EVOLUTION OF GROUTING IN KARST AT LOGAN MARTIN DAM, ALABAMA

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INTRODUCTION

Logan Martin Dam is a hydroelectric facility owned by Alabama Power Company/Southern Companies (APC/SC) and located on the Lower Coosa River in Vincent, Alabama. Construction of the dam was started on July 18, 1960 and it was placed in service on August 19, 1964. The dam itself consists of two earthen embankment portions totaling 5,520 ft (1,682 m) in length, referred to as the east and west embankments, and a central concrete powerhouse section 613 ft (187 m) long. The west embankment is 870 ft (265 m) long and the east embankment is 4,650 ft (1,417 m) long. Figure 1 presents an aerial view of the dam and powerhouse. Maximum height of the dam is approximately 100 feet above the riverbed. Seepage through the karstic limestone foundation began as soon as the reservoir started being filled in 1964. The seepage produced numerous springs and boils in the river channel and along the downstream riverbanks, and eventually at the toe of the east embankment itself. After a sinkhole developed on the downstream face of the east embankment on April 9, 1968, the first of the remedial grouting programs at Logan Martin began.

GEOLOGICAL BACKGROUND

Logan Martin Dam is underlain by Paleozoic-age sedimentary carbonates. As documented by Williams and Robinson (1997) and Redwine (1999), the lithologies comprising the foundation are primarily dolomite, chert and breccias with lesser amounts of sandstone and limestone. These are all part of the Knox Group. The rocks have undergone extensive faulting and folding which have facilitated the development of solutioning and other karstic features (Redwine, 1999). Solution cavity development is closely associated with and often mimics the geometry of structural features such as faults and zones of fracture concentration. Thrust faults and normal faults produce different depths and geometries of solution cavity development. It has been proposed by Redwine (2014) that, regionally, extensional forces associated with rifting and opening of the Atlantic Ocean during the Mesozoic era resulted in the deep-seated secondary porosity and permeability in the lower Knox Group that has in recent times been enhanced by the dissolution of the carbonate rocks. This has resulted in the development of multiple seepage paths under the dam along stratigraphic zones, joints, fold axes, and faults. These conditions are the contributing factors to the seepage and subsequent grouting activities at Logan Martin Dam.

The karstic foundation rock at Logan Martin Dam is highly fractured and has been subjected to multiple periods of faulting and folding. In the tailrace of the dam a tight, doubly plunging syncline can be observed at low outflow conditions. The foundation beneath Logan Martin dam and reservoir is part of a large fault block that overlies a prominent thrust fault. Three thrust faults and a series of near vertical normal faults have been identified at the dam site. The most prominent of these faults has been termed locally as the "target zone fault" which strikes approximately N72E and crosses the east embankment of the dam. The target zone fault crosses the east embankment about 420 ft (128 m) east of the spillway at Station 61+80 and intersects the west river bank approximately 984 ft (300 m) downstream of the dam (Robinson and Williams, 1997). As indicated in Figure 2, locally the faults often serve as structural boundaries that affect the flow of water under the dam. Overprinted on the rock structure is a series of joints. A N85E-75W vertical joint set is linked with the formation of conduits and this joint orientation closely parallels vertical faults at the site. The presence of well-developed solution weathering along these joints presents clear evidence that they are preferred hydraulic flow paths for water, especially where they intersect highly permeable limestone layers (Robinson and Williams, 1997).

The most significant zone of permeability beneath the dam occurs through two principal horizons; one is shallow just below the base of the dam, and the other very deep. The upper 100 ft (30m) of bedrock has experienced the most extensive karstic weathering and when it was first explored by APC/SC during construction of a deep grout curtain, 22 percent of the dolomitic formation was recorded as cavity (Williams and Robinson, 1997). At depths greater than 360 ft (110 m) beneath the dam, a 115 ft (35 m) thick interval was found to contain several limestone beds that contain caverns and convey considerable seepage beneath the dam. The upper and lower zones are interconnected vertically by fractures (faults and joints) that serve to enhance vertical permeability and associated underseepage. The highly anisotropic conditions within the karst foundation at Logan Martin Dam result in enhanced permeability in both horizontal and vertical directions and to depths of more than 430 ft (131 m) below the surface as reported by Williams and Robinson (1997).

DAM CONSTRUCTION

Construction at the site began in 1960. The embankments were constructed as homogeneous earth fills consisting of a cherty clay residuum with a maximum height of 100 ft (30 m). The first stage of construction consisted of installing coffer cells for the west embankment closure for the powerhouse and spillway excavations. As the excavation proceeded, large cavities were exposed in the underlying bedrock (Figure 3). These cavities were cleaned and

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treated with dental concrete. The west embankment construction was completed in 1963. The coffer cells were removed and water diverted through the spillway at this point. The second stage of construction proceeded on the east embankment. Two dumped rock cofferdams were constructed that eventually became the upper and downstream toes of the dam through the original river section. A clay blanket was placed along both the upstream and downstream slopes. Similar to the west embankment, once the foundation bedrock on the east side was exposed, larger and more numerous cavities than expected were encountered. Due to the size of the embankment as well as the number of cavities present, it was decided that a reinforced concrete slab would be constructed beneath the upstream slope of the east embankment through the original river section. The slab was approximately 89 feet (27 m) wide and ranged in thickness from 3 to 7 ft (1 to 2 m). Prior to the slab construction, any cavities exposed were cleaned by hand and filled with crushed rock. A 4 ft (1.2 m) thick blanket drain was installed beneath the downstream portion of the dam.

