THE BEAR CREEK DAM, ALABAMA

Keith A. Ferguson, P.E.[1]
Donald A. Bruce, PhD, C.Eng.[2]

ABSTRACT

In July of 2007, the Tennessee Valley Authority (TVA) selected the authors to serve on a two member senior review board (Board) for the investigation, design and construction of the rehabilitation of the existing Bear Creek Dam, Alabama. This Board was empowered to make original and fundamental contributions to the project in real time. The Engineer of Record for TVA was P.C. Rizzo and Associates. This embankment dam was built on a karstic limestone foundation. Since construction, up to 1500 gpm of seepage had developed through the dam's foundation during flood pool storage levels. Previous efforts to reduce and control seepage had been unsuccessful.

The paper provides an overview of the safety issues with the existing dam, and the strategies developed for 1) characterization of the karst foundation materials, 2) construction risk management, 3) the configuration and details of the foundation excavation and treatment program that was completed including a multi-line grout curtain and discrete karst feature cutoff panels. In addition, the paper discusses the stability evaluation of the dam, development of the dam cross-section, mix design, and seepage control details of the new replacement Roller Compacted Concrete (RCC) berm. This was constructed immediately downstream of the existing dam while the embankment was maintained in service.

BACKGROUND

Bear Creek Dam is the lower of two dams on Bear Creek, a tributary to the Tennessee River in northwest Alabama. The dam impounds a multi-purpose reservoir for flood control, water supply, and recreation. The dam has a homogeneous embankment cross section (Figure 1). It has a total crest length of 1,385 feet, a maximum structural height of about 85 feet from the dam crest to the bottom of the key trench, and a maximum hydraulic design height of about 68 feet at the maximum section. The dam crest is at elevation 618, the normal summer pool is held at elevation 576, and the maximum design water surface elevation during flooding is elevation 613. The maximum design water storage in the reservoir during a flood is over 40,000 acre-feet.

Planning, design and construction of the dam were completed in the 1960's and the reservoir was first filled in 1969. Appurtenances include a primary 9-foot-diameter tunnel outlet works in the lower right abutment, and an uncontrolled ogee crest emergency spillway in the upper left abutment (Figure 2).

---

[1]Senior Dam Safety and Design Consultant, 10135 Summit View Pointe, Highlands Ranch, CO 80126, kferguson10135@aol.com
[2]President, Geosystems, L.P., P.O. Box 237, Venetia, PA. dabruce@geosystemsbruce.com

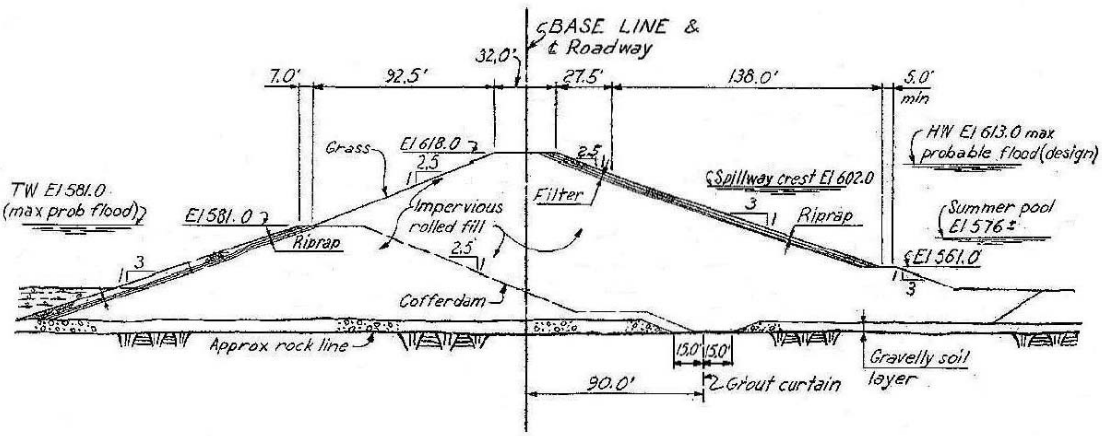

Figure 1. General Cross-section of Bear Creek Dam

Figure 2. Photograph of Bear Creek Dam, Spillway and Outlet Works

The dam was originally constructed with an incomplete single line grout curtain installed from the bottom of a key trench excavated under the upstream shell of the dam. The key trench and grout curtain were omitted in the upper left abutment beginning about 300 feet right of the spillway structure. No foundation grouting or treatment was performed under the spillway structure. During key trench excavation, numerous solution features and voids were encountered: some portions of the soft materials filling these karst features and the highly weathered bedrock were removed and replaced with embankment materials. No other foundation treatment such as dental concrete was conducted. Large grout takes were common during the grouting program.

Upon first filling in 1969, seepage was observed along the downstream toe of the embankment. The first of two remedial grouting programs was completed in 1972. During this program, the left abutment was treated to close the original curtain and to try and reduce seepage flows and allay safety concerns. During a high water event in December 2004, a number of boils, small sinkholes, and increased seepage flows of between 1200 and 1500 gpm were observed along the toe of the dam. Subsequently, TVA completed an exploratory drilling program including coring of the foundation bedrock, and installation of piezometers. Cone penetration testing was also completed in the embankment and foundation soils under the dam to determine if any damage had occurred as a result of the increased foundation seepage. TVA determined that the primary source of the seepage was via the karstic formations in the left abutment foundation, and a second remedial grouting program was initiated in 2004 and 2005 along the upstream slope of the dam. However, grouting operations were interrupted by a flood and the grout curtain could not be economically brought to closure.

