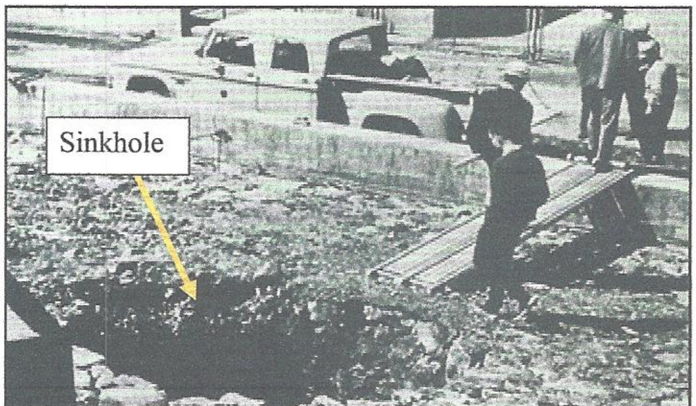

FIGURE 7. Sinkhole that led to emergency grouting (Source: USACE)

A first campaign of emergency grouting involving several grout lines was conducted from 1968 to 1971 (Figure 8). All lines were concentrated on the area of the dam where the concrete dam meets the embankment section and all lines were split down to 2.5-foot centers, regardless of grout take. The definitive description is that of Simmons (1982) from whose publication much of the following is derived.

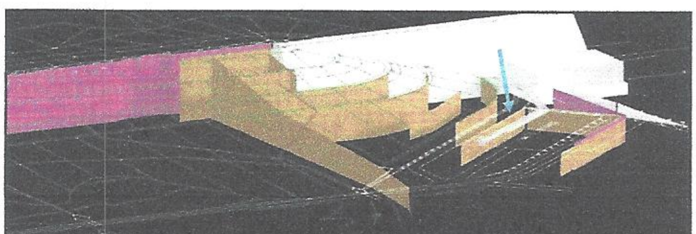

FIGURE 8. A 3-D view of the grout lines (brown) installed during emergency grouting; view is looking upstream. Blue arrow shows location of sinkholes. (Source: USACE)

The grout holes were drilled using mud-rotary techniques through the embankment fill and alluvial overburden, and were 6.75 inches in diameter. The drill penetration rate was limited to 0.6 feet per minute. After a stable borehole was achieved to the top of the rock, a 4-inch (101 mm) casing was inserted and a 3-inch borehole was drilled with a tri-cone roller rock bit to EL 500.

All grout holes were gravity grouted. As documented in inspection reports, "A 1.5 inch 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 inch 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." Simmons (1982) also states, "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."

"The total program, which required about two years to complete, included 174,666 feet of overburden drilling, 97,032 feet of rock drilling, and 290,087 cubic feet of injected grout solids." Apparently, this huge undertaking was judged to have "held" the situation long enough for further "permanent" measures to be considered, and in fact, a second phase of this grouting campaign followed from 1973 to 1975 as a prelude to the construction of the revolutionary concrete cutoff wall described by Fetzer (1979), and others. Again, the definitive account of the grouting is that of Simmons (1982).

He states, "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 feet centers carried about 50 feet into the Catheys Formation to EL 475, approximately 300 feet deep. The holes were split-spaced down to 3-1/8 feet" (i.e., to Quaternaries). Given the depth, and the deviation which must have occurred, however, one must not envision a perfectly regular, parallel array of equally-spaced boreholes in the rock at depth.

All holes were gravity backfilled, as was done during the previous (1968-1971) campaign. "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." The mix chosen had 1 part cement, 1.5 parts clay, 8 parts sand and 4 parts water, presumably by volume (Simmons, 1982). This mix yielded a Marsh funnel viscosity of 42 seconds and a unit weight of 113.6 pcf. "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 total alignment; most of them were cored with NQ wireline tools at the election of the drilling contractor. The total footage drilled was 239,460 feet, and the total grout injected was 146,461 cubic feet, for a unit take of 0.61 cubic foot/linear-foot of hole." Simmons (1982) also noted, "Fifty-one holes took over 1,000 cu feet of grout before refusal; the maximum grout injected into any hole in rock was 6,971 cubic feet." The locations of the grout lines (and the subsequent cutoffs) are shown in Figure 9.

FIGURE 9. Generalized alignment of the 1973-1975 grout lines (red) and cutoff walls (yellow), installed 1975-1979 (Source: USACE)

Based on these descriptions, it is clear that the technology of drilling and grouting had changed little from the 1940's to the 1970's. Paddle mixers were used to mix unstable, neat cement grouts, some with sand. The biggest change was to the drilling, with destructive techniques (rock roller bits) used in addition to coring. The change to the use of gravity grouting in Phase 2 was simply a reflection of the perceived vulnerability of the dam, and so the goal was to minimize effective grouting pressures.

As noted above, the second campaign of the Phase 2 grouting was the prelude to the implementation of the first "permanent" remediation. An Advisory Panel adjudicated on numerous proposals from international sources, ultimately recommending the concrete cutoff wall approach of the ICOS Corporation. This was the first concrete diaphragm cutoff ever installed through and under an operational embankment dam, and its construction featured unprecedented standards of quality control and quality assurance (Bruce, 2012). It must also be noted, however, that the Panel was not fully in accord regarding the optimal length and depth of the cutoff, while at least one eye had to be kept on the equally unprecedented cost. In any event, two walls were installed by ICOS in the period 1975-1979, one along the crest of the dam: 2,200 feet long, of variable depth (maximum 280 feet, but typically only 10 feet into bedrock) and a supplementary wall along the downstream toe in the switchyard 600 feet long and primarily 95 feet deep (stepping up in depth as it connected to the powerhouse).

There is no question that the interventions of the 1960's and 1970's had a highly beneficial impact on the foundation conditions and hence the performance and stability of the dam, and these provided an acceptable level of safety until the early 2000's when signs of further deterioration and distress were noted.

PHASE 3: INTERIM RISK REDUCTION MEASURES (2007-2008)

In 2006, the safety of Wolf Creek Dam was reviewed and evaluated in accordance with the USACE's recently developed Dam Safety Action Classification (DSAC) System and it was identified as a DSAC I dam. According to the rating system, a DSAC I dam is characterized as "Critically Near Failure" or "Extremely High Risk." Listed actions items for a DSAC I dam include the following (USACE ER 1110-2-1156):

- Take immediate action to avoid failure.
- Implement Interim Risk Reduction Measures (IRRM).
- Expedite investigations to support justification for remediation using all resources and funding necessary.
- Initiate intensive management and situation reports.