CHIMNEY SINKHOLE

Immediately upon initial filling of the reservoir in 1964, mud flows were noticed downstream of the earthen portion of the eastern embankment. Boils also began forming in the tailrace area. In June of 1966, seepage was noted at the toe of the east embankment. Weir 27 was built to monitor flow at that point. On February 23, 1968 the weir began discharging mud. For a period of 40 days these mud flows were observed. On April 9, 1968 a sinkhole approximately 20 ft (6 m) in diameter and 16 ft (5 m) deep opened just downstream of the east embankment crest and was referred to as the "chimney sink." Immediate action by site personnel to shut down the roadway across the dam with backfilling (Figure 4). This sinkhole was a clear manifestation of the karstic subsurface conditions at the site and had the clear potential to fundamentally threaten dam safety. Numerous holes were drilled surrounding the sink to determine its extent. In addition, piezometers were installed across the site to further monitor water levels and temperatures. The sinkhole event also highlighted the need for further grouting programs.

OVERVIEW OF SUCCESSIVE PHASES OF GROUTING

Including the original construction grouting, there have been eight successive phases of treatment, two of which are ongoing. The following review focuses on the location, scope and purpose of each phase, and their respective means, methods and materials used. As is the case for all such projects, the documentation is voluminous and includes the reports of successive boards of consultants who, during the 45 years under review, have consistently participated in interactive and constructive relationships with site and main office personnel from the Alabama Power Company / Southern Companies (APC/SC), and the Federal Energy Regulatory Commission (FERC).

Phase 1: Original Construction Grouting (1960 and 1964)

These activities are summarized in Table 1. For reference, dam station 45+00 is on the eastern (left) embankment and increases to the west (right) embankment.

The initial foundation treatment consumed 124 yd^3 (95 m^3) of grout, mainly of low cement content and relatively high proportions of fillers such as flyash, rock dust and fine river sand. Given the practices of the day (rotary drilling, vertical holes, and minimal pressures), the inherent instability of the grouts, and the severe nature of the karst, closure requirements (based on grout takes) saw holes installed as close as 1.3 ft (0.4 m). The consolidation grouting was performed after construction of the earth berm and the concrete slab. Contemporary reports indicate that the work was considered to have been very successful, although in hindsight it would seem that the severity and depth of the karst were not at the time fully appreciated.

Phase 2: Emergency Chimney Sink Grouting (1968)

Immediately following development of the chimney sink in 1968, a remedial grouting program was initiated to try to reduce leakage in the upper portion of the foundation and through the upstream cofferdam. This curtain was installed 7 ft (2 m) downstream of the original grout curtain from Station 60+93 to 62+43, as it took less effort to create a work berm from which to grout. This was a 151 ft (46 m) long section and only 50 ft (15 m) into rock.

The drilling for the emergency grout holes revealed 49 percent of the materials installed in the original grout curtain just 4 years earlier had been removed by erosion. The removal percentage was concluded from the presence of voids found in subsequent drill holes in the area that had previously been grouted. Much of the removed material was found immediately beneath the concrete slab indicating the concrete slab may have prevented a total breach of the dam. This work, and especially grouting around station 61+70, was instrumental in reducing the flow in Weir 27 from approximately 2,700 gpm (6 cfs) to approximately 33 gpm (0.07 cfs).

Phase 3: Grouting from 1972-1990

In this phase, several different grouting programs were performed. Each program built upon the previous program to improve techniques or approaches. Due to the many sequential grouting programs that were planned, it was decided that APC/SC continue to perform the future grouting programs with their own equipment and personnel. This would allow them to make rapid changes to achieve the required results in extremely variable conditions.

In late 1971, the company decided a more robust grouting program was needed to ensure the stability of the earth embankments. It was believed that reinforcement of the upper 100 ft (30 m) of rock along some of the locations of the original grout curtain would control shallow leakage that could endanger embankment stability. As each program was completed and the anticipated results were not fully achieved, the curtains were extended or deepened (Tables 2 and 3).

The initial test program was installed from the original river bank west to the end of the concrete pad at station 61+38. Another line of grouting was performed from station 45+00 to 48+00 (see Table 2). Three exploratory holes were drilled from stations 48+00 to 51+00. This was thought to be a deep rock zone and rock was not encountered to a depth of 203 ft (62 m). As the intent of the program was only to treat the rock to 173 ft (53 m), this area was abandoned. The grout curtain deepening was performed upon the recommendation of the board of consultants through the area that had been treated following the 1968 chimney sink. The subsequent programs were completed extending the depth of the holes to approximately 173 ft (53 m). During this period of remedial work, 79,700 ft (24,293 m) of grout holes had been drilled and 7,194 yd^3 (5,500 cubic m^3) of grout had been injected into 456 holes.

Depressions were found on the upstream portion of the east embankment opposite the location of the downstream chimney sink. These depressions were filled shortly after being detected. Drilling began as soon as a ramp could be constructed on the upstream slope. The third hole drilled intercepted the flow to Weir 27 downstream. These initial programs, while effective, needed to be lengthened and deepened to improve overall stability of this portion of the embankment. This grouting program and subsequent grout lines were drilled across similar stations (Table 3). To prevent confusion with similar stationing, the multiple rows were labeled A, B, C, D, E, F, G, and H lines.

After completion of these grout curtains, dam stability was judged to have been greatly improved; however, significant seepage was still occurring beneath the dam and grout curtains. During 1990, a detailed study was performed to determine the most feasible method to cut off seepage beneath the dam. Alternatives that were investigated included blanketing the reservoir bottom, constructing a cement bentonite cutoff wall, and grouting. Of the proposed alternatives, grouting was chosen due to prior experience at the site and the ability to perform the work with internal company manpower and equipment.

The deep grouting program began in 1991 and continues in some form today (Phase 7). Deep grouting was performed from Station 54+00 to 69+00 and extended 400 ft (122 m) into rock. In 1991, a test section from Station 59+25 to 62+05 was designed to optimize drilling and grouting methods. Due to the sensitivity of the dam, drilling and grouting pressures were limited through the upper 88 ft (27 m) of rock. This program successfully lowered the piezometric levels and further reduced flow through Weir 27.

Phase 4: Deep Grouting Program (1991-2006)

After grouting the test section, the grout line was extended to the east to encounter the Weir 15 section. This operation was performed from 1996 to 1999 from station 52+15 to 59+85. This section was designed to reduce seepage in and around Weir 15. While drilling core hole 53+12 between El. 90 and 70, there was a collapse of the

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water supply hose down the hole, suggesting a vacuum down hole indicative of a true void.