The 1969 and 1972 grouting programs employed traditional grouting concepts, means, methods and materials which would not be judged acceptable or appropriate today in such remedial applications in karstic terrains. In general, however, the 1972 grouting program did achieve some short term reduction in seepage rates. The benefit of the 2004-2005 program was not demonstrated through an elevated reservoir storage period and so the potential vulnerability of the foundation remained an issue. Subsequently, TVA determined that a major rehabilitation program was required to solve the foundation seepage concern with a permanent solution, to address a hydrologic deficiency based on updated hydrologic studies of the Bear Creek basin, and to provide an appropriate overall level of safety for this dam. Specifically, a new Roller Compacted Concrete structure would be constructed immediately downstream of the existing dam. The configuration of the new dam would retain the functionality of the existing outlet and spillway structures.

It is important to note that the design and construction period of the dam predates the most recent significant seepage related problems at dams such as Wolf Creek (KY), Center Hill (TN), and Clearwater Dam (MO). The design of Bear Creek Dam has many similar features as these older dams. Consequently, many of the lessons that have subsequently been learned related to foundation treatment issues and requirements for embankment dams on karstic limestone foundation were not incorporated into its design and construction. Similarly, important lessons learned related to remedial grouting of karst foundations beneath embankment dams were not incorporated into the remedial grouting programs that were undertaken.

FOUNDATION CHARACTERIZATION AND DEVELOPMENT OF GEOLOGIC MODEL FOR DESIGN PURPOSES

The dam site is located at the contact of the Cumberland Plateau and Fall Line Hills of the Coastal Plain Physiographic Province of Alabama. Portions of the bedrock materials in this region are known to contain significant karstic developments. Bedrock at the dam and reservoir site is relatively flat-lying sandstones, limestones, mudstones and shales of the Parkwood and Bangor Formations. Under the proposed new dam alignment, only the Bangor Formation is present and in the near-surface bedrock profile consists of the upper Bangor Limestone (a cherty crystalline limestone and fossiliferous packstone) over the relatively thin Bangor Shale unit and then the lower Bangor Limestone (a fine grained oolitic packstone). Karstic features observed in exposed outcrops or in the construction records in the region around and under the existing dam suggested both structural (i.e., along joints and shears), and stratigraphic (i.e., within specific subunits of the Bangor) development and control mechanisms.

Upon review of the original site exploration information/construction information and the results of the initial phase of site explorations being performed for final design, the Board recommended that a three dimensional geologic model of the site be developed using all available information from the original design investigations, construction documentation, supplemental exploration and remedial grouting programs, and the final design level investigation program. The Board believed that such a model and approach to site characterization would provide the best chance to 1) adequately define the extent and characteristics of the karst, including the possibility of karst extending to, through, and below the Bangor shale unit (2) identify and estimate foundation treatment requirements and construction risks (both grouting and cutoff wall requirements), and 3) identify the potential for, and impact of, weak partings in the shale, and the anticipated shear strength of those partings on the dam cross-section design. Karst extending below the Bangor shale unit would significantly impact the requirements and cost of foundation treatment. Weak partings in the shale could significantly affect the feasibility and design details of a Roller Compacted Concrete (RCC) configuration for the new dam.

The final design level exploration program consisted of:

- Twenty-one vertical and inclined borings with continuous rock core sampling of the bedrock (a total of 26 borings were ultimately completed).
- Multiple geophysical techniques including micro-gravity, SASW (spectral analysis of surface waves), seismic refraction, and limited down-hole seismic surveys
- Laboratory testing of foundation soils and bedrock

A summary of the primary rock structure (stratigraphy and joints) at the site is shown on Figure 3. A summary of all of the notable voids and tools drops during various field exploration programs is shown on Figure 4. Considering only the information obtained for this site, close examination of the various families of field exploration data suggested some notable differences between the possible characteristics of the karst. For example, the joint orientation data shown on Figure 3, compared to the seismic refraction and micro-gravity data, do not give a consistent indication of the number, size, orientation and depth of the major karsts anticipated in the foundation. The seismic refraction data provided a general indication of the quality of the rock for assisting in establishing the foundation excavation objective but did not reveal any specific large karst feature locations, only some indicators of possible locations. The micro-gravity results suggested several large karst features including some features under the existing spillway,

