Unique Use of Grout Column Support of Karst Features below Logan Martin Dam, AL

Findlay, R.C., Findlay Engineering; Greene, B.H., Consultant; Bruce, D.A., Geosystems, L.P.; Williams, B.E., Southern Company Services; Williams, J.H., Southern Company Services; Mickwee, R.L., Southern Company Services.

Abstract -- Seepage through karstic limestone under Logan Martin Dam increased when the reservoir was first filled in 1964, producing springs in the downstream channel, at the riverbanks and eventually at the embankment toe. After a sinkhole developed on the downstream slope of the east embankment in 1968, the first of many remedial grouting campaigns began, some continuing to this day. The focus of this paper is a rock buttress ("the bolster") placed against a downstream portion of the east embankment to mitigate the potential of breach should a new sinkhole develop. In 2012, exploration through the bolster revealed that the upper 50 feet of the karst foundation had about 30 percent voids, raising collapse concerns. In response, a method utilizing discrete grout columns formed using low mobility grout was developed, evaluated and then fully implemented. This paper discusses the development and assessment of the method, as well as the full implementation of the remediation and associated instrumentation and monitoring.

I. Project Description and Geology

Logan Martin Dam is a hydroelectric facility owned by Alabama Power Company (APC) 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 5,462 feet (1,665 m) in length, referred to as the east and west embankments, and a central concrete intake/powerhouse and spillway section 613 feet (187 m) long (Figures 1 and 2). The dam has a maximum height of about 100 feet (30 m), and the impounded reservoir has a total storage capacity at the normal maximum operating level (elevation 465 feet) of 273,260 acre-feet.

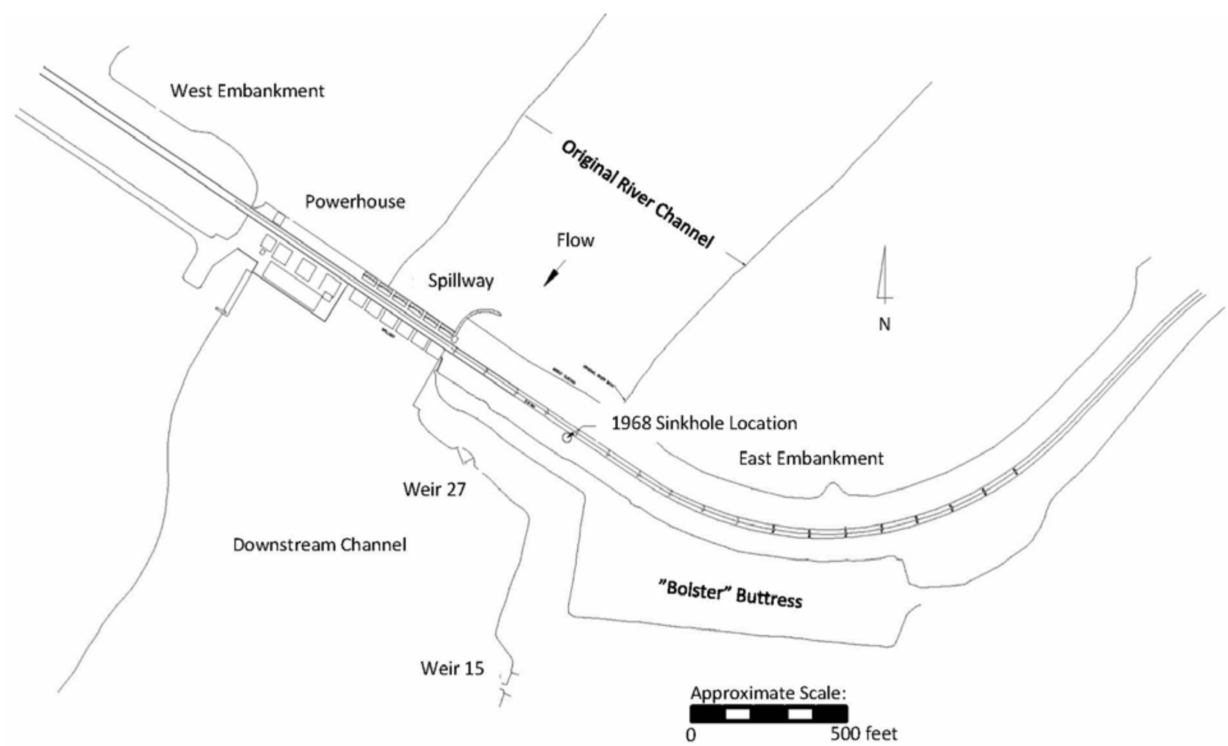

Figure 1. Project Layout Plan View (not to scale)

Figure 2. Aerial View of Logan Martin Dam

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 (Figure 1) 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 20 feet (6 m) in diameter and 16 feet (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 and backfill the hole likely prevented failure of the dam. This sinkhole was a clear manifestation of the karstic subsurface conditions at the site. Numerous exploratory holes were drilled surrounding the sinkhole to determine its extents. In addition, piezometers were installed across the site to further monitor water levels and temperatures. This event also highlighted the need for further grouting programs. Several campaigns of remedial grouting, mostly for seepage control, were conducted and presently grouting programs continue to be conducted. The history of the grouting programs to date has been summarized in two recent (as of this writing) technical papers,[1,2] as well as several other earlier papers.

With regard to the geology, the dam site it is underlain by Paleozoic-age sedimentary carbonate rocks. As documented by Williams and Robinson (1997)[3] and Redwine (1999)[4], the rocks comprising the foundation are primarily dolomite, chert and breccia with lesser amounts of sandstone and limestone, part of what is known as the "Knox Group". Locally, these rocks have undergone extensive faulting and folding which has created oriented discontinuities that have facilitated the development of solutioning and other karstic features. This has resulted in the development of multiple seepage paths under the dam along stratigraphic zones, bedding planes, joints, fold axes, and faults. Solutioning has penetrated to depths that are below elevation 0 throughout this site area, which is a depth of over 500 feet (152 m) below the crest of the dam. Conditions generally shallower than this extreme depth are the contributing factors to the seepage of concern to the dam and the

---

[1] Bruce, D.A., Greene, B.H., Williams, B.E. and Williams, J.H., (2014), "Logan Martin Dam, Alabama; 45 Years of Remedial Grouting, Proceedings of USSD 34th Annual Conference "Dams and Extreme Events", San Francisco, CA.

[2] Bruce, D.A., Greene, B.H., Williams, J.H., and Williams, B.E., (2014), "Evolution of Grouting in Karst at Logan Martin Dam", ASDSO Dam Safety Journal, Volume 12, Issue 4, pp. 21-35.

[3] Robinson, R.L. and Williams, B.E., (1997), Deep leakage through the karstic foundation at Logan Martin Dam, Waterpower 97, Proceedings of the International Conference on Hydropower, Trondheim, Norway, A.A. Balkema, pp. 806-813.