One IRRM that was quickly implemented was an extensive grouting campaign, in three sub phases:

- A 3,840 ft-long two-line grout curtain and foundation pre-treatment program, located along the alignment of the proposed concrete cutoff wall (Station 33+60 to Station 72+00).
- A 200 ft-long single line foundation grout curtain, drilled from the east end gallery of the concrete monolith section of the dam (Station 31+50 to 33+50).
- A foundation exploratory hole and instrumentation installation program along the core trench and dam embankment.

Project Design Concepts

The primary goal of the Phase 3 program was to reduce seepage flow through the foundation to reduce the risk of embankment failure and to reduce the risk of slurry losses in subsequent cutoff wall construction. However, additional goals of the Phase 3 program included implementing a subsurface exploratory program to obtain critical subsurface information that would be later used in the design and development of the cutoff wall and subsequent foundation grouting programs.

Phase 3 of the grouting operations represented a significant shift in the overall scope, design approach, materials, equipment, means, and methods used when compared to the two previous phases. The USACE incorporated the newest technologies recently developed in the grouting industry to better understand the subsurface conditions that existed at the site and to better treat the problems identified. The contract was awarded in late 2006 to Advanced Construction Techniques, Ltd., with Gannett Fleming, Inc. providing engineering services and technical assistance.

Work Platform, Hole Layout and Orientation

Work began in early 2007 with the construction of a 60-foot-wide work platform that stretched along the entire upstream slope of the embankment section, plus the transition zone between the concrete monolith section and embankment section (Figure 10).

FIGURE 10: Aerial view of concrete work platform for the Phase 3 works

The concrete capped work platform was to be constructed and utilized for both the Phase 3 foundation grouting contract and the subsequent cutoff wall contract. The construction of the concrete work platform facilitated hole layout and construction as well as moving equipment and materials. This allowed for a very fast and efficient means of conducting all phases of the drilling and grouting operations, including overburden drilling, overburden casing installation, rock drilling, in-situ permeability testing, and grout injection. Furthermore, the concrete work platform provided a clean, dry, and safe working environment to conduct the various grouting operations, many of which use water and produce spoils which would otherwise hinder overall production if not cleared away quickly and efficiently.

Grouting along the upstream embankment consisted of two grout lines, one located upstream and the other located downstream of the future cutoff wall. The two lines (designated U-Line and D-Line for their respective upstream and downstream locations) were located 24 feet apart, or 12 feet offset from either side of the proposed cutoff wall alignment. Holes were oriented 10° from vertical, angled ahead station (toward the right abutment) on the U-Line and angled back station (toward the left abutment) on the D-Line. The inclined nature of the grout holes increased both the probability and frequency of intercepting near vertical joints and features that were suspected to exist in the rock foundation. Grout holes drilled along the 200-foot section of the gallery (designated as G-Line) were also oriented 10° from vertical and were angled ahead station. The G-Line was located approximately 30 feet downstream of the proposed cutoff wall and included four separate fan lines that arranged up to 7 holes each in a fan-like pattern perpendicular to the grout line and projecting upstream and downstream of the dam. Holes were completed by split-spacing methods, with Primary holes on 20-foot centers being split-spaced to Quaternaries in places.

Drilling and Grouting Equipment

Grout holes were completed in a progressive multi-phase process, the first of which involved drilling the overburden material to gain access to the rock foundation. Due to the requirements set in ER 1110-1-1807 for drilling and advancing holes within earth embankments, there were very few acceptable drilling methods that were permissible for drilling dry to the depths between 150 to 225 feet, as required by the contract. Sonic drilling methods were therefore chosen as the best method to advance the holes through the embankment (Figure 11). Drilling work began in April of 2007. Every hole was continuously sampled and logged by a trained geologist. The overburden information recorded on the logs was compiled, and added to the As-Built Drawings to be used for the subsequent work involved in the construction of the proposed cutoff wall.

Once each hole was cased to the top of the rock formation, PVC standpipes were installed and grouted in place to protect the embankment and overburden from the subsequent drilling and grouting operations that would use significant amounts of water and pressure. Standpipes were installed by lowering them into place and tremie grouting the annulus between the standpipe and the outer casing prior to removal of the outer sonic drill casing. The casing grout was then allowed to cure, creating a permanent hydraulic barrier in the annulus between the rock formation and the overlying embankment material.

FIGURE 11: Truck-mounted Sonic Drill (Source: ACT, Ltd)

Rock was drilled with Water-Powered Down-the-Hole Hammers (WDTH) that used water to both actuate the percussion hammer as well as to remove the drill cuttings from the hole. The use of WDTH drills allowed for the fast and accurate drilling to depths of 350 feet (Figure 12). Quality control of drill hole alignment was performed with high resolution borehole imaging equipment also fitted with deviation survey sensors. The high resolution borehole equipment (Optical Televiewer) was also used to create detailed image logs of selected exploratory and production holes. Approximately 33% of the holes drilled (as measured by linear feet) were surveyed and checked for deviation. Approximately 2% of the holes drilled (including verification holes) were recorded with the high-resolution video imaging equipment.

Water pressure (in-situ permeability) tests were generally performed in 12.5 foot stages for every hole with the use of inflatable double packer systems used to isolate and test discrete intervals of rock. Grout injection was generally performed in 25-foot stages using a single inflatable packer and upstage grouting methods.

FIGURE 12. Crawler drill rig with Water-Powered Down-the-Hole (WDTH) Hammer (Source: ACT, Ltd.)

Grout Mix Design

The grout mixes for Phase 3 operations were greatly improved from prior grouting programs exploiting the advent of admixtures and new mixing technology. The grout mixes were balanced and stable, using a combination of water, cement, hydrated bentonite slurry, polymer, and superplasticizer. These balanced stable grouts produce very little bleed water as prescribed in ASTM C-940 and are highly resistant to pressure filtration as prescribed in API RP13B-1. The advantages of these mixes, relating to stability and rheology, are summarized in Weaver and Bruce (2007). Grout mixes were batched in high shear colloidal mixers for thorough and consistent mixing. Quality control testing was performed regularly during the grouting operations to ensure that grout mixes had consistent and compliant properties. The grout mixes were delivered to each hole and injected into the rock formation with use of progressive cavity positive displacement

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(moyno) pumps. These pumps are superior to piston pumps by providing a steady and even pressure within the injection system for smooth and accurate pressure injection at the hole. Injection flow rates and pressures were accurately monitored at the hole by electronic flowmeters and pressure transducers.