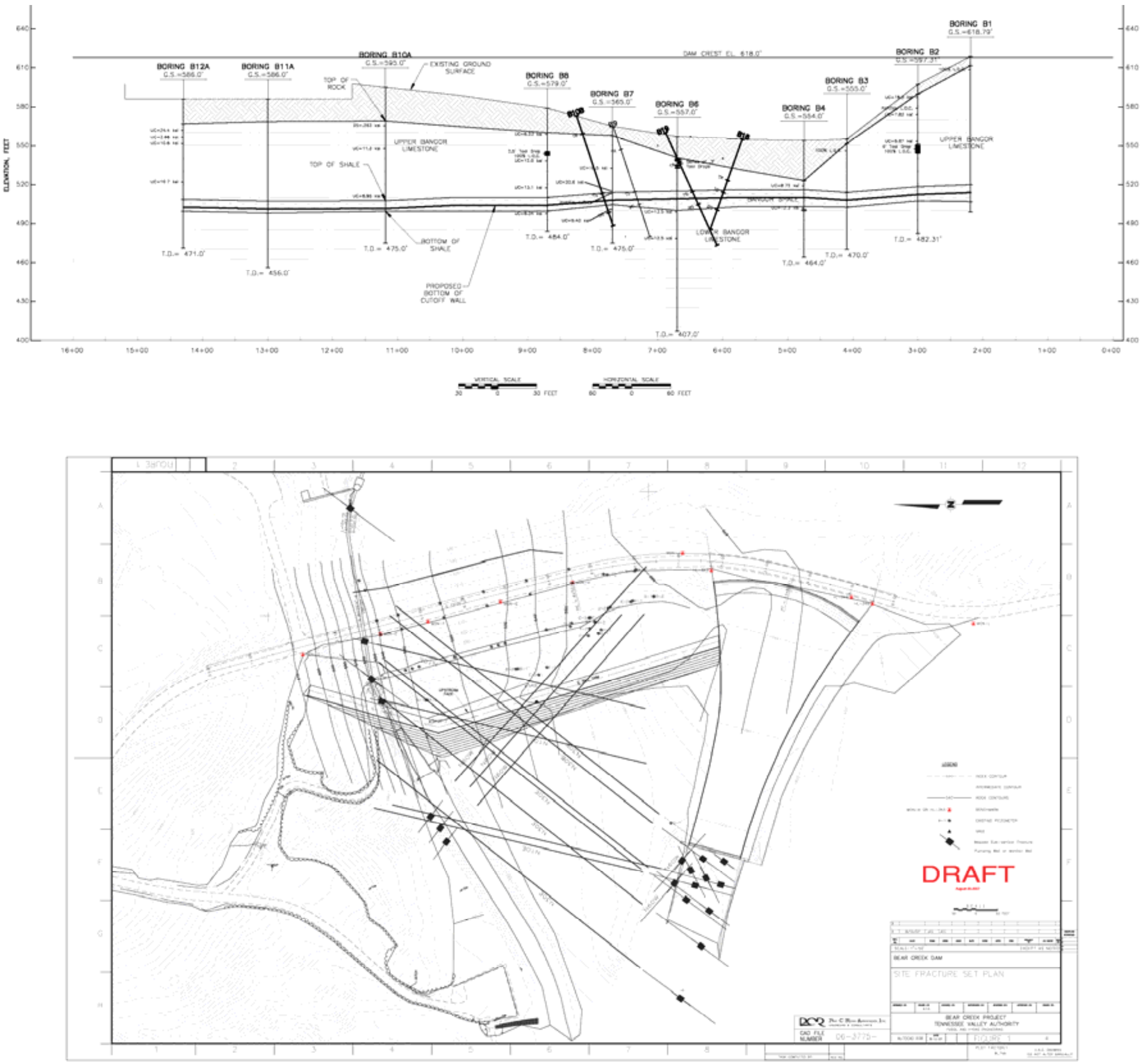

Figure 3. Bedrock Structure

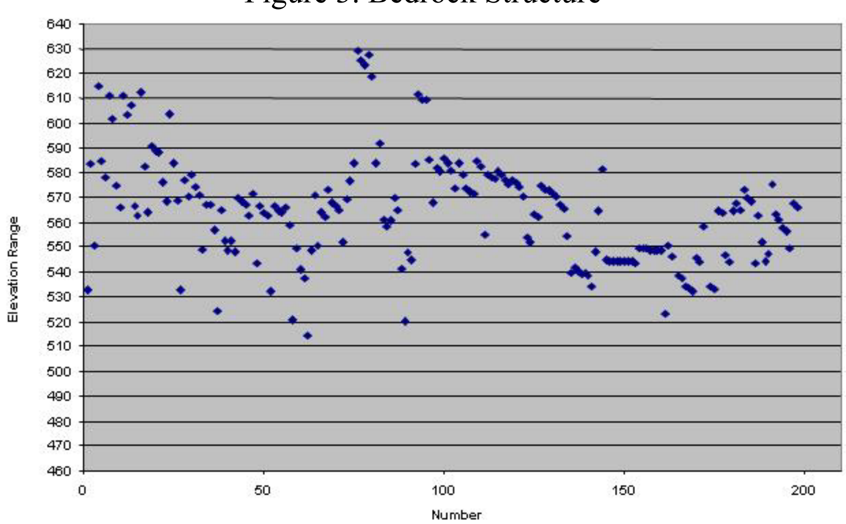

Figure 4. Elevation of Voids and Tool Drops
(Number indicates sequence of encounter)

but the orientation of these features was not consistent with the orientation of any of the major joint sets shown on Figure 3. The seismic refraction along with the void and tool drop information on Figure 4 provided a hint of some potential stratigraphic controlled karst development. However, there were clear indications that karst development in the maximum section area had the potential to extend down to and through the Bangor Shale layer and possibly into the lower Bangor Formation. Hence there was uncertainty and risk in defining the amount of karst and corresponding treatment requirements and the time required to effectively treat the conditions.

Based on the geologic model results as well as an understanding of karst formation mechanisms, karst characteristics and dam performance at other notable sites, the project parties concluded:

- Karst development appeared to be predominantly within thirty feet of the anticipated rock surface.
- Karst features had developed along sub-vertical fracture sets (structural) and possibly along sub-horizontal fracture sets associated with bedding planes (stratigraphic).
- Karst development may have penetrated Bangor Shale in the maximum dam section area
- Karst appeared to be in the form of discrete features separated by unweathered rock and, if within the upper thirty feet of the rock, would therefore be treatable with individual excavation panels: a continuous, full length secant pile wall as originally proposed would not be required, resulting in significant cost savings to the project.
- Sliding on the Bangor Shale was a major design consideration.

