

Long-Term Grouting of a Karst Foundation at Logan Martin Dam, Alabama

Donald A. Bruce¹, Brian H. Greene², R. Craig Findlay³,
Bobby E. Williams⁴, John H. Williams⁵ and Richard L. Mickwee⁶

¹ Geosystems, L.P., P.O. Box 237, Venetia, PA 15367; e-mail: dabruce@geosystemsbruce.com

² Consultant, 1207 Colonial Place, Sewickley, PA 15143; email: greenegeoconsult@gmail.com

³ Findlay Engineering Inc., 70 Old Field Road, Yarmouth, ME 04096; email: cfindlay@findlayengineering.com

⁴ Southern Company Generation, 600 North 18th Street, Birmingham, AL 35203; email: bewillia@southernco.com

⁵ Southern Company Generation, 600 North 18th Street, Birmingham, AL 35203; email: jhwillia@southernco.com

⁶ Southern Company Generation, 600 North 18th Street, Birmingham, AL 35203; email: rlmickwe@southernco.com

ABSTRACT

Seepage through the karstic limestone foundation below Logan Martin Dam, AL, developed when the reservoir was first filled in 1964, producing springs in the downstream channel, on the riverbanks and eventually at the embankment toe. After a large sinkhole developed on the downstream face of the east embankment on April 9, 1968, the first of many remedial grouting campaigns began, and they continue to this day. This paper reviews the long history of grouting performed at the dam site. There have been many changes and improvements to the means and methods of grouting in recent years that have led to success in mitigating underseepage and improving dam safety. New challenges continue to present themselves. In 2012, exploration conducted at the site revealed that the upper 15 meters (m) of the karst under a rock buttress (“bolster”) at the base of the east embankment had about 30 percent voids, raising collapse concerns. In response, a method utilizing discrete grout columns formed using low mobility grout (LMG) was developed, evaluated and then fully implemented. This application of LMG column stabilization is unique and the paper discusses the development and assessment of the method, as well as full implementation of the remediation, including instrumentation and monitoring. Finally, the on-going work to continue deep curtain high mobility grouting along the axis of the east embankment of the dam is reviewed. The use of real-time grout monitoring, balanced stable grouts, an extensive instrumentation program and the development of a 3D geologic model are guiding the way forward towards assuring dam safety while guaranteeing power generation.

BACKGROUND

Logan Martin Dam is a hydroelectric facility owned by Alabama Power Company/Southern Company Services (APC/SCS) and is 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 1,682 m in length,

referred to as the east and west embankments, and a central concrete powerhouse section 187 m long (Figure 1). The west embankment is 265 m long and the east embankment is 1,417 m long. The maximum height of the dam is approximately 31 m above the riverbed.

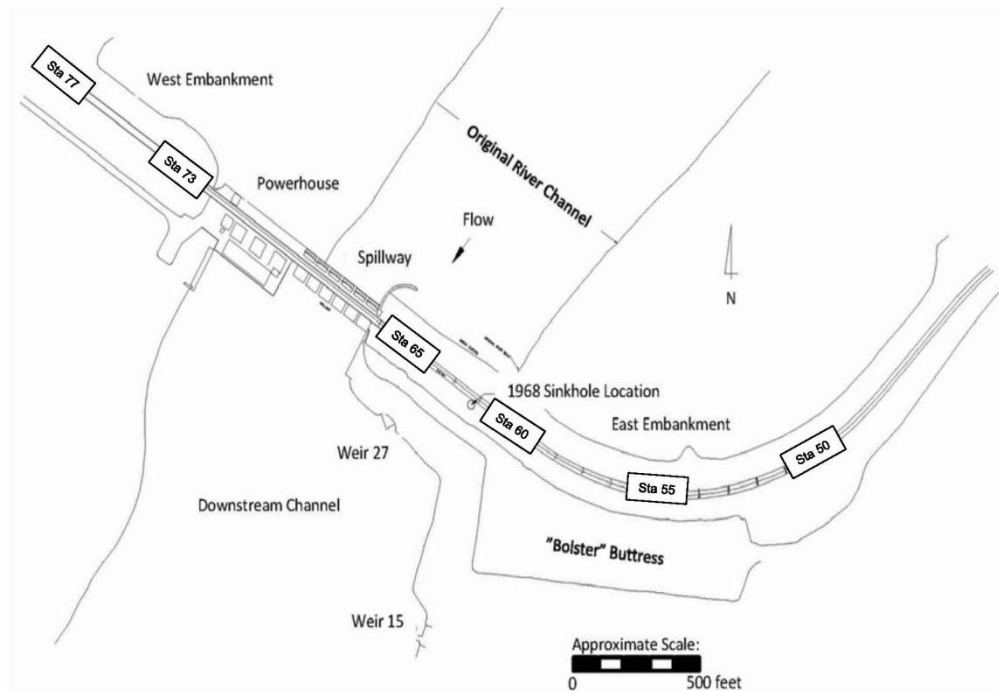


Figure 1. Project Layout Plan View (not to scale), showing stationing, and the locations of Weirs 15 and 27, the 1968 sinkhole, and the “bolster” buttress.