Computer Monitoring and Recording of Grouting Operations

For the first time on this project, significant advances in computer technology allowed real-time monitoring and recording of all water pressure testing and grout injection stages. Signals from the electronic flowmeters and pressure transducers were sent wirelessly to a computer system on site that recorded and measured the testing and grouting operations. Each individual pressure test and grout injection stage was displayed in real-time and monitored by a trained operator. Real-time displays consisted of three concurrent time-history plots of pressure, flow rate, and measured real and apparent permeability (in Lugeon units) with time. Real-time displays were used to monitor and control injection pressures within specified limits. All pressure test and grout injection stages were copied to a central project database that was used to store and display the information for daily pay items, summary of project quantities, and analysis of grouting results.

Computerized grouting allows for a reduction in staffing costs because one contractor operator and one government oversight person can monitor grouting at up to 4 holes simultaneously, with the potential for more. This stands in stark contrast to the work done previously when dedicated personnel were required at each grout hole to record pressures and volumes.

Integration of computer aided design and drafting (CADD) software with the real-time monitoring system allowed for the rapid development and update of as-built drawings for analysis of grouting results. The computer monitoring system was used to quickly compile and disseminate the grouting results to all critical personnel and project team members involved with making technical decisions. The results were used to make informed decisions and program modifications including the authorization and location of additional holes for further grout treatment.

Grouting Pressures and Refusal Criteria

An often-cited "rule of thumb" for grouting pressures used in the United States is 0.5 psi per foot of vertical depth in soil, and 1.0 psi/foot of vertical depth in rock. However, it should be noted that this "rule of thumb" was generally applied to the gauge pressure at the collar of the hole and not the effective gauge pressure applied at the stage. The latter reflects influences as the static fluid pressure, local groundwater pressure, and headlosses in the injection system. It should be also noted that "rules of thumb" for applied injection pressures used in other part of the world, such as in Europe, are higher than those of the U.S. Choosing the correct injection pressures is always a challenge with every grouting project and may change depending upon specific site conditions such as formation type and strength. Other factors include proximity to sensitive zones or structures and the level of risk involved with potential damage to subsurface material that may occur with high fluid pressures.

Appropriate pressures may change from project to project, and may even change at different locations within a single project. However, with the help of real-time computer monitoring systems during pressure testing and grouting operations, higher injection pressures can be used with greater confidence and safety. Injection pressures used during Phase 3 operations in fact varied during the course of the work and depended on the location of the stage in regards to both depth and proximity to identified critical areas at the site. Generally, for stages outside critical areas, the injection pressures used for each stage were based upon depth of the stage and the amount of overlying soil and rock above the stage. Maximum injection pressures were calculated as 0.5 psi per foot of vertical depth of soil, 1.5 psi per foot of vertical depth of rock above Elevation 500 feet and 2.0 psi per foot of vertical depth of rock below Elevation 500 feet. Maximum injection pressures were also calculated as effective pressures and then adjusted (reduced) for the corresponding gauge pressure applied via the header system at the top of the hole. For the holes that were grouted in Critical Area 1 (Station 33+60 to Station 37+00), the general rule of 0.5 psi per foot of vertical depth in soil, and 1.0 psi per foot of vertical depth in rock was applied. Critical Area 1 was so named as the section of the embankment foundation that contained the core trench and cave system where weak sensitive soil layers and clay-filled zones were known to exist.

The stage refusal criterion for the majority of the grouting during Phase 3 work was a flow rate of 0.75 gpm for 5 minutes. Some priority holes identified by the project team as requiring more intensive and complete grouting had the refusal criterion lowered to 0.2 gpm for 2 minutes.

Closure Criteria

The information collected and displayed by the computer monitoring system allowed the project team to determine where additional holes were required in order to meet project closure criteria. Closure criteria developed for the Phase 3 grouting program included a combination of maximum grout "take" (injection volume) requirements and in-situ residual permeability requirements. The primary closure criterion for interim risk reduction measures and pre-treatment of the rock foundation was based on a maximum grout take of 200 gallons of stable grout for a single 25-foot stage. Any hole with one or several stages that had injection volumes greater than 200 gallons of grout would require that additional split spaced holes be installed on either side. The goal was to provide a grout curtain in rock with a maximum permeability of 10 Lugeons in the area of the proposed cutoff wall, and a maximum permeability of 3 Lugeons in the rock below the proposed cutoff wall.

The U-Line was grouted to the Tertiary hole series (5 foot centers) along the entire alignment, with split-spaced Quaternary holes (2.5 foot centers) added as needed. The D-Line was grouted to the

Secondary hole series (10 foot centers) with split-spaced Tertiary holes (5 foot centers) added as needed. During the Phase 3 grouting program, which lasted approximately 18 months, 192,612 linear feet of overburden, and 238,625 linear feet of rock were drilled, and a total of 818,317 gallons of stable grout was injected (approximately 106,000 cubic feet of cement solids).

PHASE 3: PROJECT SUMMARY AND RESULTS

The use of advanced equipment and methods was the driving force for the completion of a large amount of work in a short period of time, thus accomplishing the immediate goal of providing the interim risk reduction measure as outlined by the Dam Safety Action Classification system. In addition, critical information concerning the existing subsurface conditions was compiled and used for the subsequent phases of work, including the continuation of foundation grouting operations as well as the construction of the proposed cutoff wall.

Some parts of the original Phase 3 scope were not accomplished. In particular, work in Critical Area 1 was suspended due to concerns regarding the appropriate selection of injection pressures, and the need to determine the appropriate drilling and grouting procedures that would effectively treat the sensitive clay-filled zones as well as address the perceived dam safety risks. Some additional holes, needed for closure along the U-Line and D-Line, were also not added during the Phase 3 grouting program and were deferred for convenience to Phase 4.

PHASE 4: EMBANKMENT CONTACT GROUTING AND THE COMPLETION OF THE DEEP GROUT CURTAIN (2009-2011)

By the fall of 2008, the contract for the second and definitive phase of the Seepage Remediation Program had been awarded to a Joint Venture (JV) of TREVIICOS and Soletanche (TSJV). Integral features of this contract were a systematic investigation and pregrouting (by Limited Mobility Grout) of the embankment/rock contact zone (at the JV's initiative), and the completion and extension of the grout curtain in rock started in Phase 3. These grouting works were conducted by Hayward Baker Inc., and constitute Phase 4 of the grouting history of Wolf Creek Dam. (For information on the construction of the concrete cutoff itself, Santillan et al. [2013] provide an excellent account.)