Following this exploration and a preliminary design process, the following rehabilitation scheme was selected as the optimal remediation as detailed by Ginther et al. (2009):

- To eliminate the dangerous seepage flows through the foundation, a “composite” seepage barrier (Bruce et al. 2009) consisting of a two line grout curtain and localized “positive” cutoff panels was selected. Cutoff panel locations and depths were selected based on the results of the foundation preparation and drilling and grouting activities. In addition, during the foundation exposure phase, a large solution feature was confirmed, crossing the sluiceway tunnel. This required an additional grouting program to be performed approximately perpendicular to the two line grout curtain to ensure the longitudinal integrity of the seepage barrier.
- To prevent loss of the dam due to overtopping of the embankment during the PMF, a downstream roller compacted concrete (RCC) reinforcement or berm would be constructed.

In addition, it was recommended that TVA proceed with separate construction packages for foundation excavation/treatment, and for the dam construction itself. This would provide a significant risk management advantage related to the size and treatment requirement of karst features that were expected to be found at the bedrock surface.

Furthermore, excavation and surface preparation were within the capability of TVA's own forces (Heavy Engineering Division), while specialty subcontractors would be procured for dewatering, grouting, and installation of braced excavation systems. The Board also strongly recommended that the excavation for the RCC structure be opened as quickly as possible so as to provide early warning of the actual conditions and to afford maximum flexibility in response time and methods.

FOUNDATION EXPOSURE AND TREATMENT

Rock Excavation, Cleaning of Karstic Features, and Surface Treatment

Competent rock head for the RCC structure was defined as partially weathered rock (USBR, 2001), a rock mass rating of "good" (60 or higher) and systematic treatment of major karstic features in accordance with industry "rule of thumb" (i.e., cleaning and backfilling to depths of three times the feature width; or 30% width plus 5 feet for features greater than 2 feet in width). In practice, excavation, washing and cleaning, and dental concrete backfill of features extended much further, to the maximum effective width of the equipment available: about 20 feet below ground surface (Figure 5). The rock surface was shaped to remove overhangs and surfaces.

This process also allowed a very detailed surface geological map to be prepared by the Engineer of all features and discontinuities present across the new dam's foundation. In particular, two steeply dipping regional fracture sets, striking roughly N 30º E and N 55º W, were observed to have acted as zones of especially developed karstification (Figure 6). Also a packstone layer bounded above and below by cherty limestone illustrated stratigraphically controlled karstification (Figure 7). In all, a total of 6 major karstic features were exposed and treated during the three-phase excavation program which progressed across the site from left to right. A dewatering system was required to deal with inflows into the excavation from the reservoir, particularly from the more open N 55º W joint set.

The surface was meticulously cleaned by handwork, water and/or air jetting to remove all loose and weathered material before placement of 3000 psi dental concrete to regularize the interface. This dental concrete was placed in approximately 1-foot lifts using a pump truck, wet cured, and isolated from traffic for at least 48 hours after placement. During the subsequent rock drilling program, water pressure testing of core holes through the dental concrete and visual inspection of the cores confirmed the integrity and cleanliness of the concrete/rock contacts.

According to Ginther et al. (2009), excavation involved the removal of about 40,000 cyds of residuum, 25,000 cyds of alluvium, 6,000 cyds of fill and 10,000 cyds of moderately to intensely weathered rock. A total of 6,700 cyds of dental concrete were placed to fill irregularities to provide a working platform for the drilling and to provide a surface conducive to RCC placement.

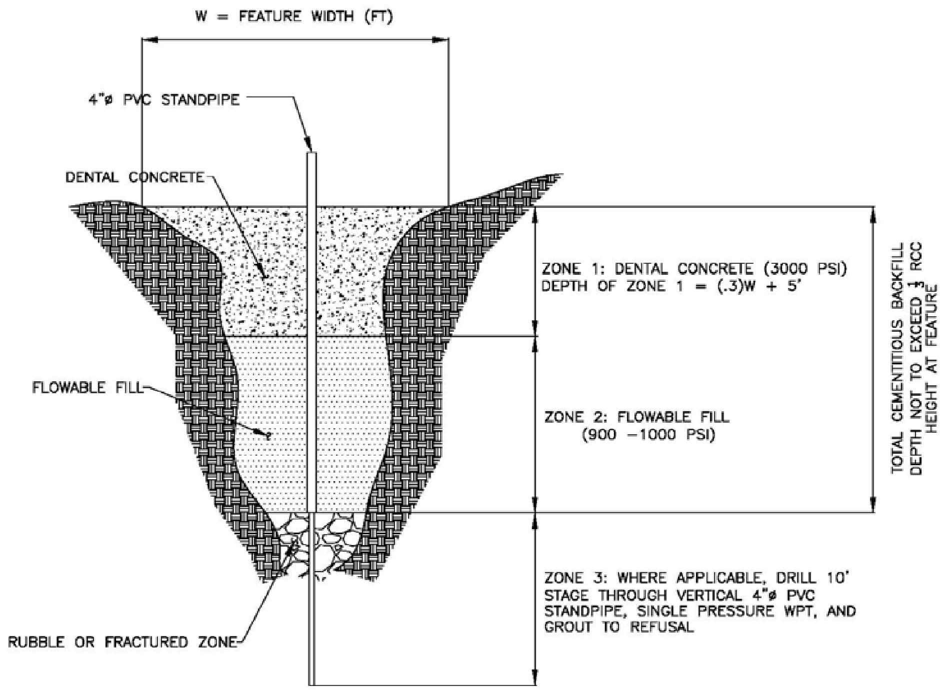

Figure 5. Karst Feature Treatment Detail

Figure 6. Excavation of Karst Feature along N30E Joint

Figure 7. Example of Stratigraphic Karst Feature near Base of Packstone Subunit in Upper Bangor Formation