Much of the success of this line of grouting was linked to activities in Hole EP 56+45, which has been one of the most interesting and challenging holes on the project. Grouting on this hole alone took place from 1995 to 1997. Two distinct cavities were encountered with large flows below elevation 124 ft (38 m). A special header was constructed to allow the different materials and quantities to be injected into the hole. For the upper cavity grouting, medium mobility grout mixes containing sand and gravel, sodium silicate and hydrophilic chemical grout were unsuccessful. Burlap strips and polypropylene sacks were added to the grout (717 yd^3 [548 m^3] of cementitious materials and 770 tons [698 tonnes] of sand and gravel were injected). For the lower cavity grouting, 2 inch (5 cm) rounded gravel sluiced with grout was injected for 2.5 months (1.260 m^3 of cementitious material, 1,017 tons [922,607 kg] of sand and gravel, and 13 tons [12 tonnes] of lightweight aggregate). A total of 1866 yd^3 (1,427 m^3) of cement, 498 yd^3 (381 m^3) of C-ash, 921 tons (835,517 kg) of sand and gravel, 587 yd^3 (449 m^3) of rounded gravel, and 13 tons (12 tonnes) of lightweight aggregate were injected into this hole. Additional grout holes 4 ft (1.5 m) away did not show the same geologic features or any injected materials from EP56+45. This program was deemed successful as it reduced seepage through Weir 15 by over 1,300 gpm (2.9 cfs) and also reduced the rate of settlement of the crest.

The purpose of the west non-overflow concrete section grouting was to reduce leakage that might possibly be traveling along the west tailrace bank. This grouting took place from 2000 to 2002 from Station 72+52 to 73+60. A special steel platform was constructed to prevent obstructing the roadway across the dam. Grout takes were generally less than 5 bags per foot down to 0.1 bags per foot for tertiary holes. No flow reduction or lowering of the piezometric levels was achieved using a neat cement grout. Since the non-overflow section had only minor successes, the curtain was extended further to the west. This program took place in 2002-2003, from Station 73+60 to 78+10. It was hoped that this curtain would further reduce seepage and lower piezometer levels along the west embankment. Some piezometers were impacted during grouting around 74+00 and between stations 75+50 and 76+50, but the piezometer elevations generally increased.

In 1997, the F line was completed to treat and prevent any further erosion of infilling that might have occurred beneath the concrete slab. This line was drilled 39 ft (12 m) downstream of the E line from Stations 61+20 to 65+20. This program had relatively high grout takes in the primary holes, especially around the old chimney sink area. Primary, secondary and tertiary holes were installed through this section.

In the time period from 1998 to 2002, the G line was installed 10 ft (3 m) upstream of the E line to try to achieve the closure criterion of 1 bag per foot. It was installed to treat the source of leakage causing temperature anomalies around the old chimney sink, a possible shallow source of seepage. In addition, previous grouting in this area had greatly reduced leakage to Weir 15. Following completion of the G line, there had been slight reductions in piezometer levels and flow through Weir 27. However, the closure criterion of 1 bag per foot was not achieved through all sections even with the quaternary holes. Therefore, the H line was installed through the area where there were higher grout takes than the prescribed closure criterion of the G line from 2002 to 2006. This line with the installation of tertiary holes was able to meet the closure criterion. Grouting in Hole HP 61+40 lowered downstream piezometers up to 3 ft (0.9 m), having more effect on piezometers than any other hole along the line.

The pervious alluvium located in a zone of the east embankment directly over the bedrock was also grouted. After a test program in 2002, microfine cement was chosen to grout this zone to allow better penetration. The alluvial grouting program was controlled by both before and after permeability tests. Reductions in permeability of 60 to 100 percent were recorded.

Phase 5: Compaction Grouting Program (2005-2008)

Several of the· previous grouting investigations had found zones of loose, unconsolidated soils in the overburden. It was decided that these areas would require treatment through compaction grouting (Table 4).

Due to the angled walls at the east end of the concrete spillway, compaction of the soil overburden was inadequate. In response to the Part 12 Potential Failure Mode Analysis (PFMA) team's findings, compaction grouting was required of this portion of the embankment. Five borings were drilled and a total 32 yd^3 (25 m^3) of low slump grout were injected. Standard penetration tests were performed to verify consolidation of the soils. Most standard penetration test "N" values were increased by as much as 30 blows.

During the installation of the original grout curtain, large amounts of grout were injected into the loose soils above the rock as well as into the rock area known as the "Deep Rock Zone." The intent of the compaction grouting program for this area was to see if the overburden or the top 30 ft (9 meters) of rock would accept the 6.5 yd^3 (5 m^3) slump grout injected at 500 psi (3.4 MPa) pressure. Five borings drilled for this only took 24 yd^3 (18.3 m^3) of grout. Therefore, this area was considered stable.

Phase 6: West Embankment Grouting (2008-2010)

In 2002, a group of piezometers on the west side of Logan Martin Dam showed a slight and varied increase in water elevation. The increase, so slight as to pose no concern to embankment stability, did not follow any known pattern of fluctuation in the project piezometers. The temperature surveys that followed indicated that the temperature anomaly that had existed in certain deep into-rock piezometers at the toe of the embankment had also changed. The elevations of these instruments remained relatively stable over the next few years, with a slight decreasing trend. The temperature anomaly also remained stable.

In 2007, investigation into the depth of the temperature anomaly led to the installation of exploratory holes, new piezometers and the refurbishment of existing instruments. Seven continuous monitoring data loggers ("trolls") were installed in the area of interest. Five trolls read water level and temperature and two read water level, temperature, and pH/conductivity. There were also eight continuous water level recorders installed in the area. Salt tracer and dye tests were conducted with mixed results. There was some indication of a shallow flow connection running northeast to southwest through the piezometers that experienced an increase in elevation in 2002. An overburden to rock interface and shallow rock grouting program was initiated.