The Phase 4 grouting in fact included five separate features of work:

(i) A Low Mobility Grout (LMG) double line pre-grouting program performed as an incidental item to the TSJV cutoff wall construction contract (March - May 2009).

(ii) Main embankment dam double line High Mobility Grout (HMG) curtain grouting (May 2009 - November 2011).

(iii) Right rim HMG single line HMG curtain grouting program (May 2009 - March 2010).

(iv) Over-water work, near embankment dam to concrete monolith contact area. This program was modified and became a "fan grouting" program (October 2010 - April 2011).

(v) Critical Area 1 HMG Curtain Grouting Program (October 2010 - November 2011).

These areas are shown in Figure 13.

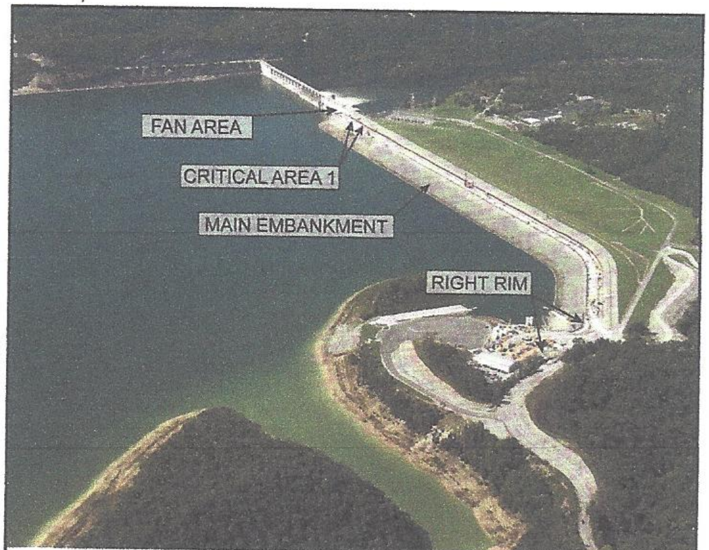

FIGURE 13. Wolf Creek Dam in September 2009. TSJV Plant in foreground. HMG grouting program and barrier wall construction installation in center.

(i) Low Mobility Grouting (LMG) Pre-grouting Program

Prior to the HMG grout curtain work, the TSJV initiated a pre-grouting program for the embankment/foundation rock interface zone, using LMG. The purpose of the pre-grouting program was two-fold. First, the program provided TSJV information related to the interface conditions prior to the cutoff wall construction process, which was to initially use a hydromill and polymer slurry. Second, the program reduced the possibility of major slurry loss, and allowed direct pre-treatment of potentially problematic areas at the embankment/ foundation rock interface.

Using rotary-duplex drilling methods compliant with ER 1110-1-1807, a 5-inch nominal inner diameter casing was advanced (Figure 14) through the embankment and the upper epikarst zone to 2 to 5 feet below the top of competent rock. After ensuring the casing interior was open, it was tremie filled with a nominal 3-inch slump LMG material to displace the drilling fluids. Pressure grouting began, and each location was carefully grouted from the bottom of the hole up in 1-foot intervals until the nominal 15-foot-thick epikarst treatment zone was grouted. Refusal criteria included quantity refusal, pressure refusal, and observations. An injection rate of 2.5 cubic feet per minute (cfm) was used with a pressure refusal criterion of less than 200 psi over line pressure. After grouting the treatment zone, the casing was withdrawn with the LMG installation rig while being topped off with LMG grout. Advanced Data Acquisition (DAQ) was used to monitor and record all aspects of the LMG injection process, an early application of the technology for LMG operations.

The pre-grouting holes were in 2 lines about 7 feet upstream and 7 feet downstream of the future cutoff wall centerline. Approximately 250 locations were grouted, requiring nearly 45,000 lineal feet of LMG-related drilling and the injection of approximately 662 cubic yards of LMG.

FIGURE 14. LMG Installation Rig used to install casing and remove casing during LMG process (Source: Hayward Baker Inc.)

(ii) Main Embankment Curtain Grouting

The curtain grouting scope of work consisted of two sections, including a 2-line grout curtain with lines positioned 12 feet upstream and downstream of the cutoff wall centerline, respectively, and extending the entire length of the embankment section from Station 31+47 to 72+00. This program was designed to align with and complete the grouting of the Interim Risk Reduction Program of Phase 3. The Embankment Section grout holes extended from the top of the working platform (at approximate EL 749.0) to EL 425.0, which was approximately 50 feet below the bottom of the future cutoff wall. Similar to the Phase 3 program, holes were oriented 10° from vertical, angled ahead station (toward the right abutment) on the U-Line and angled back station (toward the left abutment) on the D-Line.

The closure criterion varied depending on the location and depth of the grouting zone, with the criteria below the final cutoff wall elevation tighter. Specified criteria were as follows:

Station 31+47 to 32+31	3 Lugeons or less
Station 32+31 to 60+00	10 Lugeons or less above EL 500 feet
	3 Lugeons or less below EL 500 feet
Station 60+00 to 69+00	10 Lugeons or less above EL 600 feet
	3 Lugeons or less below EL 600 feet
Station 69+00 to 72+00	10 Lugeons or less above EL 660 feet
	3 Lugeons or less below EL 660 feet

(iii) Right Rim Grouting

The rock-grouting program was extended beyond the cutoff wall on the right rim between Stations 72+00 and 83+00. The right rim had minimal overburden, generally less than 40 feet. The scope consisted of a single row of holes installed to a 3 Lugeon acceptance criterion. As anticipated due to the relatively competent rock revealed during the investigation phases of work, a single line of grout holes was planned. Split spacing with 2.5-foot center-to-center hole spacing down to a quaternary pattern was required to achieve the required closure criterion. A total of 516 grout holes were used over the 1,151-foot-long right rim treatment length (Figure 15).

(iv) Over-Water and Fan Grouting Program

The section of grout curtain from Station 31+47 to 34+40 was modified after contract award to accommodate the use of fan drilling, which allowed the grouting work to proceed without the use of a temporary over-water platform section. Fan drilling required complex logistics to coordinate both sonic overburden drilling and rock drilling with water testing and grouting. Close coordination was needed to maintain mandatory dimensional separations between the various operations to ensure dam safety and avoid damaging completed grouting work.