Drilling and Grouting

As befitted its pivotal role as a component in the "composite wall" concept, the drilling and grouting had three principal goals:

1. to seal "groutable" (i.e., relatively clean and open) fractures and features;
2. to act as an exploration and design tool to determine the extent of the "positive" wall elements needed to reliably cutoff the deep, clay-filled, karstic features; and
3. to act as a pretreatment o these features to thereby facilitate cutoff panel construction.

Data gained from the preliminary site investigation, and obtained during the rock excavation activities, were used to "customize" the drilling and grouting design and specifications. Details of this program are provided by Ginther et al. (2009) and Kitko (2009). Highlights follow:

- Thirty-four exploratory HQ core holes were first installed at 80-foot centers on both sides of the two-row curtain (Figure 8). Core was logged, and the holes geophysically and optically surveyed before being subjected to multi-step water pressure testing. This process "baselined" precisely the rock conditions under the heal of the new RCC structure.
- The holes in each of the two rows were inclined at 15º off vertical, the holes in each row being inclined in different directions. The rows were 10 feet apart.
- The holes were drilled rotary percussion with water flush. A Drilling Parameter Recorder was used on each rig to record the "drillability" characteristics of the rock and in particular the presence of voids and zones of lost flush.
- The curtain was installed in a Primary-Secondary-Tertiary sequence, with the downstream row in advance. The Primary-Secondary spacing in each row was 20 feet.
- Primary holes were all extended several feet into the Bangor shale: the depths of higher order holes were determined based on a review of all previous drilling, water testing and grouting data in the vicinity.
- Computer-controlled real-time data monitoring and control of all water pressure testing and grouting operations was conducted. Summary charts were updated (Figure 9) and reviewed on a daily basis to optimize tactical decisions.
- Most of the work was conducted with a suite of stable, multicomponent, High Mobility Grouts (HMG's). In addition, Low Mobility Grout (LMG) was used to fill larger voids and/or zones of flowing water.
- The curtain was brought to closure at a maximum of 5 Lugeons, as measured by verification holes drilled and tested at regular intervals between the two rows.
- A special grouting program was implemented to treat successfully the large solution feature running under the sluiceway tunnel.

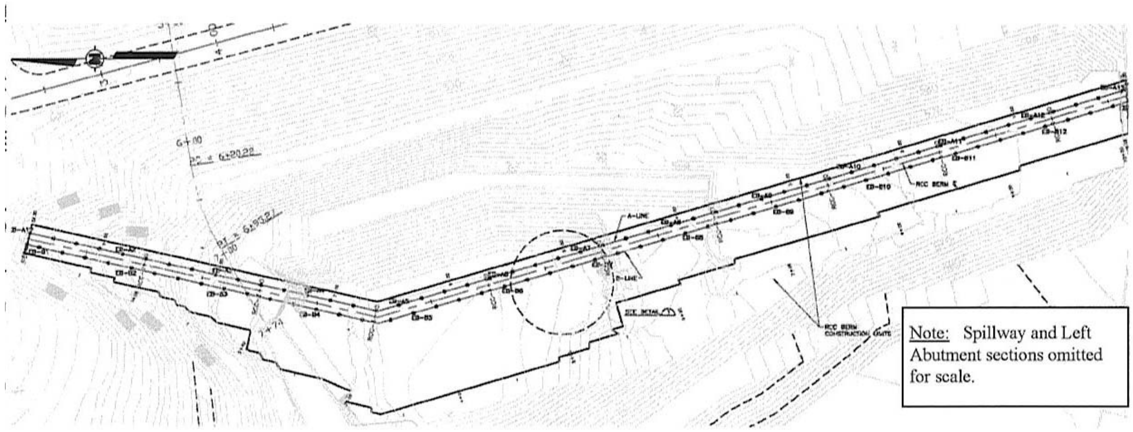

Figure 8. Layout of Curtain and Exploratory Holes (Ginther et al. 2009).