There were several significant grout takes in 2008. Not all of the larger takes affected instrumentation, but it was significant that the area of takes that did affect instruments were generally northeast of the piezometers affected in 2002. More notably, the effects followed the same northeast to southwest trend, through the same instruments with the same relative water elevation changes as seen in 2002. The elevations were increasing and there were pH hits in the trolls at the west embankment toe, indicating that the grout was travelling downstream and cutting off a portion of the drainage through the area, but the flow continued to maintain the temperature anomaly.

Grout takes from 2008 through the second quarter of 2009 followed the same patterns, with the same level of effect on the piezometers. In the third quarter of 2009 the grout takes increased the piezometer water levels in the core area significantly but did not affect those instruments on the periphery of the area as in the past. More significantly, the temperature in those core area piezometers dropped between five and ten degrees and the water elevations began to fall slowly.

The fourth quarter of 2009 and the first two quarters of 2010 saw slow but steady decreases in the water elevation throughout the area. The winter temperature study indicated further reduction of the temperature anomaly in the core of the area and a return to ground water temperatures for five piezometers on the periphery. The pattern of water level decrease and positive temperature change continued to increase confidence that the smaller fissures were being grouted from the downstream area toward the feature where the flow originated. Five new piezometers were installed at the toe of the west embankment in the area of interest. Three were drilled into rock for confirmation that the existing piezometers were functioning properly and for additional future information. Two piezometers were drilled to a depth of approximately 20 ft (6 m) to discern whether the pressures were in the overburden at the toe or only being transmitted up from the deeper rock. The latter was the true condition and the two shallow piezometers remain dry.

In the third quarter of 2010 an angle hole through the initial area of suspected flow had a heavy grout take near the top of rock. The elevations in all of the previously affected piezometers dropped significantly and immediately. Over the following week the elevations continued to slowly drop to levels not seen since before 2002 (Figure 5). The troll data indicated that the temperatures were affected by this grouting insofar as the rate of temperature change in the piezometers was subdued as compared to the change in reservoir temperature for the time of year.

The February 2011 temperature survey indicated that every piezometer in the area of the west embankment was at or slightly above groundwater temperature. The higher temperatures were noted in the core area piezometers which could have been exposed to the most grout. It was proposed that the increase in temperature was due to the heat of hydration from the grout.

The water elevations have remained relatively stable throughout the area of the west embankment. The piezometers with slightly higher temperatures showed a steady decline to ground water temperature throughout the spring of 2011. The temperatures have remained stable since that time.

There was a very slight, but notable increase in the vertical deformation on the crest of the west embankment in the area of the drilling and grouting. There were no adverse trends in the horizontal deformation. The vertical deformation rates returned to lower levels after cessation of drilling and grouting. Conditions have remained stable through the present time; however, continued close surveillance is being maintained by project staff.

Phase 7: Deep Curtain Regrouting (2011-Present)

This phase of the Logan Martin Dam grouting project is the completion of the Q-holes between stations 52+00 and 60+05 designated as the Weir 15 section. There will be 74 to 80 grout holes for this phase, and each hole will extend 400 ft (122 m) into rock. The overall goal is to reduce or stabilize the flow through Weir 15, eliminate any temperature anomalies (hot spots), reduce piezometric pressure, and solidify the upper fifty feet of rock to mitigate ongoing settlement along this section of the dam.

All grout holes are drilled with a track-mounted Cubex QXW1710 drill. The drilling is accomplished by the use of a Wassara water-powered down-the-hammer (WDTH) as described by Bruce et al. (2013). The advantage of the water hammer system is a cleaner operation, less harm to the foundation and minimal hole deviation. Each stage is a maximum of 30 ft (9 meters). A virtual log of each grout hole is developed by the use of an optical televiewer (OTV). In addition to the televiewer, a caliper tool is used to log the configuration of the drill hole in order to select the best location to seat the packer. Without the caliper data, the risk exists of inflating the packer in a large open fracture or void, thus bursting the membrane.

A water pressure test is performed prior to starting the grouting operation of each stage. The purpose of the test is to assist the grouting geologist in planning the magnitude of the grout take. The test is typically the standard five minute test using the maximum allowable grouting pressure for the stage which is depth dependent. A Lugeon value is calculated for each stage and used as an evaluation tool.

The grout plant consists of two different mixers and delivery systems. For the high mobility grouts (HMG) a Colcrete mixer and agitator have been automated to deliver the various stable grout mixes. Rather large voids/cavities can be encountered between depths of 272 ft (83 m) and 413 ft (126 m). These voids can require grout with bulk fillers such as sand and gravel. A Hany grout plant is used to deliver these materials within a medium mobility grout mix (MMG).

As for the grout mixes, the Logan Martin Dam grouting project has been and continues to be a laboratory for developing stable grout mixes that are effective in grouting a karst foundation. As stated above, the original grouting was performed with very fluid grouts containing bulk fillers such as rock dust and flyash to reduce the cost per batch. Based upon today's knowledge of grouts, these grouts were unstable. During all post-construction drilling at this site, very little original grout has been found in the core samples. The grout used now contains cement, water, bentonite, and glenium (super plasticizer). All mixes satisfy the required values for the pressure filtration coefficient, Marsh flow, bleed, and shear resistance of a stable mix.

An in-line data logger and computer are utilized to record flow, measure the grout pressure, and compute the "apparent" Lugeon value in real time. These parameters are used to determine proper grout refusal for each stage.

Not only have the grouting techniques changed over time, the instrumentation and monitoring of the dam during the grouting process has also been enhanced. In addition, high mobility, stable grouts and higher grouting pressures have been used where they could be employed safely. A telemetry system utilizing instruments that measure pressure, pH and conductivity has been added to the project. The system allows 24-hour monitoring capabilities of key piezometers and weirs. The telemetry system is solar powered. Alarm levels have been established and a message is sent via cell phones and e-mail to the project team members if these levels have been exceeded.