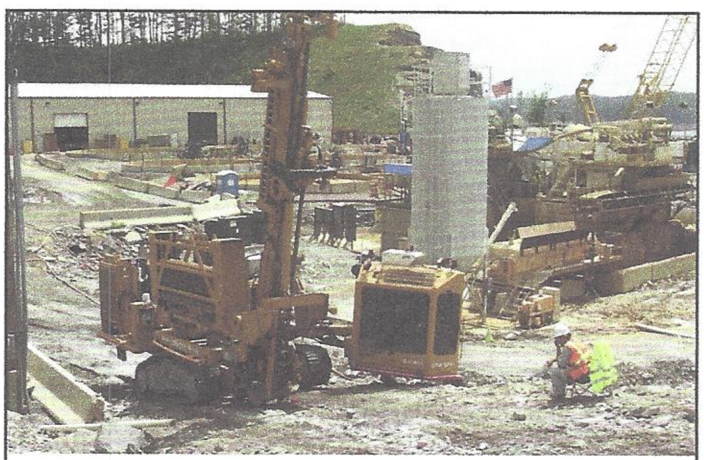

FIGURE 15. Right Rim rock drilling with TSJV cutoff wall slurry and desanding plant in background (Source: Hayward Baker Inc.)

The planned over-water work in this area was located in a critical trout hatchery water intake area. The use of the fan hole alignment as detailed in Figure 16 effectively mitigated environmental and safety risks.

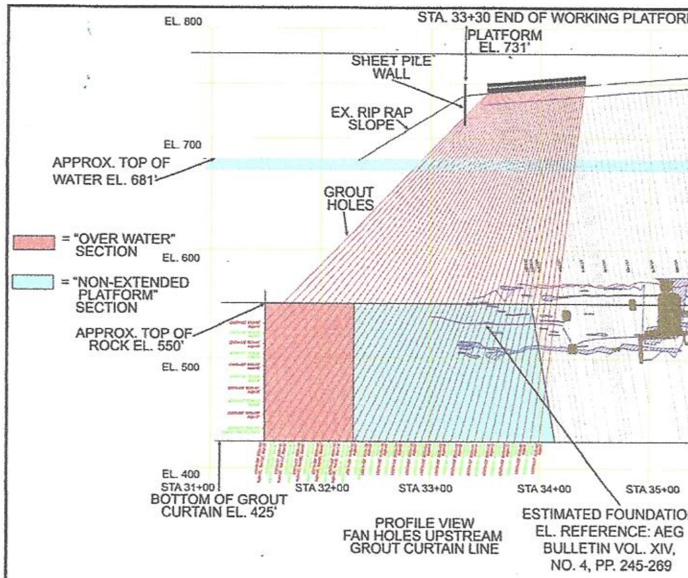

FIGURE 16. Over-water work – Fan treatment upstream curtain line.

Except for holes in Critical Area 1, the grout holes required sonic drilling to advance temporary steel casing through the embankment and underlying alluvium and then 2 feet into the foundation rock. A 4-inch PVC standpipe was then set and grouted while withdrawing the steel casing. The remaining hole length was advanced through the formation rock using WDTH holes of 3 7/8-inch diameter.

In all areas outside of Critical Area 1, a series of balanced, stable grout mixes was injected into the rock formation, involving real time computerized monitoring, and data collection of the entire water pressure testing and grouting process using similar methodologies as the Phase 3 grouting.

(v) Critical Area 1 Curtain Grouting

In Critical Area 1, a different drilling and grouting method was used because of the highly sensitive cave-like conditions and the critical nature of the work. During the Phase 3 Program, Critical Area 1 grouting was suspended due to dam safety concerns, and the USACE deferred the work to Phase 4 to allow more instrumentation to be installed prior to grouting. The complex geology in Critical Area 1 included the compacted clay-filled cave features, alluvium-filled cave features, and the core trench.

Seven different procedural variations were attempted during Critical Area 1 grouting, including work within the loose clay filled zone. The process began as the Alternative Cave Treatment Work including LMG and four HMG variations depending upon conditions encountered and the treatment zone. Some holes were designed to penetrate through the cave features and treat the underlying rock

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only, while other holes specifically targeted the cave features and other possible loose zones that might exist within the embankment above the cave features.

During the initial LMG injection performed for the Alternative Cave Treatment work, transient piezometric spikes occurred from instruments very close to LMG locations. Piezometric head pressure dissipated within 24 hours as anticipated. However, the spikes were not considered a tolerable dam safety risk by the USACE and the LMG process was discontinued. Once this occurred, similar to the suspension of work in Critical Area 1 during Phase 3, work was suspended from February 2010 until September 2010, while alternate means and methods were evaluated.

Following this suspension, the grouting work was resumed in different forms. Through cooperative discussions and field work trials between the USACE, TSJV, and the grouting contractor, a new specification was implemented in October 2010. This new procedure required the use of modified drilling and grouting processes, utilizing vertical holes, and downstage sonic drilling and grouting methods. The requirement for water testing was eliminated to protect the sensitive clay-filled zones from the (faint) possibility of erosion. In addition, grouting pressures were limited to gravity pressure only as in Phase 2, and the standard project grout mixes were utilized. This process allowed for carefully controlled grouting and emphasized the real time review of information as a critical component to maintain dam safety as the highest priority during the grouting process.

PHASE 4: PROJECT SUMMARY AND RESULTS

Using the closure criteria developed for the Phase 4 grouting program, the main work features of the program were completed successfully to allow for safe cutoff wall construction.

The LMG interface investigation and treatment program directly addressed concern over the potential for slurry loss during the installation of the cutoff wall. The LMG program included 45,000 lf of drilling and over 650 cubic yards of LMG injection which was monitored and recorded using DAQ technology.

HMG grouting was performed on the Main Embankment, the Over-Water Section (using the Fan Alternate), and in the Right Rim areas. For all HMG work performed, DAQ and control system monitored and controlled the grouting process utilizing wireless control from technician stations.

The main embankment program was grouted to the specified closure criteria in areas that had not been completed. The U-Line and D-Line were grouted to Quaternary hole series (2.5 foot centers) where needed to locally achieve closure.