[4] 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.

subsequent grouting activities.

In the tailrace of the dam is a tight, doubly plunging syncline that can be observed at low outflow conditions. The foundation beneath the 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 N72°E and crosses the east embankment of the dam (Robinson and Williams, 1997). 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 N85°E to75°W 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). A photograph of showing well-developed solution features that were revealed during construction is presented as Figure 3.

Figure 3. Well Developed Solution Features Revealed During Construction

Because of the challenges the project presents with regard to the karst foundation issues, the dam has undergone almost constant grout remediation campaigns since the 1968 sinkhole. The project and its staff has maintained a Board of Consultants (BOC) during a much of the time since 1968 to review and advise on remediation activities. The work discussed in this paper represents but a very small portion of the considerable effort and resources expended since construction to keep Logan Martin Dam safe and viable.

II. CONCERN REGARDING FOUNDATION CONDITIONS IN THE BOLSTER AREA

In 2012, Alabama Power Company/Southern Company Services (APC/SCS) was conducting a grouting program at what was referred to as the E-line, in the dam and foundation upstream of the bolster area. 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 feet (100 m), mapping of geology downstream of the dam, and identification of potential flow conduits was desired. Additional deep into-rock piezometers were drilled downstream of the dam on the bolster to provide additional information and future grout monitoring instrumentation. Difficulties in setting casing into the rock surface, communications of pressure and drilling fluid between drill holes and other instrumentation prompted further investigation into the condition of the upper bedrock beneath the downstream half of the dam. The interval of drilling difficulties coincided with a zone of ongoing crest settlement between dam Stations 53+00 and 60+00 (Figure 4) that had been noted in deformation monitoring, adding to the concern of this newly identified area. A review of historic drill log data, framed in reference to the recent drilling data and the communication anomalies, led APC/SCS to reach the conclusion that additional protection was needed to mitigate the potential of sinkholes developing along this section of the downstream shell of the dam. This interval of concern coincides with the interval referenced above as the bolster, and was estimated to be approximately 700 feet (212 m) along the axis of the dam and about 150 feet (46 m) wide. The initial borings through the untreated upper bedrock zone found it to be about 50 feet (15 m) thick, consist of about 30% voids (filled and unfilled), and that it seemed underlie a relatively thin "roof' of competent rock.

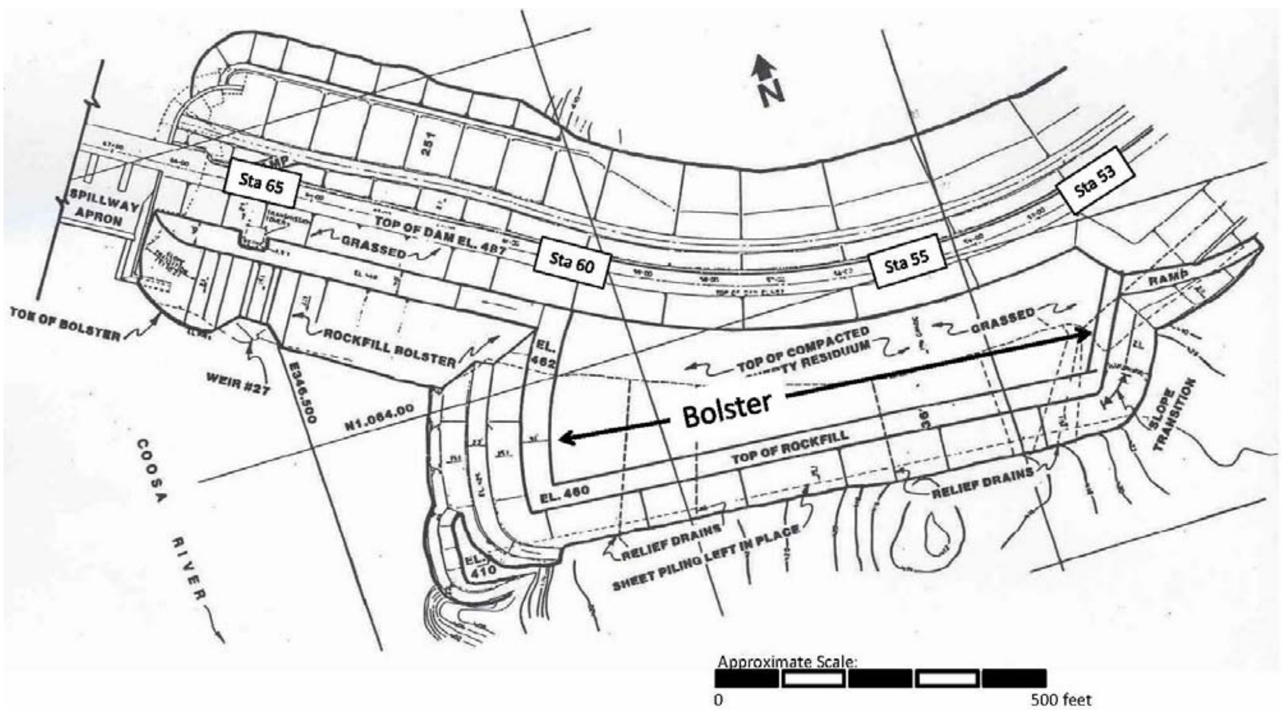

Figure 4. Plan of Bolster Area with Stationing

There was some question as to whether or not this simplistic model of the conditions under the bolster may be unrepresentative, since deep vertical karstic features may well penetrate the thin "roof" rock, significantly weakening stability and reducing the ability to bridge any collapse area that might develop. The project's concept was therefore to improve this zone to guard against sudden sinkhole development. Although not judged to be an immediate dam safety issue, the APC/SCS project team felt that this was a concern which should be addressed forthwith, and received significant attention prior to and during a scheduled meeting of the Licensee, the Federal Energy Regulatory Commission (FERC) and the Board of Consultants (BOC), which included a brainstorming session on February 27, 2013. The identified condition was discussed at length during the meeting, and at the time it was the consensus of the participants to investigate methods to improve this zone by grouting. Two methods, one utilizing discrete grout columns with Low Mobility Grout (LMG); and the other taking a block infilling approach, using Medium and/or High Mobility Grout (MMG and/or HMG) were debated as possible remedial measures. As a result of the session, the following provisional points were presented by the BOC, subject to further consideration and expansion as more information became available:

- The "roof collapse" phenomenon was regarded as a credible Potential Failure Mode (PFM), meriting remediation.

The BOC therefore concurred that this PFM be addressed and supported the priority placed upon it.