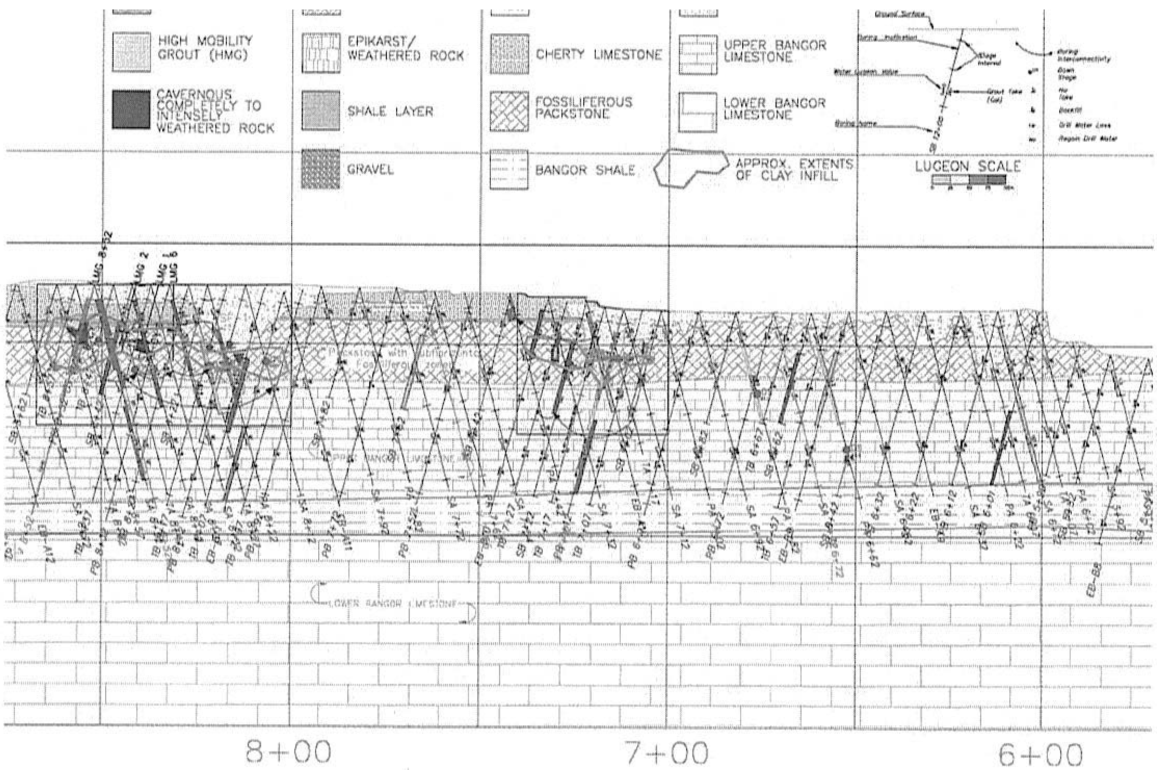

Figure 9. Typical drilling and grouting Record Chart (Ginther et al. 2009)

Concrete Cutoff Panels

By comparing and analyzing the entirety of the geological information provided by the initial investigation, the excavation, and the drilling and grouting program, it became clear that "positive," concrete cutoff panels were needed at 4 locations along the centerline of the curtain (Table 1). These locations and extents were further confirmed by additional investigatory holes. These four panels were installed to ensure a robust and durable seepage barrier in conditions not amenable to drilling and grouting techniques, i.e., containing significant amounts of clay.

Table 1. Cutoff Panel Details

CUTOFF PANEL NUMBER	STATION EXTENTS	EXPECTED MAX DEPTH (FT)	REASON FOR PANEL	AS-BUILT MAX DEPTH (FT)	AS-BUILT CUTOFF PANEL AREA (SF)	CONCRETE VOLUME ACTUALLY PLACED (CYDS)
1	8+00 to 8+67	35	Clay infill/void activity at depths 25-30′.	32	2013	594
2	7+00 to 7+40	35	Clay infill at depths up to 30′.	22	754	276
3	3+10 to 4+77	35	Cutoff very weathered zones in the Bangor Shale at the maximum section of the new structure.	32	5490	1416
4	2+4 to 2+50	23	Cutoff the continuation of N 23º E sluiceway solution feature, act as test panel for construction method	23	250	100

Technical and cost considerations ruled out the use of blasting and secant pile cutoffs, respectively. Instead, the simple expedient of using a hoe ram and long reach excavator was chosen, the equipment (and sequencing) being with the direct control of internal TVA forces.

After several days of set time, verification holes were cored through the centerline of each panel at 20- to 30-foot spacing, and water tested. No hole showed any measureable water take.

Performance of the Composite Cutoff in Service

Since completion of the new composite seepage barrier at Bear Creek Dam, several high headwater events have provided opportunities to evaluate the effectiveness of the grout curtain and cutoff panels acting together. Evidence that the composite seepage barrier is performing as designed includes observations of historical seepage flow outlets downstream of the treatment area. These locations currently do not exhibit boils or muddy flows, and no additional downstream sinkholes have developed. Additionally, the pumping rates required to remove seepage flows that issued from exposed rock surfaces upstream of the treatment area during high water events drastically increased, indicating that previously open flow paths had been closed, causing seepage to build up and flow from the untreated surface upstream of the RCC berm foundation prior to its construction.

RCC DAM DESIGN AND CONSTRUCTION

The cross section for the new berm was developed through a series of engineering evaluations and design reviews by the Board, TVA and the Engineer. Figure 10 represents the maximum height section and it engages the residual strength characteristics of the Bangor Shale Formation. The cross section was reduced in the upper left abutment area when the shale became deep enough to not adversely affect the stability calculations.