For the right rim program, a single grout line was used to extend treatment past the termination of the future cutoff wall to a residual permeability criterion of 2.5 Lugeons, which required treatment to Quaternary hole series (2.5 foot centers) for most of the line.

For the Over-Water Work section which connects the Main Embankment Dam section to the Monolith section, a carefully sequenced double line series of inclined fan holes was used. This method offered significant environmental and safety risk reduction advantages to the project.

For the Critical Area 1 work, several iterative steps and collaborative efforts between the USACE, TSJV, and the grouting contractor were required to develop the grouting work plan. The area was considered extremely sensitive, and the conservative plan involved several different procedures depending upon the conditions encountered in the particular hole. Down stage sonic drilling together with tremie grouting methods were prominently featured in this area. The work in Critical Area 1 resulted in a 7-line curtain. Grout volumes on the final row were barely above theoretical hole volume.

As a total scope of work summary, Phase 4 LMG and HMG grouting together included drilling a total of about 274,000 linear feet of embankment and rock and injecting 375,000 gallons of grout. Collaborative efforts between the parties ensured that dam safety was the highest priority in treating the problematic Critical Area 1. The use of advanced data acquisition and control of the grouting process was a vital aspect of grouting technology on this project.

Barrier wall construction was successful in the critical area without slurry losses or other technical problems. The cutoff wall across the entire embankment was completed in the spring of 2013 (Santillan, 2013).

PHASE 5: CONCRETE GALLERY AND PLAZA GROUTING (2011-2012)

With the Phase 4 grouting reaching completion, and the cutoff wall installation proceeding successfully and satisfactorily, the USACE decided to extend the foundation treatment by grouting under the concrete section (from the preexisting grouting and drainage gallery) and from the "plaza" (a location near the concrete section on the downstream of the dam). These works were conducted by The Judy Company.

(i) Gallery

The gallery tunnel of the concrete gravity structure extended for its full length, roughly 20 feet above the rock interface. The grout holes were drilled in the center of the gallery, between the original Phase 1 grout holes on the upstream edge and the drain holes on the downstream edge. They were drilled as specified by traditional coring techniques to minimize the amount of cuttings waste produced by the drilling, to avoid clogging the gallery sump pumps. Roughly, 30% of holes were surveyed with a down-hole camera for alignment. At the conclusion of gallery grouting, the old gallery drain holes were reamed out. The goal of this gallery program was more to identify any solution features than to reduce permeability; it was expected that the Lugeon values would be below 3.

ISSN 1944-9836 - Association of State Dam Safety Officials

A suite of balanced, stable grout mixes were used, mixed with colloidal mixers. The grout was injected using computer-control and monitoring. All of the grout for this phase was dyed yellow (Brosi and Rathbun, 2013) to differentiate the 2011-2012 grouts from the 1968-1975 grouts encountered in the foundation. Figure 17 shows dyed grout from an Optical Televiewer Log recognized in a Verification Hole.

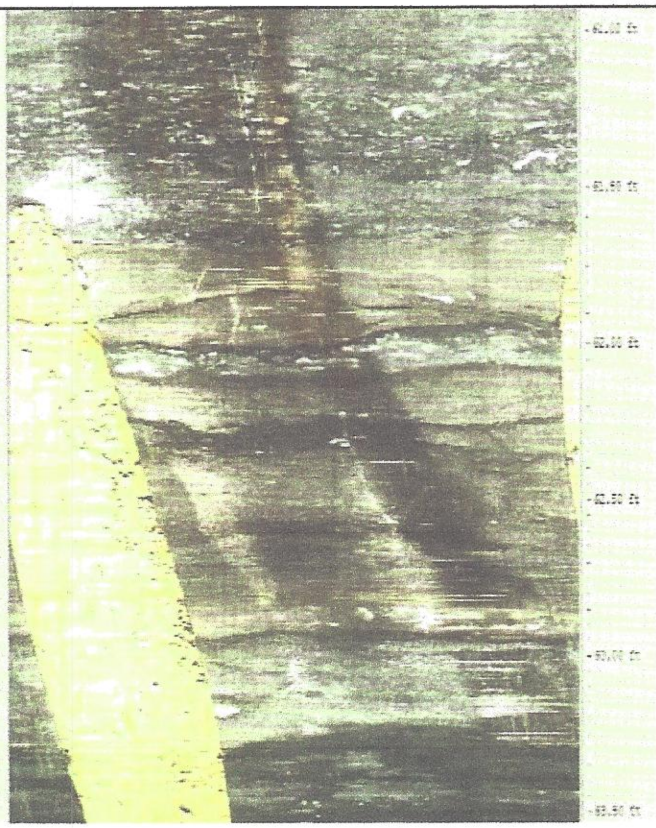

FIGURE 17. Down-hole televiewer image (360-degree view) showing dyed grout in an intersected grout hole (Courtesy of The Judy Company)

This phase also included the first use of an 'instrumented packer' on a USACE grouting project (Paul et al, 2013). This instrument reads the pressure at the injection point, rather than at the surface via a pressure sensor mounted below the single packer.

The 1,700-foot-long gallery can be divided into three sections. The first or right section was the far-right 200 feet that was previously grouted (during Phase 3). The second or middle section, grouted as part of this work (Phase 5), was roughly 1100 feet long. The last or left section, 200 feet long, comprised the leftmost abutment tie-in which was an angled section of mostly stairs. The leftmost section was not drilled or grouted. A total of 40 new holes was drilled and grouted in the gallery, and 17 exploratory holes (previously drilled by others in the rightmost section) were grouted to refusal. The borehole depths averaged 135 feet, to reach a uniform bottom elevation of 425 feet (the same bottom elevation as in Phase 3). The middle section of the gallery had primary hole spacing on 80 feet centers. Every primary and secondary hole was installed. Several tertiary holes were targeted adjacent to the highest Lugeon primary and secondary holes. Finally, verification holes were drilled, often targeted near higher-Lugeon holes.

Only one gallery grout hole had a grout take greater than 10% above theoretical, and it had a hydraulic connection with a nearby drain hole. Every primary hole and every hole with Lugeon values above 3 in the gallery was surveyed by the down-hole camera, looking for any open fractures or solution-enlarged bedding planes. No solution features were found with only enlarged bedding planes less than half an inch in aperture. The gallery hole with the highest measured excess pressure (60 psi) required grouting with polyurethane grout. Primary gallery holes had a mean permeability of 1.5 Lugeons. For all (non-verification) stages in the gallery, the mean permeability was 1.3 Lugeons, and the median was 0. Verification hole stages had a mean permeability 1.0 Lugeons, and a median of 0 Lugeons.