- An audit/evaluation of all existing dam instrumentation in the immediate vicinity was recommended to be conducted. Additional (shallow) piezometers might be needed to monitor the effects of the remediation to address the BOC's concerns about possibly raising the piezometric levels under the embankment, and issues relating to short- and long-term settlement.
- The BOC had concerns that a complete "block" infilling exercise could cause a rise in piezometric levels under the dam (i.e., between the downstream toe and the upstream grout curtain) and noted the demonstrated difficulties inherent in controlling the travel of HMG in these geological conditions. Therefore, the concept was discussed of installing discrete grout columns of LMG, in the fashion of a column-supported embankment structure. A multi-hole test program was recommended at the location to be conducted as soon as possible. This test program could be expanded to experiment with different grouting concepts, if thought beneficial.
- Measures of success would include a) confirmation that grouting parameters and limits have been met, b) no permanent piezometric raise has been created, c) no short-term settlements would occur. In the long-term, a further measure of success would be the diminution in the rate of crest settlement.
- The BOC stressed that it provided these observations as preliminary and conceptual ideas. The BOC acknowledged that the responsibility for designing and implementing the final solution; whatever it may be; would lie with the APC/SCS project team, subject to BOC and FERC review. It was recommended that as soon as possible, a series of detailed plans should be developed by the APC/SCS project team, including a drilling and grouting design; method statement and QA/QC plan; a focused instrumentation and monitoring plan; and a focused "event response" plan.

The discrete column concept as discussed in the BOC meeting was (and is) believed to be a very unique approach to providing collapse protection for a downstream portion of an embankment founded on karstic strata. LMG grouting techniques were proposed to be utilized to construct discreet grout "columns" to partially fill the voids while providing support of the thin "roof" of competent rock. It is noted that rather than true columns of LMG grout, the "columns" would merely displace the soil infilling in the limestone cavities, forming a solid "column" of grout and intact limestone within a target 4 foot diameter to provide structural support of the overlying overburden from collapse into a widening solution void. At the interface of the overburden and the top of rock, the concept was that the grout mix could be adjusted to a higher slump and injected to form a "mushroom" cap along the top of rock wider than the grout columns to broaden the support against collapse. 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. It was key that free drainage be allowed between the emplaced columns so as not to increase pore pressures in the embankment and foundation soils which could have the effect of decreasing slope stability.

III. FIELD TEST PROGRAM

A. Development of the Plan

Following the BOC meeting, APC/SCS performed the recommended instrumentation audit and additional instrumentation locations were selected, with the understanding that adjustments based on the findings of the test sections would likely be needed as work progressed. APC/SCS developed a test section program to assess both the "block" infilling and discrete grout column methods, with the intent that a detailed plan for the production stage of the remediation program would be based on the results of the test program. The test program proposed, including discrete columns (Plan A) and in-block filling (Plan B) is summarized as follows:

<u>Plan A- LMG Grout Columns</u> - The test section for Plan A was proposed to be constructed from Station 54+60 to Station 54+80 and consisted of two rows with four grout holes per row (Figure 5). It was proposed that each hole be drilled with a Casagrande M-9 drill. A grout hole spacing of 20 feet (6 m) was determined based upon a review of the fracture patterns of the rock shown in construction photos of the old river bed. The hole closest to the slope of the dam was drilled at a 15 degree angle upstream. The overburden was drilled with a 4-inch (10.1 cm) drag bit or claw bit and the rock was drilled utilizing a $3^{7}/_{8}$-inch (9.8 cm) tricone bit. Following completion of the drill hole (about 150 feet or 46 m deep), NW casing was set to within 5 feet (1.5 m) of the bottom of the hole. The grout hole was stage grouted using the following mix (per cubic yard):

- 350 lbs. (159 kg) of cement
- 2000 lbs. (909 kg) of concrete sand
- 1200 lbs. (545 kg) of fly-ash
- Enough water to yield a 3 to 4-inch (7.6 to 10.1 cm) slump (in the range of 40 gallons or 151 liters)

As a grouting termination criterion, each five feet stage were grouted until an effective pressure (gage pressure minus line pressure) between 150 and 200 psi (1,033 to 1378 kPa) was reached or about 3.5 cubic feet (0.1 cu m) of grout was injected, whichever came first. The objective was to form approximately 5 foot (1.5 m) diameter columns (based on a 30 percent void ratio). A Schwing BP450 concrete pump was be used to inject the grout at a maximum rate of 5 cubic feet per minute (0.14 cu m/min) through a 3-inch (7.6 cm) diameter grout hose. The grout casing was then raised to the next 5-foot (1.5 m) stage and the process repeated. This procedure was followed until grout columns were constructed within the upper 50 feet (15.2 m) of cavernous rock. 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. As for the sequence of grouting, the downstream hole was grouted first followed by the upstream hole. No more than two holes were open at a time. Following the completion of these two holes, the middle grout holes were drilled.

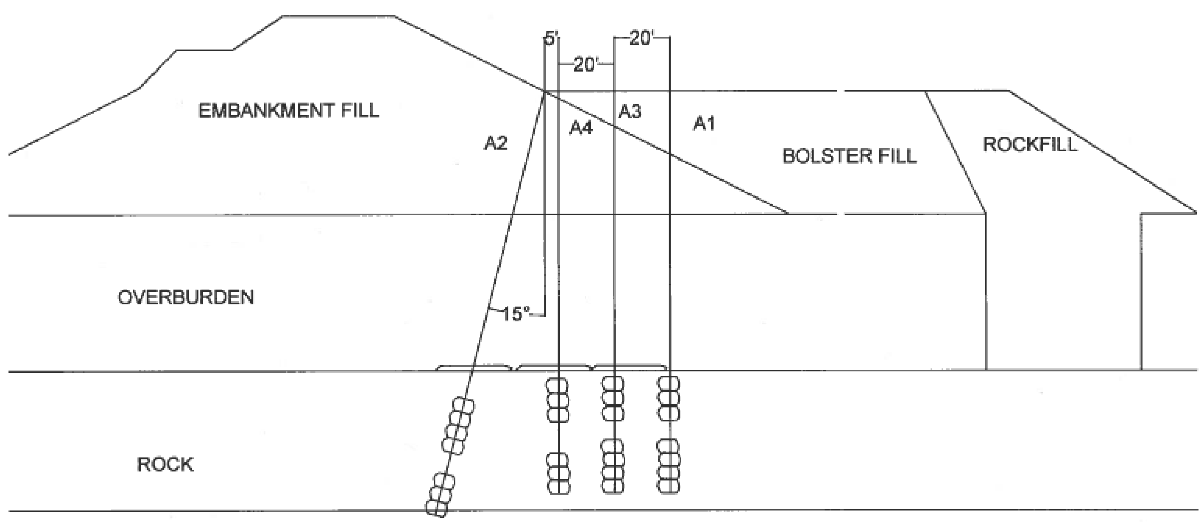

Figure 5. Plan A- LMG Grout Columns (not to scale)

Plan B- Void Filling with More Mobile Grouts - The test section for Plan B was located from Station 54+00 to Station 54+20, and was considered to be in-block filling of the top 50 feet (15.2 m) of the karst. There were two rows with three grout holes per row (Figure 6). The hole closest to the slope was drilled at a 15 degree angle in the upstream direction. All the grout holes were drilled with a Casagrande M-9 drill. With the exception of the angle holes, the spacing was 20 feet (6.1 m). The overburden was drilled with a 4-inch (10.2 cm) drag bit or claw bit. Four inch (10.2 cm) inside diameter casing was then reamed through the 4-inch (10.2 m) hole to the top of rock using J-teeth on the starter casing. Fifty feet (15.2 m) of rock was drilled with a $3^{7}/_{8}$ inch (9.8 cm) tricone bit. Upon completion of the hole, a header was attached to the top of the casing, and the fifty feet (15.2 m) stage of rock was grouted. Different grout mixes were used ranging from a 1:1 mix (by weight) with 4% bentonite to a 0.5:1 by weight. Sand was also a possible additive to thicken mixes. Maximum pressure was 40 psi (276 kPa). The initial grout limit per hole was 500 cubic feet (14.1 cu m); however, adjustments were be made if grout was detected in the surrounding instrumentation. The same drilling and grouting sequence used for Plan A was applied to Plan B. If cave-in within the drill hole became a problem, downstage grouting could be used.