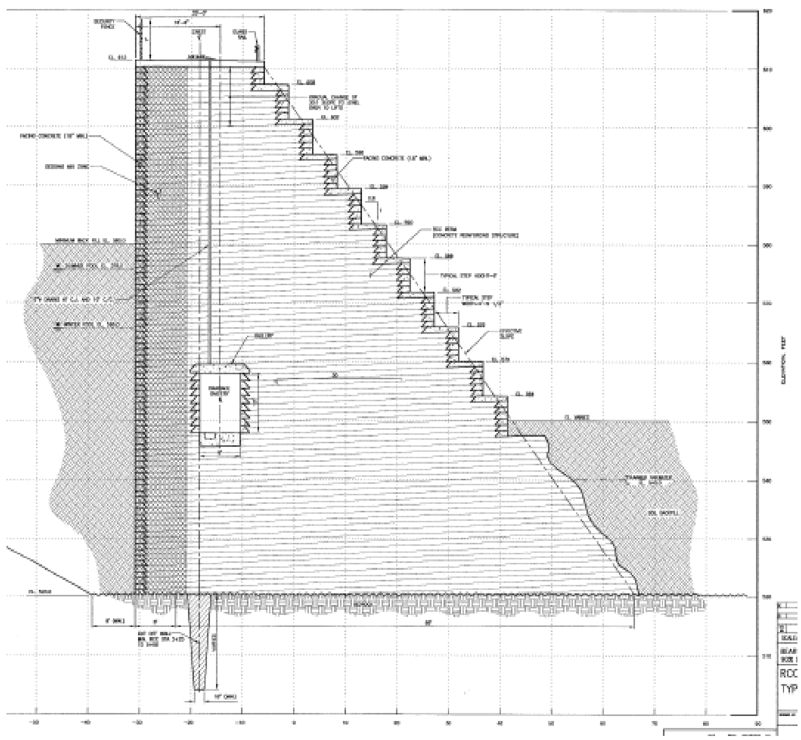

Figure 10. RCC Berm Cross Section

Overall the system of provisions in the dam and foundation design that provide for seepage control and safety include 1) the foundation grout curtain, cutoff panels, and excavated karst feature backfill treatment connected to the base of the dam, 2) an upstream facing on the dam of conventional concrete, 3) bedding mix treatment of each RCC lift surface immediately downstream of the conventional concrete facing, and 4) a drainage gallery that dam and foundation drain holes can be connected to should any seepage develop along lift surfaces, or uplift pressures in excess of the design pressures be measured by foundation instrumentation.

The RCC mix design was based on an integrated concrete and soils approach with the following design objectives and requirements:

- Strength and Workability:

  - One year Compressive Strength: 2300 psi
    Revised Design Criterion: 2000 psi. A mix with 130/130 (cement/flyash) proportions provided the required strength with factor of safety.
  - One Year Tensile Strength: 230 psi
    Revised Design Criterion: 200 psi
  - Vebe Time: 27 +/- 5 seconds
  - Placement Temp: 70º F
- Cement:
  - National Cement Company - Type II
- Fly Ash
  - Colbert (Type F Pond Ash)-Mixes BC 1–10 Aug 07
  - Cumberland (Type F)–Mixes BC 11-13 Jan 08
  - Colbert (Type F Silo Ash)–Mixes BC 14-15 May 08 and Mixes BC 16-17 Jul 08
- Aggregate:
  - VULCAN – Russellville Limestone. 50/50 blend of the Alabama 599 and 825 standard gradations

The specified aggregate grading is shown on Figure 11, and the results of the mix design testing showing the relationship between compressive and tensile testing of the RCC materials is shown on Figure 12.