(ii) Plaza

The "plaza" was the term used for the portion of the dam between the powerhouse and the downstream end of the electrical switchyard. This section partly underlies disturbed ground just beyond the toe of the embankment. A first plaza grout line was laid out parallel to the cutoff wall built between 1975-1979 that ran from the powerhouse to the switchyard and along the front (riverside) two-thirds of the switchyard. This second (exploratory) grout line was then drilled upstream (uphill) of the switchyard. Overburden drilling was performed with sonic drills, and a PVC standpipe was installed inside the hole and grouted in place to allow for later rock drilling. Rock was drilled by traditional coring techniques, as required by specification. Approximately 20 percent of the holes were surveyed with a down-hole camera. Grouting was conducted under the same specification in place for the Phase 5 Gallery work.

There were 206 borings in the plaza grout line along a 600 feet length. All primary, secondary, and tertiary holes were drilled to an elevation of 425 feet, so all holes were 150 feet deep. The minimum spacing was 5 feet (tertiary), with quaternary and verification holes located according to grout takes. Exploratory holes were drilled on 80 feet centers, and they were considered primary holes for closure analysis. Primary holes averaged 6.4 Lugeons and had a median of 0.3 Lugeons. Both quaternary and verification holes had a mean Lugeon value of 2.9, and medians of 0 Lugeon. The goal of the plaza grouting was to achieve 3 Lugeons residual permeability.

There were few significant takes in the plaza area. The largest take was 4174 gallons on one 10-foot stage. There were twelve stages that had takes over 1,000 gallons and 35 stages with takes between 100 and 1,000 gallons. For context, there were a total of 1,700 grout stages in the plaza area.

There were some instances of communication between boreholes, a phenomenon which, on account of geology and layout of the site, had become particularly evident in the Phase 3 works. Exploratory holes were drilled on 80-foot centers and all were completed before further drilling. There was no communication between exploratory holes. When primary holes were drilled, on 20-foot centers, many were left open at the same time, allowing communication between boreholes to a maximum of 120 feet. During communication, a mechanical surface packer with a flow valve was used to control flow. During pressure testing, both open and shut permeability values were measured. When grouting, water was allowed to flow until intact grout reached the packer, after which the mechanical packer flow valve was shut. Subsequent to grouting the primary holes, communication was limited to spans of 30 feet, and decreased with each series.

There were 12 holes along a 260-foot-long exploratory grout line just upstream/uphill of the electrical switchyard. The majority of holes encountered solution features, and so Lugeon values and grout takes were significant. This exploratory line did not attempt to reach closure. The mean grout take was 60 gallons, or ten times the theoretical volume of each 10-foot stage.

OBSERVATIONS AND CONCLUSIONS

As a result of flaws in the original 1930's design assumptions and construction approaches, and the inherently poor, erodible nature of its karstic foundations, Wolf Creek Dam has seen five separate phases of foundation grouting, at least two of which can be regarded as "Emergency" in nature, or, in current parlance, as "Interim Risk Reduction Measures." Each in its own way, and within its own mission, was successful, although it can be emphasized that the latter part of Phase 2 and at least part of Phases 3 and 4 were specifically conducted to facilitate the safe installation of "permanent" concrete cutoffs. This is the principle of "Composite Wall" design and construction (Bruce et al., 2012).

It is fascinating to see the evolution of rock grouting practice in the U.S. — and in particular on USACE projects — effected in the different aspects of design and construction over the 70 years of drilling and grouting at Wolf Creek Dam (Table 1).

TABLE 1. COMPARISON OF DIFFERENT GROUTING ERAS

<table>
<tr><th>PHASE</th><th>ERA</th><th>ROCK DRILLING METHOD</th><th>GROUT MIX</th><th>GROUT MIXING</th><th>CONTROL AND ANALYSIS</th><th>PRESSURES USED</th><th>PRINCIPAL CLOSURE CRITERION</th></tr>
<tr><td rowspan="2">1</td><td>1942-1943 Core Trench</td><td rowspan="2">Coring</td><td rowspan="4">Neat, unstable</td><td rowspan="4">Paddle</td><td rowspan="4">Hand</td><td rowspan="2">Up to 1 psi/ft</td><td rowspan="4">Grout Take</td></tr>
<tr><td>1948-1949 Gallery</td></tr>
<tr><td rowspan="2">2</td><td>1968-1971</td><td>Rock Roller-Bit & Coring</td><td rowspan="2">Gravity</td></tr>
<tr><td>1973-1975</td><td>Rock Roller-Bit</td></tr>
<tr><td>3</td><td>2007-2008</td><td rowspan="2">Rotary Percussion (Water DTH)</td><td rowspan="2">Balanced-Stable HMG</td><td rowspan="3">Colloidal</td><td rowspan="3">Computer</td><td>Up to 2 psi/ft</td><td rowspan="3">Residual Permeability</td></tr>
<tr><td>4</td><td>2009-2011</td><td>Up to 1.5 psi/ft but gravity head in a particularly sensitive zone</td></tr>
<tr><td>5</td><td>2011-2012 Gallery/Plaza</td><td>Coring</td><td>Balanced-Stable HMG with Dye</td><td>Up to 1 psi/ft</td></tr>
</table>

ISSN 1944-9836 - Association of State Dam Safety Officials

For example, the adoption of colloidally-mixed, stable mixes, providing superior Pressure Filtration Coefficient resistance (Weaver and Bruce, 2007) as High Mobility Grouts, reflects unquestionable U.S. and international practice, as does reliance on permeability testing (and Optical Televiewing) to engineer and verify the target residual permeability. Computers that monitor, control and analyze water test and grout injection operations are a creation of the 1980's, but the unremitting reliance placed upon them in Phases 3 to 5 is also a common thread of the Wolf Creek works. Intriguing, however, is the philosophy in two particular areas—drilling method and grouting pressures—since a clear progression is not logically evident, and indeed a certain retrogression can be inferred.

Prior to Phase 3, the favored rock drilling method was coring. This reflects practice from the earliest days of dam foundation grouting, as dictated by the equipment available to the construction market (controlled by the diamond trading houses) and thereafter sustained by the recycling of traditional, prescriptive specifications. It may also be added that, in earlier times, percussion drilling was synonymous with the use of air (as opposed to water) flush, and it is to the credit of earlier grouters that they clearly recognized the benefits of water flush for rock fissure grouting and the disadvantages of air flush. (This is a subject which has been long debated, but one disguised, disingenuously, as one being only between rotary and percussion drilling methods, as opposed to one between water and air flush.)