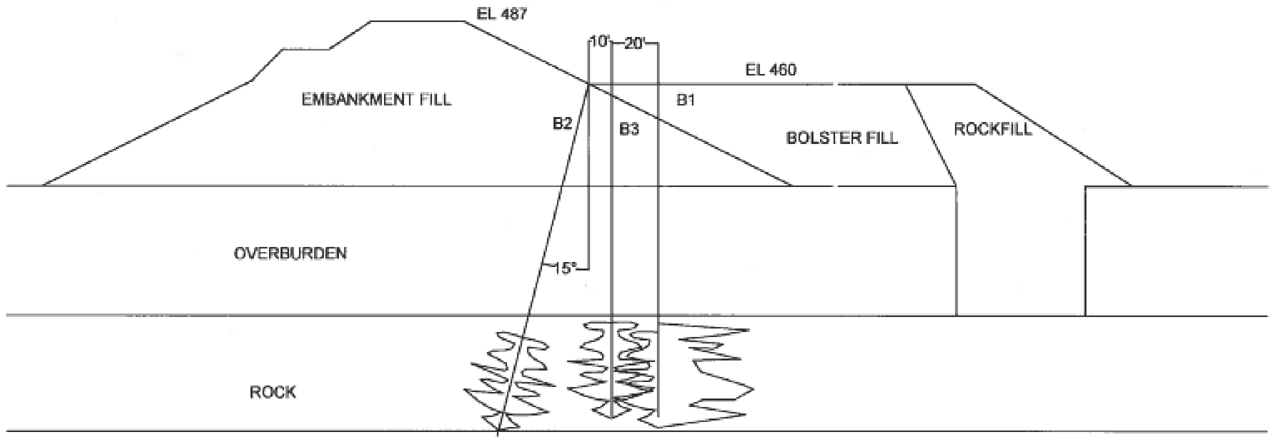

Figure 6. Plan B- Void Filling with More Mobile Grouts (not to scale)

APC/SCS indicated in the plan description presented to the BOC and FERC for review and comment was that one of the advantages of choosing the locations listed above for the test sections was the existence of eleven piezometers within a 150 feet square (14.2 sq. m) area starting from the crest and continuing 150 feet (46 m) downstream of the crest. The existing piezometers included three screened at the top of rock, two with tips in the longitudinal drain, four screened in the embankment, and two screened deep into rock. Additional piezometers were added at both test locations, and monitoring during construction would include monitoring of existing lateral drains under the bolster for grout shows and flow changes. Select piezometers and drains were monitored for pH changes. In addition, a laser survey array was established in order to monitor for any heave during the work.

As recommended by the BOC, an event response plan was also developed. For the discrete column test section, a rise in piezometer levels within the vicinity of the test area was to be evaluated immediately. During the grouting process some increase in piezometric level was anticipated. However, the levels were expected to return back to normal in the piezometers installed into the rock. It was recommended that any increase in levels within the embankment piezometers would require the grouting process to stop for further evaluation. In addition to piezometric levels and pressures, the ground surface in and around the test section was monitored for heave. The plan was that grouting operation would be stopped if any heave was detected, and grouting pressures would be adjusted to prevent heave.

An event response plan was also developed for the in-block filling test section. Due to the potential for grout travel in different directions, more instrumentation was installed around the test area than in discrete column test section. For instance, two new piezometers with 50 foot (15.2 m) screens were installed to detect any grout travel. In the plan, if grout were to reach these piezometers, the mobility of the grout was to be re-evaluated. Other piezometers existed to record level or pressure changes. It was recommended that a sustained increase in the water level in an embankment piezometer would require an evaluation as to whether or not any seepage was being diverted in an upward direction beneath the downstream shell of the dam. If the lateral drains increased in flow or carried sediment/muddy discharge, the grouting operation was to be stopped. The test area was monitored for surface movement. Due to the use of the more mobile grouts at the block infilling test area, however, no movement was considered likely to occur. If it should, the plan was that grouting operation would be stopped.

B. Field Implementation of the Test Program

Following development of the test section plan, with review by the BOC and the FERC, APC/SCS initiated the field work. Upon completion of the test sections, the discrete column approach was found to be more appropriate due to larger grout takes found to be needed for infilling due to the large void ratio. To further test the column approach, APC/SCS opted to expand the test program by drilling/grouting additional Plan "A" test sections at approximately 100 feet (30.4 m) spacing across the length of the east embankment bolster. It was felt that additional test sections would provide useful subsurface data across the embankment while simultaneously allowing the project team to further tailor materials, means, and methods that would enhance productivity during the production stage of the remediation program. In the expanded program, each test section consisted of two grout rows along the downstream shell of the embankment, twenty feet (6.1 m) apart, with four grout holes per row. Each hole was drilled with the project's Wassara down-the-hole water powered hammer (WDTH) mounted on a Cubex Drill rig. The hole spacing matched the spacing from the previous Plan "A" test section (Figure 5). The initial plan was to grout using the following mix, supplied by a local ready-mix plant, which was adjusted based on project experience (per cubic yard):

- 300 lbs. (136 kg) of cement
- 1800 lbs. (818 kg)of concrete sand
- 900 lbs. (409 kg) of fly-ash
- Enough water to yield a 3 to 4-inch (7.6 to 10.1 cm) slump

Similar grouting procedures, pressure and acceptance criteria were proposed as for the initial test section. The sequence of hole grouting used for the initial test section was planned to be used. With regard to instrumentation, an additional embankment piezometer (30 feet or 9.1 m deep) would be installed just upstream from each of the proposed new test sections. This piezometer was to be monitored closely during grouting in addition to nearby existing instrumentation. Other details of the monitoring for the initial Plan A would be maintained. The additional test sections for the expanded program were at Stations 53, 56, 57, and 58.