To date, this phase of the project is 44 percent complete. A total of 1,339,302 gallons (5,069,811 liters) of grout has been injected into Q-holes for this section of the dam and 12,808 ft (3,904 m) of rock has been grouted. It is too early to predict the final outcome of Phase 7 of the grouting operation since closure of the curtain has not everywhere been achieved. However, there has already been a slight decrease in the flow through Weir 15, and based upon a pH monitor in the weir, direct flow conduits into the weir are still being encountered in grout holes at the predicted elevations.

Phase 8: Downstream Shallow Low Mobility Grout (LMG) Grouting (2013-Present)

The Weir 15 section of the E-line grouting had progressed to a point where further definition of the rock at depths greater than 328 ft (100 m), downstream geology, and flow conduits were desired. Additional

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deep into-rock piezometers were drilled downstream on the 1979 bolster to provide additional geologic model information and future grout monitoring instrumentation.

Difficulties in setting casing to the rock surface, communications of pressure and drilling fluid between drill holes and other instrumentation prompted further investigation into the condition of the upper 50 ft (15 m) of rock beneath the lower half of the east embankment of the dam. A review of historic drill log data, framed in reference to the recent drilling data and the communication anomalies, led APC/SCS to the conclusion that additional protection was needed to prevent the formation of sinkholes along this section of the dam.

The board of consultants recommended a corrective measure utilizing LMG grouting techniques to construct discreet grout columns to partially fill the voids while providing support of a thin "roof" of competent rock. Since the grouting would be performed on the downstream side of the dam, the grout injection process had to be conducted in such a manner as to not change the piezometric levels or potential flow directions of any shallow leakage within the upper fifteen meters of rock so as not to increase pore pressure and affect embankment stability. Therefore, a test program was conducted to develop the grouting plan and ensure that the required outcome could be achieved.

The initial test section consisted of four grout holes per row. With the exception of the angle holes, each hole was spaced 20 ft (6 m) apart. The grout holes were drilled to a depth of 150 ft (46 m) using the WDTH method. Using an LMG grout supplied by a local ready mix company, the 50 ft (15 m) of rock at each grout hole location were grouted in 4 ft (1.5 m) stages. Refusal criteria was either a maximum pressure (gage pressure minus line pressure) of 250 psi (1.7 kPa) or 5 yd^3 (3.8 m^3) of grout per stage. The objective was to form approximately 4 ft (1.5 m) diameter columns. At the interface of the overburden and the top of rock, the grout mix was adjusted to a higher slump and injected to form a cap along the top of rock wider than the grout columns. The wider cap formed at the top of each grout column was placed to provide support to the base of the embankment.

Based upon the information gathered during the initial test section, the test program was extended to include rows spaced on 20 foot centers along the area of concern. Two additional angle holes were added at each location to improve the treatment under the dam crest. A cross section showing the orientation and extent of the six grout columns per row beneath the zone of treatment is shown as Figure 6. At the time of writing this paper, 212 column locations have been grouted. A total of 1961 yd^3 (1499 m^3) of low mobility grout has been injected into the formation. Typically the take at each column location has been 9 to 10 yd^3 (7 to 7.6 m^3) which correlates to a 27 percent void ratio within the treatment zone.

The existing instrumentation within the area of treatment included deformation monuments along the downstream crest of the dam, piezometers and observation wells, and lateral toe drains. New piezometers were added in close proximity of the grout rows. Laser survey equipment was arranged on the slope of the dam or on the bolster immediately upstream of active grouting locations to monitor for heave.

Piezometric elevations were influenced by drilling and grouting but returned to normal following the completion of the grouting activities. There were no changes in the flow, clarity, or pH in the lateral toe drains. Near the completion of work, only three piezometers have been compromised by the grouting work as of October 2014. Measured deformation has been within threshold levels, both horizontal and vertical. Vertical deformation is measured with conventional digital levels and horizontal deformation is measured with RTK GPS. No occurrences of heave have been measured during the grouting sequences, although it is noted that some settlement has occurred concurrent with active grouting work that has returned to normal rates following the completion of work. The instrumentation and surveillance data indicate that the construction of the grout columns along the downstream half of the dam has not changed the existing piezometric levels, nor altered the preferred seepage direction of any shallow seepage, as was the intent using the column support approach.

Overall, a 600 ft (183 m) section of the east embankment is in the process of being improved to prevent the possibility of a collapse of the thin roof of competent rock. The corrective measure consists of constructing support columns by grouting with a low mobility grout. The work is proceeding as planned and, based upon the results to date, all the objectives are being achieved.

DEVELOPMENT OF THE GEOLOGICAL MODEL

As recommended by the board of consultants1, APC/SC project staff have recently been developing a geological model to help guide the direction that current and future grouting operations should take. Geological modeling is the applied science of creating computerized representations of portions of the subsurface based on integrating all available investigational data. At the Logan Martin Dam site, a wealth of subsurface data dating back to original construction in 1964 exists and has since been expanded by an array of subsurface explorations, optical televiewer, geophysical surveys, borehole caliper, dye tracing tests, instrumentation, and, of course, the findings of the grouting phases. A recent strategic initiative has been undertaken to consolidate all the available data to build a robust geological model that will help in the understanding of current phenomena and the forecasting of future events and needs.

It is clear from years of studies that the flow paths beneath Logan Martin Dam are highly complex; thus a clear understanding of how the site geology controls permeability and flow is of key importance. In undertaking grouting programs with the goal of significantly reducing seepage beneath the dam's east and west embankments, the current board of consultants recommended that a site-specific geological model was needed to successfully progress and target the foundation grouting. With concurrence by APC/SC staff and by FERC, the model development was initiated. It was agreed by all parties responsible for the project that such a model would be an invaluable tool to guide the direction that current and future grouting programs would take. The geological model that has been developed uses all current 3D computer modeling technologies and is a tool that guides the remedial grouting, identifying those areas of the foundation that require focused grouting. Figure 7 presents sample model output and depicts completed grouting of the downstream shallow LMG grouting. Areas of underseepage are now being prioritized for grouting based on a more quantifiable understanding of foundation rock structure, the development of local karst, and associated flow conditions. The grouting program at Logan Martin Dam is now one which is continuously refined and adjusted as additional data are fed into the model.