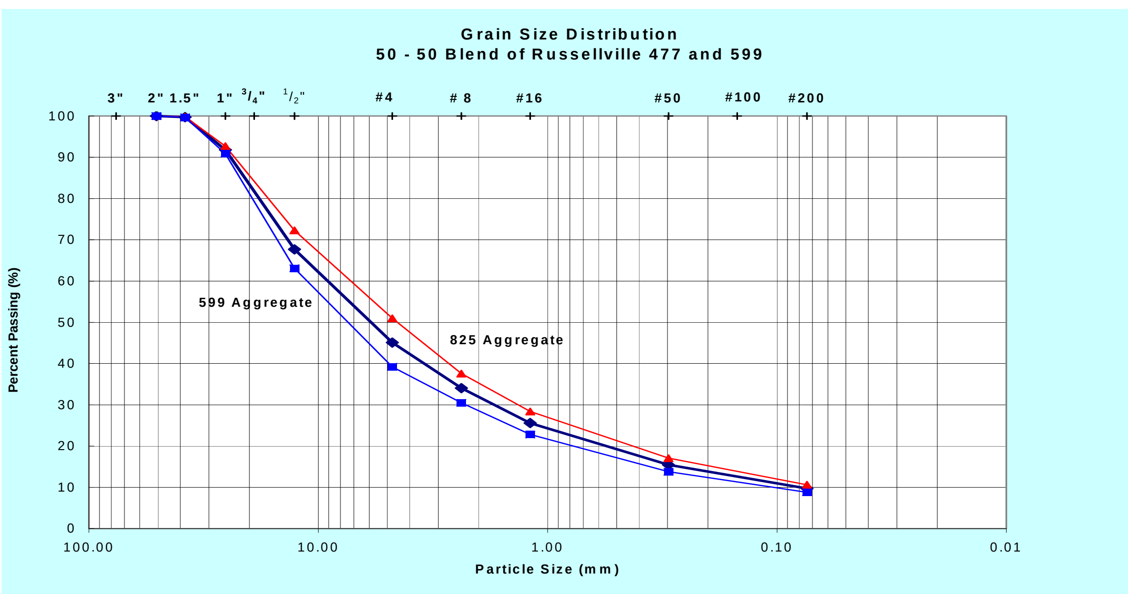

Figure 11. RCC Aggregate Grading Requirement

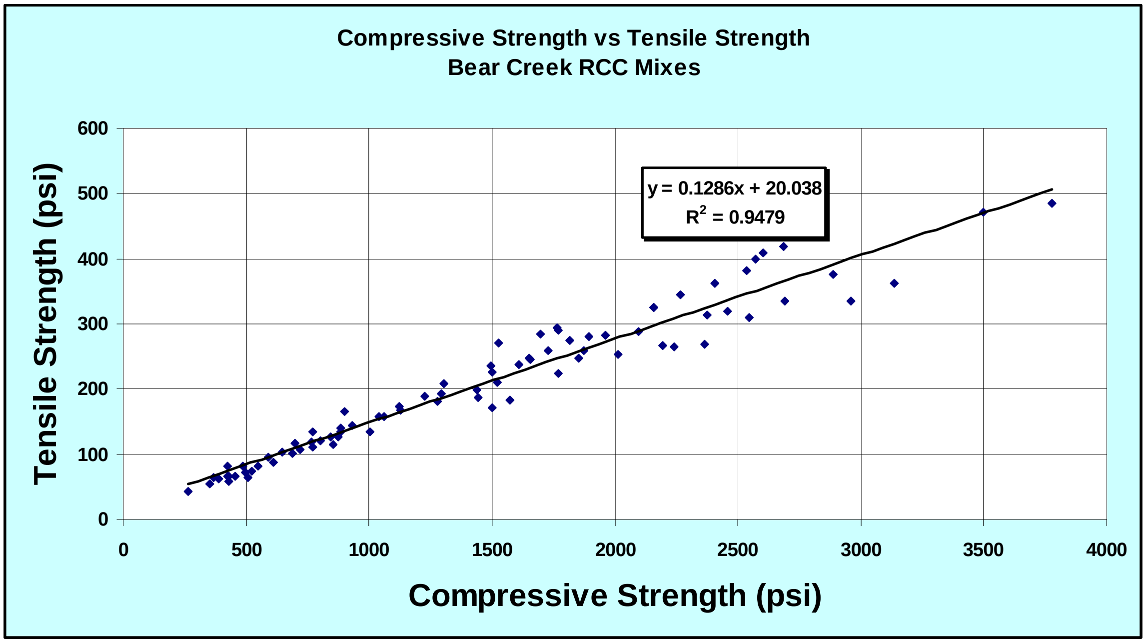

Figure 12. Relationship between Compressive and Tensile Strength determined for Bear Creek Berm RCC Mix Design

The downstream face of the completed berm is shown on Figure 13. A rigorous QA/QC program was performed by the Engineer during construction with periodic site visits and reviews by the members of the Board. To date, mix design and performance objectives have been met, a testimony to the thorough effort of all parties to achieve a high quality work product.

Figure 13. Photo of Downstream Face of Completed Berm

SUMMARY AND CONCLUSIONS

Some very important lessons were learned as part of this project:

- Karst Characterization: The characteristics of karst are unique to each specific dam site. It is difficult to characterize karst with conventional exploration methods, and experience on other sites and the performance of dams on karst is an important consideration when making decisions on the best approach to remediation or when developing the concept and footprint for a new dam. The development of a geologic model of a site that takes advantage of a full suite of existing and new characterization information is required to provide a safe design and address critical potential failure modes.
- Separation of excavation/foundation treatment and dam construction contracts: This approach to design and construction of the new RCC structure was an effective method to reduce Owner's costs and risks. Such an approach would seem to have merit on other sites involving significant karstic features.
- Some sites may be conducive to selective treatment of structural karst defects. At the Bear Creek site, the stratigraphic and structural limits of the karst provided the opportunity to use discrete panels excavated with a conventional hoe ram to fully cutoff the clay filled karst features. These panels combined with a modern and effective grouting program, and excavation and treatment of the defects at the contact between the bedrock and the dam (a thick layer of dental concrete) provide a robust and long term means to control foundation seepage and provide adequate dam safety. This use of the composite wall concept resulted in a savings of well over 10 million dollars over a continuous cutoff wall (secant pile method) as initially envisioned for the Bear Creek project.
- Independent reviews by experts experienced with karst sites are a key component to project success and cost reduction.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the following key project participants who made significant contributions to the overall success of this project:

Tennessee Valley Authority
Russell Tompkins, P.E., Manager of Dam Safety;
James Niznik, Geotechnical Engineer; John Duncan, P.E., Senior Engineer; and Scott R. Kramer, P.E., Construction Engineer.

Paul C. Rizzo and Associates (Design Engineer)
John Osterle (Project Manager), Grady Adkins (Senior Geotechnical Engineer), John Charlton (Engineering Geologist), Paul C. Rizzo (Principal-in-charge), and Conrad Ginther (grouting engineer)

RCC Berm Contractor – ASI Constructors Inc., Pueblo, Colorado

REFERENCES

Bruce, D.A., T.L. Dreese, D.B. Wilson (2009). "Design, Construction and Performance of Seepage Barriers for Dams on Carbonate Foundations," USSD Annual Conference, April 20-24, Nashville, TN, 16 pp.

Ginther C.H., S.R. Kramer , and J.A. Niznik (2009). "Comprehensive Foundation Rehabilitation at Bear Creek Dam." Proceedings of ASDSO Dam Safety 2009 Conference; September 27- October 1, Hollywood, FL, 25 pp.

Kitko, M. (2009). "Grouting at Bear Creek Dam, AL." Presentation at ASDSO Northeast Regional Conference, June 16, Hershey, PA.

USBR (2001)