Following FERC and BOC review and concurrence with the expanded program, LMG grout column work proceeded in July of 2013. A meeting of the BOC, FERC and APC/SCS was convened in August, 2013 to review the progress and to conduct a special Potential Failure Modes Analysis (PFMA) of Potential Failure Modes (PFM) associated with the proposed remedial work (long term and construction conditions). It was reported that the average overall rock mass "voidage" encountered had been found to be around 26-30% in the 50 foot (15.2 m) thick treatment zone. It was noted that caprock on top of the weathered karst was as thin as 1 to 2 feet (0.3 to 0.6 m). It was taking about 34 minutes for one 8 cubic yards (6.1 cu m) truck to be injected (about 6-7 strokes/minute, each stroke being 0.67 cubic foot or 0.02 cu m). Extra grout was being pumped in the uppermost stage "mushroom" effect) just under the cap rock, and reportedly did not require "watering up" the grout to aid injection. The same effect was being employed at the lowermost stage of the treatment zone to create a "floor" for the column since the underlying rock was also heavily karstified. At the time of the August BOC meeting, about 25-30 holes had been injected for a total of 301 cubic yards (230 cu m) of LMG. Some modifications of the original plan had been made by the project crews including minor variations in mix slump depending on time of day (temperature) and the situation at the hole. Also, the inclination of upstream hole was changed from 15º to 20º, and the volume grouting termination criterion in most stages was increased slightly from 3.5 to 4 cubic yards (2.7 to 3 cu m). Also, the active pressure criterion was increased to about 200 to 250 psi (1,378 to 1,722 kPa). With regard to instrumentation response to the expanded program, a couple piezometers experienced slight rises during drilling; however, no pH hits or flow changes were noted.

With regard to the special PFMA, numerous construction and long term candidate PFMs were considered, and those most credible were "Piping along Weak Features in Bedrock", "Piping of Alluvium into Bedrock Discontinuities", and "Collapse of Thin Roof Rock under Toe and Bolster, Steady State Seepage Static Loading", all garnering Category I classifications (highlighted credible failure modes with significant consequences). Several risk reduction measures were developed, many of which were posed as recommendations of the BOC report for the August 2013 meeting. These included recommendations for a limited coring program in a previously grouted area to investigate the vertical continuity of the LMG columns, and to evaluate the lateral extent of grout travel. It was thought that this could be achieved by say 3 to 4 holes in and around the location of a typical grout hole. However, given the variability of the ground, it was noted that the field team needed to be realistic and pragmatic in the interpretation of the data from these exploratory holes. It was recommended that the LMG supplier should provide strength results on the LMG mix as delivered, for reference and for future analytical purposes. It was also suggested that the grouting sequence of holes be adjusted to drill from downstream and upstream in order to assure the most uniform treatment under the entire dam footprint. It was also suggested that adjacent holes in adjacent station rows be installed in a PST (primary, secondary, tertiary) sequence so as to help evaluation of grouting effectiveness. The project team was also asked to consider/evaluate upstream angle holes to place additional columns more directly under the crest of the embankment as well as to supplement an upstream curtain (E-Line) with an additional line of upstream holes.

Following the August 2013 meeting, work on the expanded test program continued and was essentially near completion and APC/SCS was awaiting approval of the production program by the time of the October, 2013 BOC meeting. At the October meeting, APC/SCS reported that 43 column locations had been grouted. A total of 614 cubic yards (469 cu m) of low mobility grout had been injected into the formation. Typically, the grout take at each column location was 14 to 16 cubic yards (10.7 to 12.2 cu m). Prior to beginning the treatment, limited data indicated the foundation conditions would be similar between the proposed treatment stations. Well into the treatment, that prediction had proven to be accurate with most drill holes encountering on the order of 30 percent voids in the upper fifty feet (15.2 m) of rock. APC/SCS indicated their intent to restart the E-line grouting upstream of the remediation once the remediation was completed. Project staff had completed two of the upstream angle holes to place additional columns more directly under the crest of the embankment, and intended to drill and grout more on completion of the bolster LMG column remediation. The plan for the angle holes was to get a representative sampling of holes across the 52+90 through 58+10 interval. Additional holes would be added adjacent to areas with larger takes.

During the October meeting, it was reported that compressive strength tests were performed on cylinders taken on September 17, 2013. The purpose of determining the strength of the grout was to compare the results to the compressive strength used in an estimate made by the Logan Martin Deep Grouting Project BOC in their October 22, 2013 BOC meeting report. For the estimate, the bolster was assumed an approximately 50 foot (15.2 m) thickness of unsaturated soil above the residual and alluvial soil. The top of rock (and thus top of columns) is about 90 feet (27.4 m) below the surface of the bolster. Based on depth and estimated total unit weight (130 pounds per cubic foot or 2082 kg/m^3 assumed), the vertical effective stress at the top of the columns is estimated to be about 9.2 ksf (0.44 MPa). The tributary area supported by each column is 20 feet (6.1 m) by 20 feet (6.1 m), or 400 square feet (37.9 sq. m). Based on this, it was estimated that that in a worst-case situation, each column could be called upon to carry up to about 3,680 kips (16,368 kN), dependent on the degree of erosion of the surrounding karst materials. The load carrying capacity of the columns, based on a target 5 foot diameter (about 20 square foot or 1.9 sq. m cross section) and assuming a 1,500 psi (216 ksf) grout unconfined compressive strength was estimated to be about 4,320 kips (19,214 kN) capacity. Therefore, the capacity of the columns is greater than the overburden weight which could be imposed if all the karst material supporting the tributary area were to erode around a column (e.g., a 20 foot or 6.1 m wide void feature). The 28 day break strength of the test cylinders was 1,154 psi (7.96 MPa). Based upon published charts, the strength gain between 28 days and 56 days is typically 15%. Therefore, the 56 day compressive strength was projected to be 1,327 psi (9.15 MPa). Using this lower strength yields a load carrying capacity for each column of 3,822 kips (17,000 kN) which is greater than the calculated worst case load of 3,680 kips (16,367 kN). Higher compressive strengths were recorded for the grout used to form the cap at the top of the columns; however, these values were not considered to be useful. In conclusion, the high content of the fly ash most likely delayed the strength gain in the grout. It was concluded that the final strength of the grout should be more than adequate to serve the intended purpose of the grout columns.