The 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. 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 HISTORY OF GROUTING AT LOGAN MARTIN DAM

Indicators of Distress

Immediately upon initial filling of the reservoir in 1964, mud flows were noticed downstream 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 mud flows were observed. On April 9, 1968 a sinkhole approximately 6 m in diameter and 16 feet deep opened just downstream of the east embankment crest and was referred to as the “chimney sink.” Immediate action was taken by site personnel to shut down the roadway across the dam and backfill the sinkhole. This sinkhole was a clear manifestation of the karstic conditions at the site and had the clear potential to fundamentally threaten dam safety. Numerous holes were drilled surrounding the sinkhole 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.

Details of the history and evolution of grouting at Logan Martin Dam have been provided at different times by Williams and Robinson (1997), Sprayberry et al. (2012), and Bruce et al. (2014), and the interested reader is invited to consult these publications accordingly.

The following is a brief summary of the eight successive phases of grouting from the initial foundation treatment in 1960 to the current works.

Phase 1: Original Construction Grouting (1960 and 1964)

The initial curtain consumed 94 m³ 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 well-developed nature of the karst, closure requirements (based on grout takes) required holes installed as close as 0.4 m to depths of 49 m. Consolidation grouting was performed after construction of the earth berm and a concrete base slab cast on the exposed rock surface under the upstream face of the dam in the original river valley. 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 karstic features 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 2.1 m downstream of the original grout curtain from Station 60+93 to 62+43 (note: stationing in feet), as it took less effort to create a work berm from which to grout. This was a 46 m long section and only extended 15 m into rock. For reference, dam station 60+00 is on the eastern (left) embankment and increases to the west (right) embankment. Stationing is shown on Figure 1.

The drilling for the emergency grout holes revealed almost half of the materials installed in the original grout curtain just 4 years earlier had been removed by erosion. Much of the removed material was found immediately beneath the concrete base slab referred to above. The

presence of this transported material under the slab indicated the slab may well have prevented a total breach of the dam. The emergency grouting work, and especially grouting around station 61+70, was instrumental in reducing the flow in Weir 27 from approximately 10,221 liters to approximately 7571 liters.

Phase 3: Grouting from 1972-1990

Several different grouting programs were performed in this phase. Each program built upon the previous grouting campaign to improve techniques or approaches. Due to the many sequential grouting programs foreseen, it was decided that APC/SCS continue to perform the grouting with their own equipment and personnel.

In late 1971, APC/SCS decided a more robust grouting program was needed to ensure the stability of the embankments. It was believed that reinforcement of the upper 33 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 each time, the curtains were extended or deepened to over 58 m.

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 ensure the overall stability of this portion of the embankment.

Phase 4: Deep Grouting Program (1991-2006)

After completion of these grout curtains, dam stability was judged to have been significantly 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 133 m into rock as summarized by Williams and Robinson (1997). 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 29 m of rock. This program successfully lowered the piezometric levels and further reduced flow through Weir 27.

After grouting a test section, the grout curtain was extended to the east to try to impact and reduce Weir 15 flows. This operation was performed from 1991 to 1999 from station 52+15 to 59+85. While drilling core hole 53+12 between El. 90 and 70, there was a collapse of the water supply hose down the hole, suggesting the presence of a true void.

Much of the success of this grout line 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 41 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 grout were unsuccessful. Burlap strips and polypropylene sacks were added to the grout (548 m³) and 700 metric tons of sand and gravel were injected. For the lower cavity grouting, 2-inch round gravel sluiced with grout was injected for 2.5 months. A total of 1427 m³ of cement, 381 m³ of C-flyash, 835 metric tons of sand and gravel, 449 m³ of round gravel, and 11.8 metric tons of lightweight aggregate were injected into this hole. Additional grout holes 1.4 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 4,921 liters and also reduced the rate of settlement of the dam crest in this area.

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. Reductions in permeability of 60 to 100 percent were recorded by comparing pre- and post-grouting permeability tests.