CONCLUSIONS

Logan Martin Dam has been in service for over 50 years, and was built on a karstic limestone foundation of extreme complexity and severity. The fact that it is still a fully functional operational facility, in which there is a high degree of confidence regarding its fundamental safety, is a testament to the continuing efforts of the grouting and geotechnical engineers who have monitored and directed successive phases of responsive remedial works over the decades.

Reflected in the various modifications to the grouting processes at Logan Martin Dam are the state-of-practice trends in national grouting practices (Weaver and Bruce, 2007). These include, for HMG grouting operations, the use of water-powered down-the-hole hammer drills; systematic use of permeability testing; injection of balanced, modified, stable grouts at the highest, safe pressures; and the routine use of computers to control, display and analyze all injection events. For LMG works, very close monitoring of pressure-volume-time characteristics for each stage is standard, together with the use of grouts of low slump and high internal friction. For all types of grouting, close monitoring of existing and ad hoc instrumentation is conducted to assure that the drilling and grouting are conducted in a safe and controlled manner, and to verify the impacts of the grouting.

A geological model has been created utilizing historic data, and is being continuously improved based on the drilling, grouting, and instrumentation data being currently generated. This model will shed light on the nature of the very complex geological and hydrogeological conditions at the site, and provide a sound basis for designing future phases of remediation.

Finally, with the execution of each successive phase of grouting and the associated improvements made to each, APC/SC project staff have continuously improved dam safety at the project. The west embankment has reached closure and all piezometers indicate temperatures at or slightly below groundwater and stable water levels. The focus of the current grouting effort is on the east embankment.

The project staff have updated internally operated drilling equipment, grout batch plant, down-hole logging, instrumentation, computer-based grout monitoring, and development of a 3-D model to aid in understanding the geological features of the variable karst foundation and in guiding current and future grouting.

Co-Authors Drs. Bruce and Greene have served on the Board of Consultants for Logan Martin Dam for the past 5 years.

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ISSN 1944-9836 - Association of State Dam Safety Officials

FIGURE 1. AERIAL VIEW SHOWING FROM LEFT TO RIGHT, THE WEST EMBANKMENT, CENTRAL POWERHOUSE, AND EAST EMBANKMENT WITH BOLSTER. TOTAL DAM LENGTH IS 6,133 FT (1,869 M). NORTH IS TO THE TOP OF THE PHOTOGRAPH.

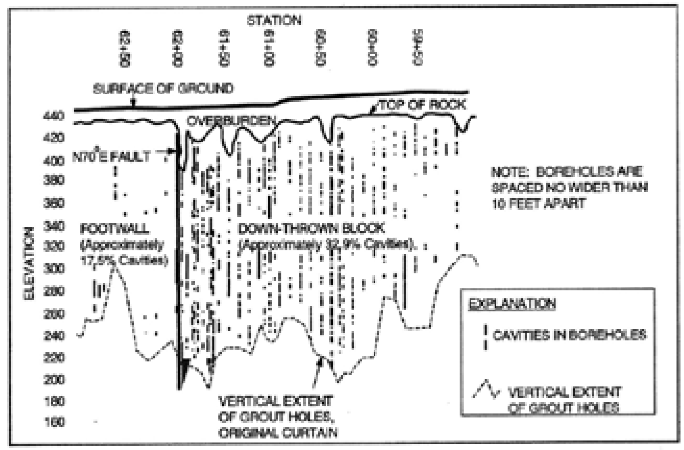

FIGURE 2. GEOLOGIC CROSS-SECTION DEPICTING A MAJOR NORMAL FAULT AND IMPACTS ON KARST CONDITIONS AT LOGAN MARTIN DAM (REDWINE, 1999).

FIGURE 3. PHOTOGRAPH OF OPENED FOUNDATION DURING CONSTRUCTION SHOWING PRONOUNCED KARSTIC SOLUTIONING ALONG STRUCTURAL GEOLOGIC FEATURES.

FIGURE 4. BACKFILLING OF THE CHIMNEY SINKHOLE ON THE EAST EMBANKMENT IN 1968.

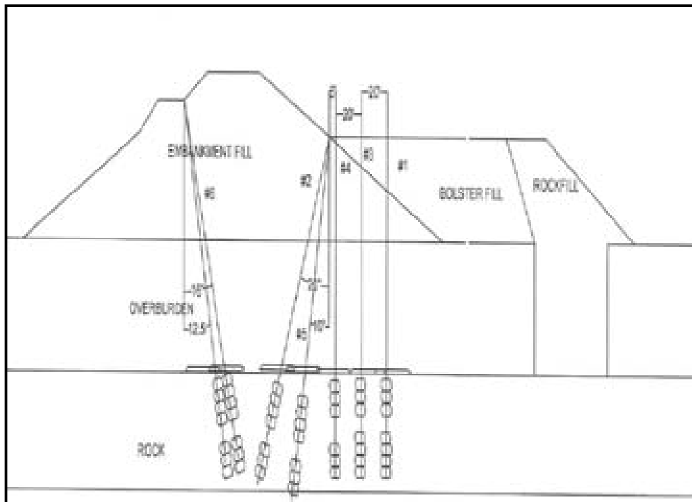

FIGURE 6. CROSS-SECTION DEPICTING LMG COLUMNS EMPLACED IN THE TREATMENT AREA BENEATH THE EAST EMBANKMENT TOE.