During the October, 2013 meeting, APC/SCS reported that it had drilled and sampled exploratory holes to investigate the lateral extent of the grout travel, the vertical continuity of the treatment, the interpretation of the top of rock and verification of the establishment of a grout cap at the interface of the top of rock and the alluvium. Initially, three holes were planned at two locations, one at the center of a column, one five feet (1.5 m) upstream and one five feet (1.5 m) downstream. At the first location, 52 + 90 Hole #4, no grout was found at the center core hole or the upstream hole. Grout was encountered in the downstream hole at the expected elevations. At the second location, 56+10#1, several holes were drilled with very little grout recovered. It became apparent that deviation was playing a role in the inability to select a location for a test hole and actually core through grout. With the use of the project team's optical televiewer (which included an accelerometer), deviation within the core holes was measured and ranged from four to eight feet (1.2 to 2.4 m). Rocks in the overburden, boulders, and pinnacles at the top of rock deflect the drill casings for both the coring program and the grouting program. Therefore, centering a test hole on the ground surface directly over the location of a column does not guarantee coring through grout. The first two locations selected for core drilling were based on the large quantities of low mobility grout injected in the holes. Refusal for most of the stages at these locations had been due to grout quantity limit rather than reaching the grout pressure criteria. The grouting pressures reached were only slightly above line pressure. The decision was made to core drill at a third location. This time a grout column location, 56+90 Hole #1, was chosen based upon high grout takes and higher injection pressures (200 psi to 250 psi). The test hole was offset two feet east of the center of the column location. A very good grout cap at the top of rock was found as well as other grout in the rock where it should be in accordance to the grout records. APC/SCS indicated that the attempt to core samples of the grouted columns proved very beneficial. As a result of the findings during the coring operation, it was determined that grout pressures in excess of line pressure must be reached or the grout column may not be constructed as planned. It was decided to increase the grout limit to at least 5 cubic yards (3.8 cu m) and occasionally higher depending on the driller's description of the cavities. The exploratory borings also indicated that the interpretation of the top of rock drilling with the Cubex versus core drilling is not the same. The weight and size of the rods, the loss of water while drilling and the power of the Cubex drill make it difficult to pinpoint precisely the interface of the rock and the alluvium. Based upon the core logs, the interface is 8 to 10 feet (2.4 to 3 m) higher than indicated on the Cubex driller's logs. Therefore, APC/SCS proposed to add two more stages above what would have been logged as the top of rock. It was agreed by the meeting participants that this change would improve the chances of forming a grout cap at the intended interface. Subsequent work included remobilization to each of the completed columns to add these two upper stages.

The cores from the exploratory holes were observed by the BOC. The grout could clearly be seen to fill the extensive void areas, and a significant amount of intact rock core was retrieved (see Figure 7). A couple of intervals of apparent void (one three feet or 0.9 m thick) were encountered. The BOC believed these "voids" were likely filled with soil material that was washed out during the core drilling process. If these were truly empty voids, the grouting would have been expected to have filled them. The reduction in column strength represented by such a void is believed to be made up for by the higher strength of rock making up the walls surrounding or adjacent to the void. Based on the information presented at the meeting, the BOC concluded that discrete columns of LMG/rock are being built in the ground, since there are no systematic reduction ratios between the successive phases of holes, and there is no evidence from the piezometers that this extremely permeable horizon is being systematically sealed.

Another recommendation to improve the bolster LMG column program was the addition of an angle hole (16°) adjacent to the present upstream 20 degree hole to the proposed production phase column installation. In addition, as can be seen in the test section cross section of grout holes (Figure 5), there was a 45 to 50 foot (13.7 to 15.2 m) gap between the bottom elevations of Hole #2 and Hole #4. Consequently, a 10 degree angle hole between #2 and #4 was proposed (denoted #5) for the production phase of column installation. Figure 8 shows the cross section of column lines proposed for the production phase work, discussed in the next section. These additional holes were intended to better protect the crest and downstream shell of the dam from development of sinkholes large enough to threaten stability.

Figure 7. Photo of Rock Core and Grout Encountered in Drilling through Column Locations

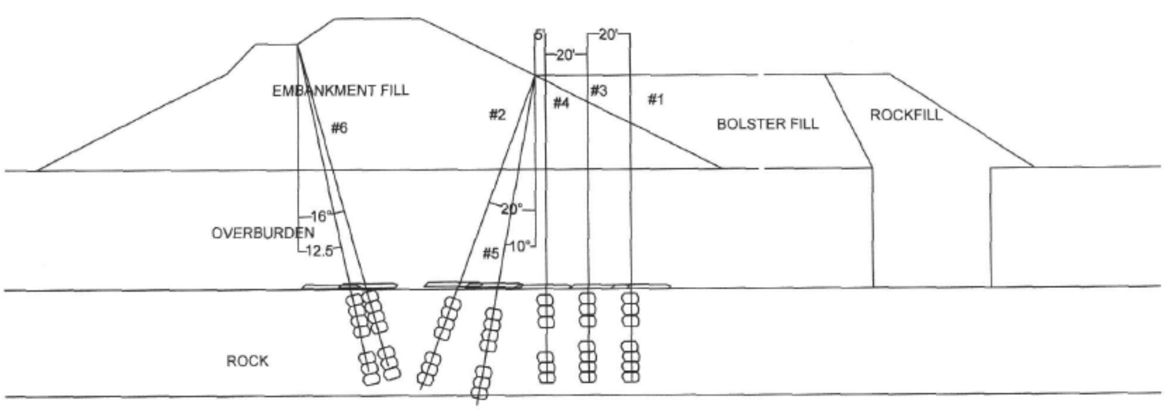

Figure 8. Final Configuration of LMG Column Layout at Each Station (not to scale)

Following receipt of the supplemental PFMA report, a report summarizing the extended test program, and a plan for the production phase of work, approval was given by the FERC to complete the remediation on October 30, 2013.

IV. PRODUCTION PHASE DRILLING AND GROUTING FROM THE BOLSTER

A. Drilling and Grouting Details of the Production Phase

Following completion of the test sections, grout row stations along the top of the bolster were staked every 20 feet (6.1

m) beginning at station 52+70 and proceeding west. The hole spacing of 20 feet (6.1 m) was based upon a review of the fracture patterns of the rock shown in construction photos of the old river bed. There were seven grout holes at each station along the dam (Figure 7). Each hole was drilled to a depth of approximately 150 feet (45.6 m) with the WDTH mounted Cubex drill rig. Following completion of the drill hole, approximately 150 feet (45.6) of NW casing was set to within five feet (1.5 m) of the bottom of the hole. The grout holes were then grouted using LMG supplied by a local ready mix company. The hole closest to the downstream slope of the dam (Hole #2) was initially drilled at a 15 degree angle upstream, but was increased to 20 degrees to improve the column pattern. Two rows of holes were grouted along the upstream work berm between stations 52+70 and 60+00. The grout holes in the first row were designated as "#6 holes" (Figure 7) and drilled on a 16 degree angle in the downstream direction. The second row of holes was drilled at 12.5 degree angles split spaced between the #6 holes. These holes were also drilled 50 feet (15.2 m) into rock and grouted with a slightly higher slump LMG. If the total grout take per hole was greater than 4 cubic yards (3.1 cu m), additional closure holes were drilled at a spacing of 6.7 feet (2 m). The purpose of these holes was to provide column support underneath the crest of the dam and to complete the pattern initiated from the downstream bolster.