Phase 5: Compaction Grouting Program (2005-2008)

Several of the previous grouting investigations had found zones of loose, unconsolidated soils in the overburden under the embankment. It was decided that these areas would require treatment through compaction grouting.

Due to the angled walls at the east end of the concrete spillway, compaction of the adjacent soil was judged inadequate. In response to the Part 12 Potential Failure Mode Analysis (PFMA) team's findings, compaction grouting was required for this portion of the embankment. Five borings were drilled and a total 24 m³ of LMG 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 per foot.

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 determine if the overburden in the top 9 m of rock would accept the low mobility grout (LMG) in significant quantities. Five borings drilled for the investigation of the upper 9 m. of rock only took 18 m³ of grout. Therefore, this area was considered stable and not in need of further treatment.

Phase 6: West Embankment Grouting (2008-2010)

In 2002, a group of piezometers on the west side of the 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 water level elevations measured by 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 had earlier experienced an increase in elevation in 2002. An overburden to rock interface and shallow rock grouting program was then initiated.

Following a long period of HMG grouting, 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 as recorded by the network of piezometers 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 west embankment has reached closure and all piezometers indicate temperatures at or slightly below groundwater and stable water levels.

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. All deformation readings taken along the west embankment have remained stable through present time.

Phase 7: Deep Curtain Regrouting (2011-Present)

This phase is the completion of the Quaternary holes (“Q-holes”) between stations 52+00 and 60+05 (Phase 4) designated as the Weir 15 section. There are about 80 grout holes for this phase, and each hole extends 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 fifteen meters of rock to mitigate ongoing settlement along this section of the dam.

To date, this phase of the grouting program is over half complete. A total of 5,069,810 liters of grout has been injected into the Q-holes and 3,903 linear meters of rock stages had 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 been achieved in all areas of the Phase 7 reach of grouting. 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. The innovations now being routinely employed in the Phase 7 work are discussed below.

Phase 8: Downstream Shallow Low Mobility Grout (LMG) Grouting (2013-2015)

The Weir 15 section of the Deep Curtain Regrouting (Phase 7) had progressed to a point where further definition of the rock at depths greater than 100 m, downstream geology, and flow conduits were desired. Additional deep into-rock piezometers were drilled downstream into a stabilizing berm (“bolster”) installed in 1979 as shown in Figure 1. These additional piezometers

were installed to provide additional geologic model information and future grout monitoring instrumentation.

Difficulties in setting casing at the rock surface, communications of pressure and drilling fluid between drill holes and other instrumentation data prompted further investigation into the condition of the upper 15 m of rock beneath the downstream section of the east embankment (Figure 1). 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.

As detailed by Findlay et al. (2015), a very systematic approach to the development of the design and construction of this phase of work was adopted.

The Board of Consultants recommended a corrective measure utilizing LMG grouting techniques to construct discrete grout columns to partially fill the voids while providing structural 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 15 m 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 6.1 m apart. The grout holes were drilled to a depth of 46 m using Wassara water-powered down-the-hole hammer (WDTH) drilling methods. Using a low mobility grout (LMG) supplied by a local ready mix company, the upper 15 m of rock at each grout hole location was grouted in 1.5 m stages. Refusal criteria were either a maximum pressure (gage pressure minus line pressure) of 1,724 kPa or 3.8 m³ of grout per stage. The objective was to form approximately 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 to form a “mushroomed” 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 of grout holes spaced on 6.7 m centers along the area of concern. Two additional angle holes were added from the upstream berm at each location to improve the treatment under the dam crest. A total of 1,648 m³ of LMG has been injected into the formation into 263 columns. Typically the take at each column location has been 7 to 8 m³ which correlates to a 27 percent void ratio within the treatment zone. The as-built configuration is shown in Figures 2 and 3.