It is therefore logical and laudable that, by Phase 3, the rock drilling method of choice was the Water-Powered Down-the-Hole Hammer (Bruce et al., 2013), offering the best of all rock drilling worlds with respect to productivity, straightness, cleanliness, and environmental impact. For Phase 5, coring was prescribed, superficially a step backwards. However, coring was prescribed for environmental reasons, presumably relating to the practicalities and issues inherent in operations in a confined space galley. These restrictions would not have applied technically or environmentally to the Plaza drilling but, for work of somewhat limited scope, it is understandable that the same drilling equipment would be mobilized in each area. The conclusion is, therefore, that the best rock drilling method may not always be the most appropriate choice for the project.

The issue of safe and allowable maximum grouting pressures is arguably the most contentious of all contemporary drilling and grouting debates (Schaefer et al., 2011, and Bruce, 2011). It is clear and correct that no party on any project would sanction parameters potentially liable to cause any distress (let alone catastrophic distress) to a dam during its remediation. Prior to the 21st century, maximum grouting pressures were typically limited, in U.S. practice, by historical, arithmetic paradigm, to 1 psi per foot. European practice was limited to 1 bar per meter, i.e., about 4 times higher (Weaver and Bruce, 2007). The paradigm is reflected in the Phase 1 parameters. During Phase 2, the perceived vulnerability of the dam led the USACE to stipulate only gravity refusal pressures, probably since their contemporary ability to control the rheological properties of the grouts was very limited, given the equipment, methods and materials then in commonly accepted use. The criticality of the dam's stability at that point was a very persuasive argument. Contemporary practice is then revealed in Phases 3 and 4, by which time there was a considerable, progressive consensus of thought that higher grouting pressures could be safely used, provided they were accurately and quickly monitored in real time (by computers), and appropriate cognizance of targeted dam and foundation instrumentation was intrinsically, constantly, granted. However, even in Phase 4, the perceived sensitivity when grouting a critical area of the curtain resulted in the decision to reduce the gauge grouting pressure to zero (the gravity grouting approach of the 1960's). During Phase 5, which was conducted in an area judged non-critical, a certain amount of pump pressure was permitted, albeit to the same levels employed in the 1940's during Phase 1.

The twists and turns in industry opinions are lucidly demonstrated by this discussion of maximum, "safe," grouting pressures. The fact remains that grouting has secured the safety of Wolf Creek Dam on at least two occasions, and has facilitated the construction of what is hoped to be the final solution, involving a concrete cutoff wall. None of the five successive grouting interventions has proved detrimental to the dam, while each has made positive contributions. This is the ultimate compliment to all who have been involved in this project over the decades.

REFERENCES

American Petroleum Institute. (1990). "Recommended Practice Standard Procedure for Field Testing Water-Based Drilling Fluids, Density by Mud Balance, Marsh Funnel and Pressure Filtration Test," *Standard Specifications, RP 13B-1.*

ASTM (2010). "Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory," *Standard C-940.*

Brosi, B.E., and Rathbun, C., (2013). "Use of Dyed Cement Grouts in Foundation Grouting," *Geotechnical News*, March 2013, pp. 9-13.

Bruce, D.A. (2011). "Rock Grouting for Dams and the Need to Fight Regressive Thinking," *USSD Annual Conference, April 12-13, San Diego, CA*, 16 pp. and in *Grout Line, Geotechnical News*, Volume 29, Number 2, June, p. 36-43.

Bruce, D.A. (2012). *Specialty Construction Techniques for Dam and Levee Remediation*, Spon Press, an imprint of Taylor and Francis, 304 pp.

Bruce, D.A, T.L. Dreese, M.C. Harris and D.M. Heenan. (2012). "Composite Cutoff Walls for Dams: Theory and Practice," *Deep Foundation Institute, 4th International Conference on Grouting and Deep Mixing, New Orleans, LA, February 15-18.*

Bruce, D.A., R. Lyon, S. Swartling (2013). "An Introduction to Water-Powered Down-the-Hole Drilling in Specialty Geotechnical Construction," *Deep Foundations Institute 38th Annual Conference, Phoenix, AZ, September 25-28,* 7 pp.

Fetzer, C.A. (1979). "Wolf Creek Dam - Remedial Work," *ICOLD, Q. 49 - R. 5*, in Volume 11, pp. 57-82.

Kellberg, J., and Simmons, M. (1977). "Geology of the Cumberland River Basin and the Wolf Creek Damsite, Kentucky," *AEG Bulletin*, Volume XIV, Number 4.

Mackey, S., and Haskins, T. (2012). "The Initial Grouting in the Embankment Foundation at Wolf Creek Dam near Jamestown, Kentucky," *Grout 2012*, 11 p.

Paul, D., Brosi, B., Schaefer, J. and Carr, P. (2013). "Development and Field Testing of a Pressure-Sensing Grout Packer," *GeoStrata*, ASCE, February, pp. 40-44.

Santillan, F. (2013). "Wolf Creek Dam Foundation Remediation Project," *Deep Foundations Institute 38th Annual Conference, Phoenix, AZ, September 25-28.*

Schaefer, J., Paul, D., and Boyer, D. (2011). "Safe Grouting Pressures for Dam Remediation," *USSD 2011*, CD-ROM.

Simmons, M.D. (1982). "Remedial Treatment Exploration, Wolf Creek Dam, KY," *Journal of the Geotechnical Division*, American Society of Civil Engineers, pp. 966-981.

Simmons, M.D. (2012). "Remedial Treatment Exploration, Wolf Creek Dam (REPRINT)," *Grout 2012*, American Society of Civil Engineers, Volume 2, pp. 1187-1203.

U.S. Army Corps of Engineers. (2011). "Safety of Dams – Policy and Procedures," *Engineer Regulation 1110-2-1156*, October.

U.S. Army Corps of Engineers. (2006). "Engineering and Design Procedures for Drilling in Earth Embankments," *Engineer Regulation No. 1110-1-1807*, March 1.

Weaver, K.D. and D.A. Bruce (2007). *Dam Foundation Grouting, Revised and Expanded Edition*, American Society of Civil Engineers, ASCE Press, New York, 504 p.

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