For the production phase holes through the downstream slope, the 50 feet (15.2 m) of rock was grouted in 5-foot (1.5 m) stages. Each five foot (1.5 m) stage was grouted until an effective excess pressure (gage pressure minus line pressure) between 150 and 250 psi (1,033 to 1,722 kPa)was reached or 4 cubic yards (3.1 cu m) of grout were injected, whichever came first. The target objective was to form approximately five feet (1.5 m) diameter columns. A Schwing BP450 concrete pump was used to inject the grout at a maximum rate of 5 cubic feet (0.14 cu m) per minute through a three inch (7.6 cm) diameter grout hose. 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 "mushroom" cap along the top of rock wider than the grout columns. As for the sequence of grouting per station at the downstream slope, the downstream hole was grouted first followed by the upstream hole. Following the completion of the downstream hole and the angle hole, the middle grout holes were drilled and grouted.

The production work proceeded without significant incident, since most of the bugs were worked out during the test section phase of work. At the completion of the work, 263 column locations were grouted. Prior to grouting, each hole was evaluated based upon available data such as the location and number of cavities (whether or not the cavities were open or mud filled) void ratio, and number of stages. A total of 2,156 cubic yards (1,648 cu m) of low mobility grout has been injected into the formation. The average grout take at each column location was 8.2 cubic yards (6.3 cu m). A total of 36 stations were treated along the downstream bolster. Each station was spaced on twenty foot (6.1 m) centers spanning from Station 52+70 to Station 60+00. In general, each station consisted of 5 holes; 2 angle holes and 3 vertical holes. The number of holes per station varied inside the batch plant due to existing structures. There were a total of 185 holes treated downstream with a total of 1,791 cubic yards (1,369 cu m) of grout.

In addition, there were 78 grout holes drilled from the upstream work berm. As noted above, these holes were angled downstream 16 and 12.5 degrees. Few split spacing holes were needed since grout takes were small. The total amount of grout injected along the upstream work berm was 365 cubic yards (279 cu m), significantly less than the downstream side of the crest. The most likely reason for the reduction was the influence of the previous grouting along a previously installed line of grout holes, referred to as the E-line. A complete layout of the LMG columns is shown on Figure 8.

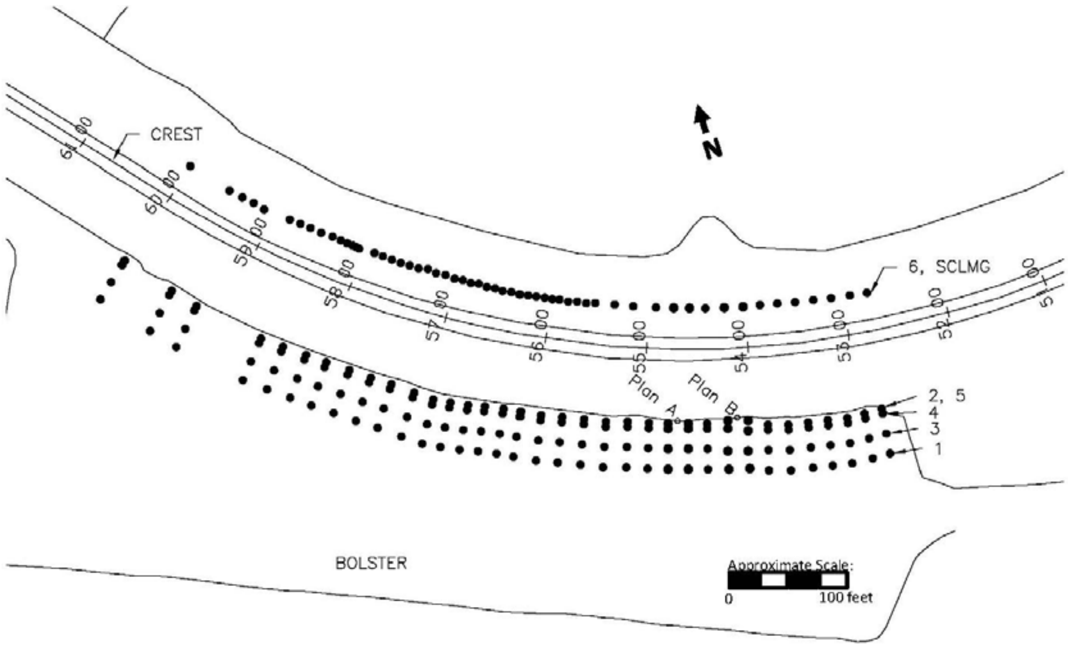

Figure 9. Final Plan of LMG Column Locations at Ground Surface as Constructed (Black Dots)

B. Instrument Monitoring during Production Grouting

The existing instrumentation monitored in the area of the East Dike Bolster during the production phase of the LMG Column construction included:

- 10 deformation monuments, located along the downstream crest of the dike at the even 100-foot (30.4 m) stations from station 52+00 to station 61+00.
- 7 lateral toe drains monitored for flow, clarity and pH.
- Laser survey equipment is arranged on the slope of the embankment or on the bolster immediately upstream of active grouting locations to monitor for heave.
- 65 open-stand-pipe piezometers and wells.

Six new embankment piezometers were installed during the two test phases, which along with two existing embankment piezometers provided the approximate 100 foot (30.4) spacing needed for the test sections. These piezometers were founded in the downstream shell of the embankment, 5 to 10 feet (1.5 to 3 m) upstream and in-between each test section's two rows of grout holes. With the most immediate concern being an increase in the water level and corresponding pressure in the embankment, these downstream shell piezometers were monitored continuously during drilling and grouting by suspending an activated water level probe just above the existing water table. Any rise in water level would then be audibly alarmed.

Other piezometers and wells within an approximate area of 150 feet (45.6 m) around the hole being drilled and grouted were monitored frequently.

Water elevations were influenced by drilling and grouting in all top-of-rock and into-rock piezometers relatively close to the grout holes. In many cases there was a rapid rise in water levels recorded by nearby piezomenters, which returned to normal following cessation of drilling or pumping grout with the exception of 108, 119A, 120A, 312, 235, 314 and 329. These 7 piezometers were lowered by 1 to 3 feet (0.3 to 0.9 m), at least temporarily. Piezometers 121, 122 and 239 appear to have been raised slightly, but continue to show tailrace pressure influences and therefore the ability to drain. Piezometers 245 and 331 were inadvertently grouted up during the grouting process . No temperature anomalies have been detected in the piezometers in the area of the east dike or bolster.

There was no change in the pH in the lateral toe drains as a result of grouting the sections. Three of the 8 lateral toe drains reduced from 0.25 to 0.50 gpm (0.95 to 1.9 lpm). Two other drains show zero flow at times. All drains have remained clear, and no grout shows were detected during the work.

During the execution of the work, deformation had been minimal along the crest with a slight increase in settlement in the area of the most concentrated drilling and grouting. This effect has been seen in other active grouting areas in the past at the project and the settlement rates generally were thought to have returned to normal following the intense activity.

However, following the completion of the work, the time rates of settlement did not return to more normal levels and at this writing are being monitored closely. There were no occurrences of heave measured during any grouting sequences.