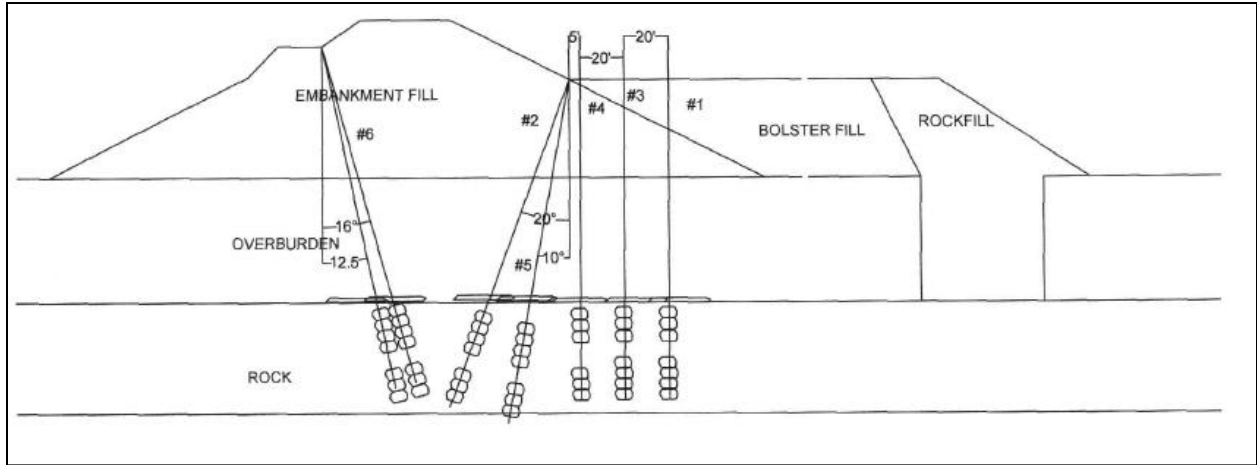


Figure 2. Final Configuration of LMG Column Layout at Each Station (not to scale).

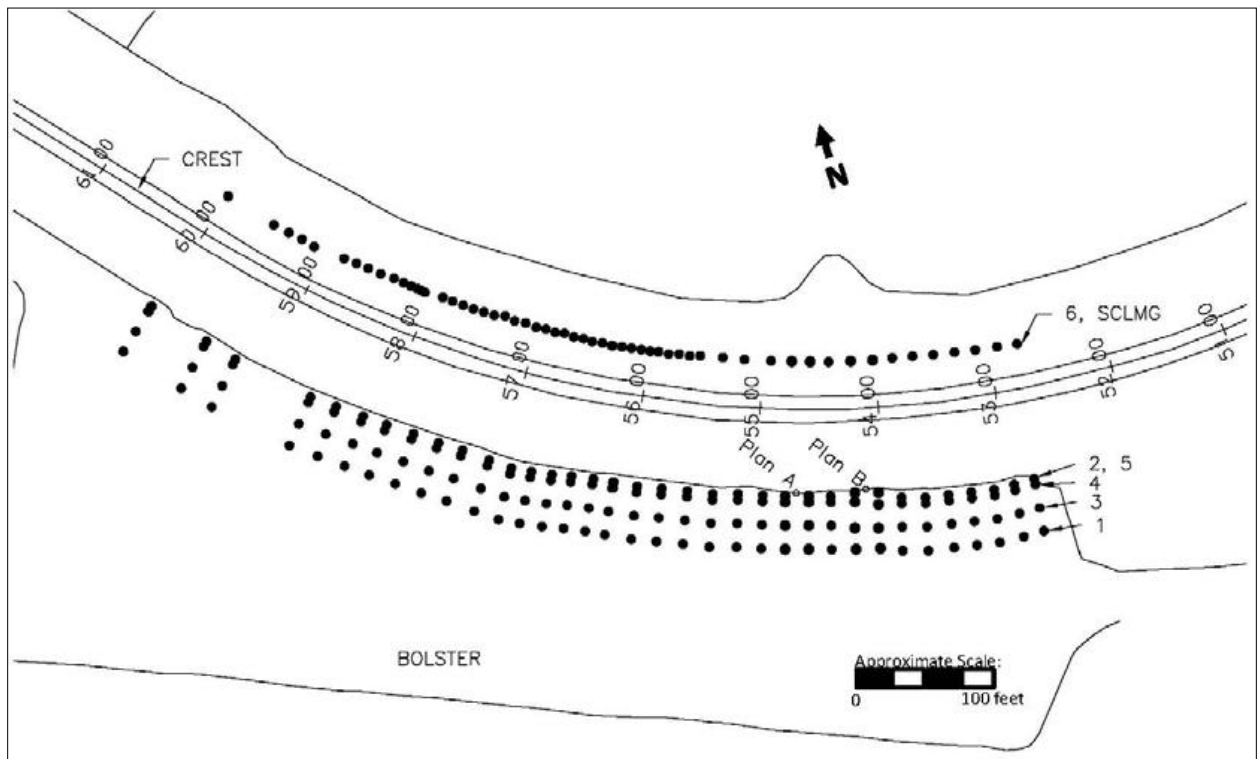


Figure 3. Final Plan of LMG Column Locations at Ground Surface as Constructed (black dots).