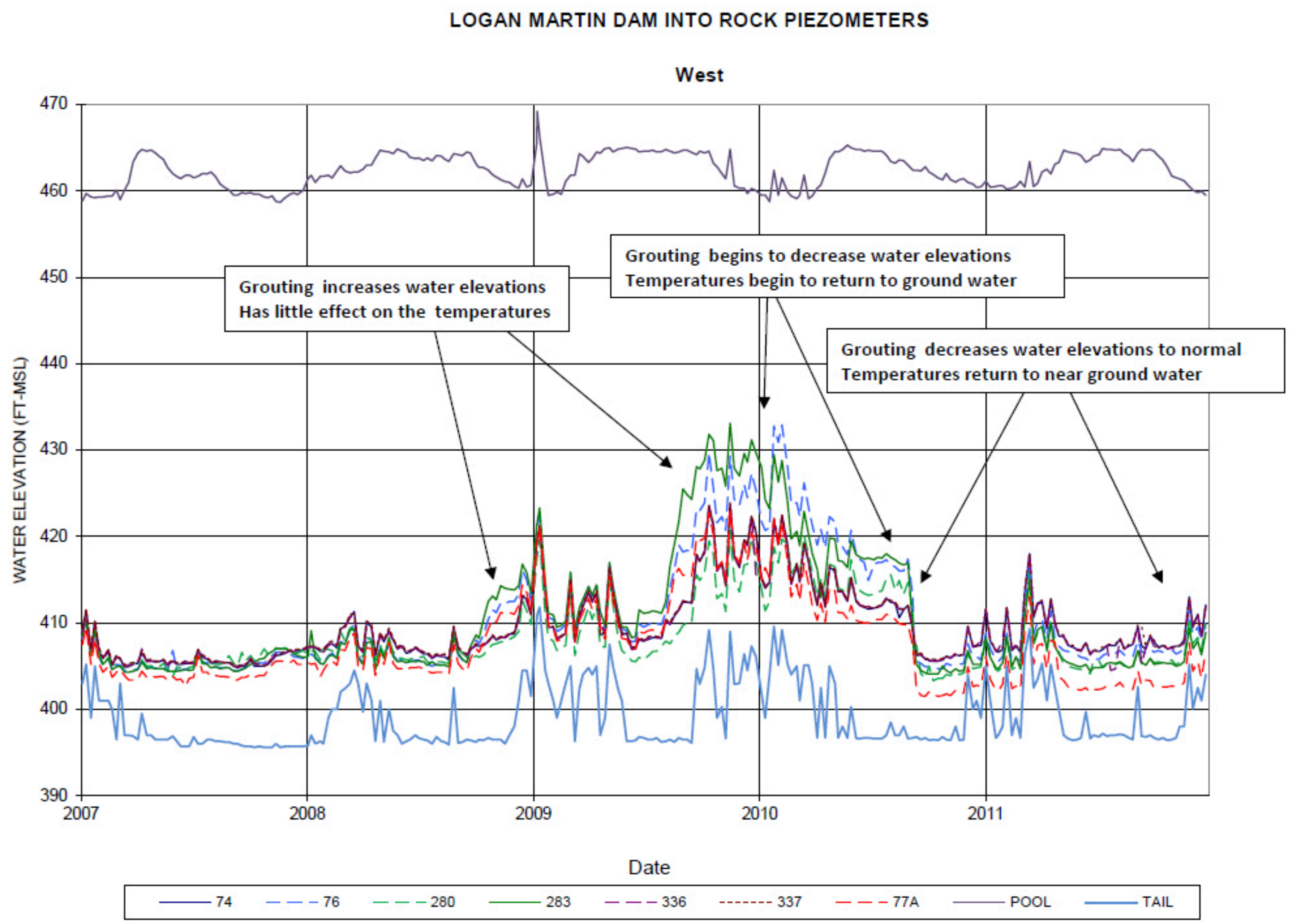

FIGURE 5. TYPICAL PIEZOMETER DATA FROM A CROSS-SECTION THROUGH THE WEST EMBANKMENT.

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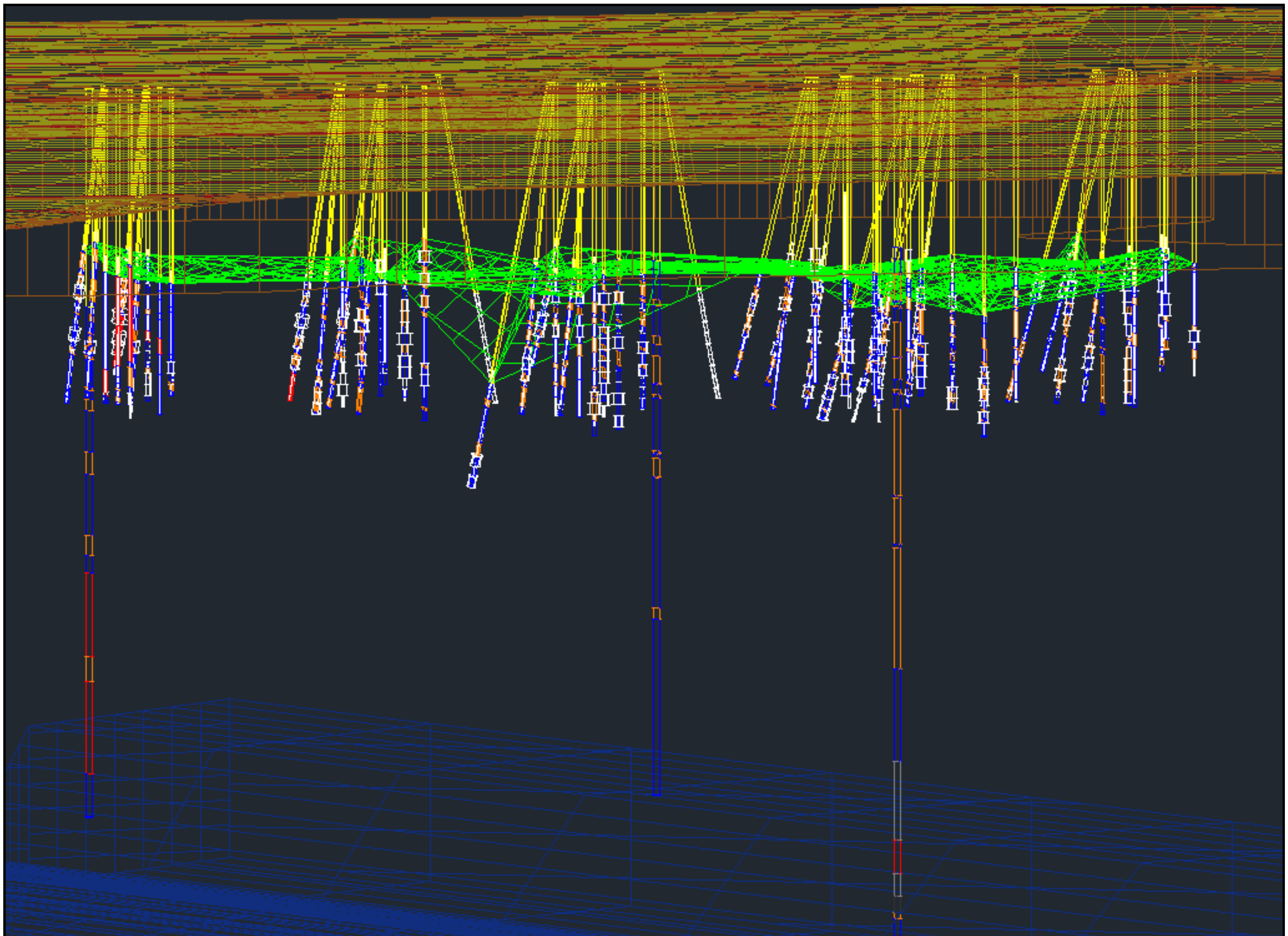