C. Final Project Statistics

Prior to beginning the treatment, limited data indicated the average void ratio was 30 percent. Void ratio was calculated by dividing the sum of the cavities (mud filled or open) by the linear feet of rock to be treated. The initial prediction proved to be fairly accurate with the final average void ratio being about 26 percent within the upper fifty feet (15.2 m) of rock. Also of interest, when the voids were filled with a fine quartz sand residual remnant of weathered carbonate rock (referred to as tripoli), the stages did not take large quantities of grout. The unique framework/relic structure of tripoli could be easily penetrated and disturbed during the drilling action. However, the infilling material could not be displaced with grout even at high pressures.

Grout unconfined strength values found from test cylinders since the test section phase of work were found to be significantly higher than those measured on the same mix in September 2013, being measured over 4,000 psi (27.6 MPa) at 28 days. It was theorized that the high content of the fly ash most likely delayed the strength gain in the grout.

Following the completion of the LMG grouting, four additional test core holes were drilled. These test holes were located at station 55+90 near Hole #3, station 56+90 near Hole #1, station 56+90 near Hole #3 and at a location split spaced between 56+90 Hole #1 and 56+90 Hole #3. Grout was found in cavities that would have been open or filled with loose material. If the in-filling consisted of a denser material, no grout was found. Also, if it was determined that the core hole had deviated no grout was retrieved in the core. As for the split spaced test hole, grout was found to exist near the top of rock; therefore, an effective cap seems to have been constructed. Since very little grout was found outside the theoretical diameter of a column, the low mobility grout had not spread far outside the intended zones. In accordance with the intent of the design, the findings of the test holes confirmed an impervious barrier downstream of the crest of the dam was not developed. If minor seepage does exist through the upstream grout curtain or at the soil and rock interface, it still can pass safely to the downstream trench drain. In other words, the construction of the individual columns did not cause any adverse change in flow or pore pressures. This is confirmed by the instrumentation results discussed in Section IV-B above.

V. CONCLUSION

This paper has presented the development and implementation of what is believed to be a unique remedial approach intended to enhance the integrity of an existing dam founded on a stratum of karstic limestone. This stratum, underlying the crest and downstream slope of the dam, was assessed to pose a potential sinkhole collapse threat to the dam. This fifty foot (15.2 m) thick stratum of high void content limestone was reinforced using rows of vertical and angled nominal 5-foot (1.5 m) diameter "columns" roughly formed using low mobility grout to minimize the size and effect of voids and collapse holes between the columns that might develop. A challenge was installing the columns so that drainage through the formation, which is downstream of an existing extensive grouted cutoff system, could continue unencumbered. The approach to implementing the remediation included significant collaboration and discussion by a Board of Consultants, the owner's project team, and the Federal Energy Regulatory Commission. It drew on Alabama Power Company's extensive 45 years of experience in grouting at the site, and utilized state of the art equipment and controls. The development of the remedial approach involved testing and revising the remedial concept using field test sections and exploratory drilling to assess the integrity of installed test columns; and incorporating the lessons learned into the full implementation of production drilling and grouting. Extensive instrumentation and monitoring were part of the effort to assure that the drilling and grouting were not having an adverse effect on the structure and that an in-filled block situation did not develop that could increase the pore pressures in the embankment and foundation that could potentially lead to instability. The project has been successfully implemented, and it is believed that the remediation has significantly mitigated the risk of sinkhole collapse at the treated location of the dam.

VI. ACKNOWLEDGMENT

The authors gratefully acknowledge the contributions and collaboration of the entire Logan Martin Project Team and representatives of the Federal Energy Regulatory Commission. Consultants to APC/SCS including Mr. David Holland and Dr. Jim Redwine made valuable contributions to the success of the completed project work. This has been and continues to be a group process where the outcomes are always improved by adopting the cream that rises to the surface through presentation, debate, discussion, collaboration, and taking the advantage of the experience that precedes us.

"By three methods we may learn wisdom: First, by reflection, which is noblest; second, by imitation, which is easiest; and third by experience, which is the bitterest." — Confucius

VII. AUTHOR BIOGRAPHIES

R. Craig Findlay, P.E., Ph.D., G.E. - Dam Safety Engineer/Geotechnical Engineer, President, Findlay Engineering, 70 Old Field Road, Yarmouth Maine, 04096. Dr. Findlay's 38 years in the dam safety, water resources and geotechnical engineering profession includes a broad variety of consulting and project engineering experience, more than 30 years of which have included involvement with dams and hydroelectric projects. He formed Findlay Engineering as his engineering consultancy in 1998, previously working as a consulting engineer for E. C. Jordan Company, ABB Environmental Services, Northrop, Devine & Tarbell, and Duke Engineering & Services.

Brian H. Greene, Ph.D., P.G. - Engineering Geology Consultant, 1207 Colonial Place, Sewickley, PA 15143. After a 32 year career with the U.S. Army Corps of Engineers, in 2010, he began consulting and currently works part-time for Gannett Fleming, Inc. out of the firm's Pittsburgh, Pennsylvania office. Dr. Greene's career has been focused on dams and he has authored and co-authored over 50 professional papers and presentations on the topic of dam remediation, dam foundations, and forensic investigations of dam failures. He presently serves as chairperson of AEG's Dams Technical Working Group and has taught college courses including Geology for Engineers and Hydrogeology.

Donald A. Bruce, Ph.D., D.GE, C.Eng., L.G., L.E.G President, GEOSYSTEMS, L.P., 161 Bittersweet Circle, Venetia, PA 15367. Dr. Bruce is a leading independent expert in geotechnical construction, with a focus on drilling and grouting techniques, anchors, micropiles, deep mixing, cutoffs, and in-situ reinforcement. Dr. Bruce's career spans over 40 years having worked on all 5 continents. He is the author of over 300 technical papers and articles, and has authored and edited several books on ground engineering techniques including the new Specialty Construction Techniques for Dam and Levee Remediation.

Bobby E. Williams, P.E. Civil Engineer, Southern Company, 600 North 18th Street, Birmingham, Alabama 35203. - Manager of Hydro Construction within Southern Company Generation. He has 40 years of experience in the design, construction, and grouting of dams. He oversees the remedial operations at Logan Martin Dam.

John H. Williams, Senior Power Generation Specialist Southern Company, 600 North 18th Street, Birmingham, Alabama 35203 – Senior power generation specialist. In his 38 year career with the Southern Company, 34 of those years have been in SCS-Hydro Services Dam Safety and dedicated to the surveillance and instrumentation of dams. Instrumentation installation, remediation and interpretation have been the focus of his work.

Mickwee, R. L., P.E. Civil Engineer, Southern Company, 600 North 18th Street, Birmingham, Alabama 35203 – Dam Safety & Surveillance Supervisor. He has 18 years of experience in geotechnical engineering and dam safety. He is the chief dam safety engineer for Alabama Power Company's fourteen hydroelectric projects.