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 to 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 occasionally influenced by drilling and grouting activities 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. Only three piezometers were compromised by the LMG grouting. Measured deformations were within threshold levels, both horizontal and vertical. Vertical deformation was measured with conventional digital levels and horizontal deformation was measured with real time kinematic global positioning system (RTK GPS) equipment. No occurrences of heave were measured during the grouting sequences, although it was noted that some settlement occurred concurrent with active grouting work, but it 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 of using the column support approach.

TECHNICAL INNOVATIONS ADOPTED IN THE DEEP HMG GROUTING PROGRAM (PHASE 7)

All grout holes are drilled with a track-mounted Cubex QXW1710 drill. The drilling is conducted with a Wassara water-powered down-the-hole hammer (WDTH) as described by Bruce et al. (2013). The advantages of the WDTH include a cleaner operation, less harm to the foundation and minimal hole deviation. Each stage is a maximum of 9.1 m long. A virtual log of each grout hole is provided by 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 OTV and caliper data, a high risk would have existed 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 engineers in estimating the likely magnitude of the grout take. The test is typically a 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. Large voids/cavities can be encountered between depths of 83 m and 125 m. These voids can require grout with bulk fillers such as sand and gravel. A Häny grout plant is used to deliver these materials within a medium mobility grout mix (MMG).

The grouting project has been, and continues to be, a laboratory for developing stable grout mixes that are effective in karst formations. 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 HMG's now used contain cement, water, bentonite, and Glenium (superplasticizer). All mixes satisfy acceptable values for the pressure filtration coefficient, Marsh flow, bleed, and shear resistance.

An in-line data logger and computer systems are utilized to record flow, measure the grout pressure, and compute the "apparent" Lugeon value in real time. These parameters are used to confirm proper grout refusal for each stage (typically around 5-10 liters per minute per stage).

The instrumentation and monitoring of the dam during the grouting process has also been enhanced. 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.

As recommended by the Board of Consultants, APC/SCS project staff have recently developed 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 Logan Martin, a wealth of subsurface data dating back to original construction 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 successive grouting phases. A recent strategic initiative has been undertaken to consolidate all the available data to build an even more 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 the dam are highly complex, and so a clear understanding of how the site geology controls permeability and subsurface flows 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/SCS 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. Areas of underseepage are now being prioritized for grouting based on a more quantified 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. It was built on a karstic limestone foundation of extreme structural complexity and severity of solutioning. The fact that it is still a fully functional, licensed 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 hammers; 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 work, 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 continues to shed light on the nature of the very complex geological and hydrogeological conditions at the site, and provides 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/SCS project staff has 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 and treating the geological features of the variable karst foundation, key zones of underseepage flows and for guiding current and future grouting.

REFERENCES

- Bruce, D.A., Lyon, R. and Swartling, S. (2013), The history of down-the-hole drilling and the use of water-powered hammers, Association of State Dam Safety Officials Annual Conference, Providence, RI, September 8-12, 11 pp.
- Bruce, D.A., B.H. Greene, B.E. Williams, and J.H. Williams (2014). "Evolution of Grouting in Karst at Logan Martin Dam, Alabama," Association of State Dam Safety Officials, The Journal of Dam Safety, Volume 12, Issue 4, p. 21-35.
- Findlay, R.C., Greene, B.H., Bruce, D.A., Williams, B.E., Williams, J.H., and Mickwee, R.L. (2015). "Unique Use of Grout Column Support of Karst Features below Logan Martin Dam, AL"; 2015 ASDSO Annual Convention, New Orleans, LA.
- Redwine, J.C. (1999). "Not Your Typical Karst: Characteristics of the Knox Group, Southeastern, U.S., Hydrogeology and Engineering Geology of Sinkholes and Karst," Pettit & Herring (eds.), A.A. Balkema, Rotterdam, pp. 111-119.
- Redwine, J.C. (2014). Personal Communication, Anchor QEA, LLC, One Perimeter Park South, Suite 100N, Birmingham, AL 35243.
- Sprayberry, S.S., Williams, B.E, Espinal-Todd, S., and J.D. Holland, (2012). "A Case History of Grouting at Logan Martin Dam," Deep Foundation Institute, 4th International Conference on Grouting and Deep Mixing, New Orleans, LA, February 15-18, pp. 1204-1213.
- Weaver, K.D. and Bruce, D.A., (2007). Dam Foundation Grouting, ASCE Press, Reston, VA.
- Williams, B.E. and Robinson, R.L., (1997). "Grouting to Great Depths, Civil Engineering," American Society of Civil Engineers, Vol. 67, No.2, December 1997, pp. 47-49.