FIGURE 7. 3-D GEOLOGIC MODEL IMAGE BASED ON COMPLETED DRILLING AND GROUTING BENEATH THE EAST EMBANKMENT TOE. (NOTE TOP OF ROCK DEPICTED IN BRIGHT GREEN.)

TABLE 1. ORIGINAL CONSTRUCTION GROUTING (1960 AND 1964).

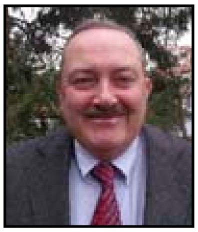
TIMEFRAME	TITLE	AREA	DEPTH	DETAILS
1960	Foundation Treatment	2300 ft (1,067 m)	161 ft (49 m)	Hole spacing down to 1.3 ft (0.4 m); 60% of grout was injected under gravity conditions. Filling cavities in rock foundation.
July and August 1964	Consolidation Grouting	East Embankment	Varied	Angle holes from the upstream crest through the concrete slab and 10 ft (3 m) into rock. Tied grout curtain to concrete slab.

TABLE 2. REMEDIAL GROUTING (1972-1977).				
TIMEFRAME	**TITLE**	**AREA**	**DEPTH**	**DETAILS**
April 1972 - May 1974	Remedial Grout Test Section	STA 56+00 to 61+38	173 ft (53 m)	Initially designed to reinforce top 100 ft (30 m) of rock and stabilize earth embankments
Dec 1973 - June 1976	Remedial Grout Test Extension	STA 45+00 to 48+00	173 ft (53 m)	
1976 - 1977	Grout Curtain Deeping	STA 60+93 to 62+43	173 ft (53 m)	Averaged 40.3% voids
Feb 1976 - June 1978	Remedial Grout Extension	STA 51+00 to 56+00	173 ft (53 m) or deeper	From Aug 1976 to Jan 1979, 6 APC and 3 contracted drill rigs, 3 grout plants, and 37 employees engaged in the works
Aug 1976 - Dec 1977	East Embankment	STA 62+23 to 66+26	173 ft (53 m)	
Sept 1976 - May 1977	West Embankment	STA 74+00 to 81+30	173 ft (53 m)	

TABLE 3. MULTIROW GROUTING (1977-1990).			
TIMEFRAME	**TITLE**	**AREA**	**DETAILS**
July 1977 - Oct 1978	Multirow Grout for Depression on Upstream Toe of East Embankment	STA 54+50 to 55+90	Placed downstream of a new depression. Included lines A, B and C
Aug 1977	Multirow Row Grout Line Opposite ’68 Chimney Sink	STA 60+13 to 62+63	To increase stability of the upstream side of the dam
Oct 1977 - Sept 1981	Multirow Extension	STA 55+90 To 56+40	
	Deeper Treatment of Multirow	STA 54+50 to 56+40	
	Second Multirow Grout Curtain, East Riverbank	STA 59+82 to 62+52	Deeper treatment from STA 60+75 to 62+10 to EL 200 ft (61m) meters for C line only
Sept 1981 - Aug 1984	Company Recommendations	See details	Deepen holes from STA 54+50 to 56+50; Experimental grouting on the 10 deep holes in the concrete structure; Determine source of near reservoir temperatures at piezometer 76 on toe of west embankment
Mar 1984	D line Addition to Multirow	STA 60+90 to 62+10	Seven primary holes on 19 ft (6 m) centers to EL 298 ft (91 m)
1985 - 1990	Periodic Grouting		Deeper treatment of selected holes and additional quaternary holes

TABLE 4. COMPACTION GROUTING 2005-2008.

TIMEFRAME	TITLE	AREA	DEPTH
Sept 2005	Compaction grouting, block F	East end of Concrete Spillway	5 borings with a combined take of 32 yd^3 (24.5 m^3) of low mobility grout
2006-2008	Compaction treatment of deep rock zone	Station 48+00 to 51+00	5 borings with a combined take of 24 yd^3 (18.3m^3) of low mobility grout



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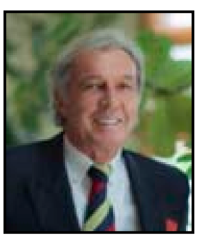

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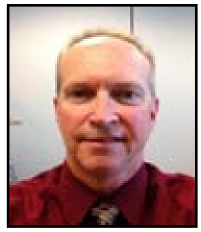

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