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Letter from the Editorial Committee

The purpose of the ASDSO Journal is to provide opportunity for the dam safety community to share important information, lessons learned, and new technology. If you have a project or topic that you feel is worthy to share with your peers through the Journal, please feel free to contact us with an abstract of 1/2 to 1 page in length for our evaluation. Please address comments, questions, or abstracts to the Editorial Committee chair at keith.ferguson@hdrinc.com.

We would appreciate feedback on any of the papers published in any of the recent issues of the Journal. Comments, questions, and/or personal experiences related to articles that are published, or on a topic of interest are welcome.

The committee wishes to thank the authors of the articles in this edition for the timeliness and quality of their work.

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Lyle Bentley (State of Tennessee)
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ASDSO Dam Safety Journal

The Journal of Dam Safety welcomes articles from any writer, but the opinions expressed in those articles do not necessarily reflect the opinions and policies of ASDSO.

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Lake Townsend Dam impounds the primary water supply for the City of Greensboro, North Carolina. The concrete gated spillway is suffering from severe deterioration due to alkali silica reactivity (ASR) and has inadequate hydraulic capacity to meet spillway design storm (SDS) requirements of North Carolina Dam Safety.

The project includes a replacement spillway designed to have hydraulic capacity similar to the existing gated spillway. The embankment will be armored to allow overtopping for storms up to the SDS. Subsurface investigations revealed soft and loose alluvial soils in the original stream and floodplain. Excavation and replacement of these soils is necessary for support of the new spillway and earth embankments.

The effect of spillway submergence suggests that a fixed crest labyrinth weir is more appropriate than a gated spillway. In addition, the City of Greensboro prefers the limited operation and maintenance requirements of a fixed crest spillway. The replacement spillway is a seven cycle, 300 ft wide, 20 foot high labyrinth weir. The embankment will be armored with articulating concrete blocks (ACB) to protect against failure from overtopping.

The design included hydraulic modeling using computational fluid dynamics (CFD) and a physical model study of the labyrinth and energy dissipater. The structural design included finite element modeling of the labyrinth weir. Plans and specifications were completed in 2008 and construction began in spring 2009.
**PROJECT DESCRIPTION**

Lake Townsend is a 1,635-acre impoundment with a reservoir storage capacity of 6,330 million gallons at normal pool. The City of Greensboro Water Department serves a population of approximately 250,000 and Lake Townsend provides approximately 70 percent of the City’s raw water storage capacity. The dam is located on Reedy Fork Creek, approximately 10 miles northeast of downtown Greensboro, west of US Highway 29 near Browns Summit (Figure 1).

The drainage area at the dam is 105 square miles. Lake Brandt, Lake Higgins, and Lake Jeanette are located upstream of Lake Townsend. Lake Brandt and Lake Higgins are both water supply reservoirs owned by the City of Greensboro.

Lake Townsend Dam consists of a gated concrete spillway with earth embankments to the left and right. The spillway is an ogee-shaped weir divided into nine, 25-ft wide bays and one 15-ft wide bay. Ten-foot-high vertical lift gates are located atop the concrete spillway in each of the 25-ft wide bays and a skimmer gate is located in the 15-ft wide bay. The water supply intake and pump station are integral to the spillway. A 200-ft wide earthen auxiliary spillway was excavated through the left abutment, about 1000 ft north of the concrete spillway section. Table 1 summarizes elevations of the dam and spillways.

Lake Townsend Dam is regulated by the North Carolina Department of Environment and Natural Resources (NC DENR) and classified as a Class C (high hazard), “Large” dam. Based on this NC DENR classification, the Spillway Design Storm (SDS) for Lake Townsend Dam is the 3/4 Probable Maximum Precipitation (PMP).

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**Table 1: Lake Townsend Dam - Project Elevations**

<table>
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<th>Feature</th>
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<tr>
<td>Crest of Ogee</td>
<td>705.5</td>
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<tr>
<td>Top of Gates (normal pool)</td>
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</tr>
<tr>
<td>Auxiliary Spillway Crest</td>
<td>718.5</td>
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<tr>
<td>Top of Dam (embankment crest)</td>
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PROJECT UNDERSTANDING AND OBJECTIVES

The Lake Townsend Dam was constructed in 1966-1967 when testing of concrete aggregates for Alkali Silica Reaction (ASR) was not common in the southeast. Concrete including reactive aggregates in a damp or submerged state at Lake Townsend Dam resulted in ASR and the concrete in the spillway and pump station has exhibited significant expansion, cracking, and deterioration, as shown in Figures 2 and 3.

Lake Townsend Dam was designed prior to implementation of a North Carolina dam safety law or specific design requirements related to spillway capacity for dams. The original design reportedly considered “the maximum storm for the area”, which was computed to be 50,000 cfs (Papp 1970). Several reports dating back to 1980 included hydrologic and hydraulic analyses that resulted in a computed 3/4 PMP inflow greater than 115,000 cfs, indicating inadequate capacity to pass the 3/4 PMP without overtopping of the embankment (Thomas 1980; Hazen and Sawyer 1988; Simons 1989).

The primary objectives for the project were to address 1) the ASR concrete deterioration through a spillway repair or replacement and 2) the inadequate spillway capacity per NC DENR dam safety criteria. Because Lake Townsend is Greensboro’s primary water supply, key project considerations were to maintain the reservoir and to provide for uninterrupted water supply throughout construction. A three phased design approach was applied to the evaluation and design for the project as described below.

PHASE 1 PRELIMINARY ALTERNATIVES ASSESSMENT

Phase 1 of the project included an evaluation of the existing structure and alternatives for rehabilitation or replacement.

Spillway Condition Assessment

Cracking and signs of distress of the concrete spillway had been observed and documented for more than 25 years, but the cause of the deterioration was not identified until recently. Jewell Engineering Consultants, PC (Jewell 2005a and Jewell 2005b) developed two reports that included extensive investigations documenting the ASR concrete damage. Several recovered concrete cores were partially disintegrated, resembling slightly cemented sand and gravel. These reports also suggest that a long-term repair of the structures is not feasible and complete or substantial replacement of the concrete structures was recommended.

Phase 1 included inspections and a thorough evaluation of Lake Townsend Dam. An underwater inspection by Glenn Underwater Services, Inc on October 30, 2006 revealed numerous cracks in the upstream face. Most of the cracks were about one to two inches wide on the surface, narrowing to less than one-eighth inch wide at a depth of about one to three inches. At two locations, the diver was able to advance a folding ruler 25 and 28 inches into the cracks. Stability analyses of the spillway indicated that the spillway does not meet generally accepted stability criteria for gravity dams. Stability along horizontal joints was of particular concern based on the depth of selected cracks on the upstream face and observations of seepage at these joints on the downstream face. To address these stability concerns, the normal pool was lowered by two feet on November 3, 2006 until post-tensioned anchors were installed January through March 2007 to increase stability of the upper portions of the structure so that the lake could be maintained throughout design and construction.

Spillway Capacity

Reports dating back to 1980 indicate that Lake Townsend Dam has inadequate spillway capacity. The computed capacity of the existing spillways (gated and auxiliary) at top of dam is about 80,000 cfs, assuming all gates are fully opened. The computed 3/4 PMP inflow to Lake Townsend is 136,700 cfs, and the outflow is 110,600 cfs, resulting in 2.6 feet of overtopping. The estimated overtopping duration is 5.7 hours for the 3/4 PMP.
Overtopping of the Lake Townsend Dam embankments would likely cause a dam failure, resulting in an uncontrolled release of reservoir storage, loss of the water supply reservoir, and potential loss of life and property. In addition, all three dams located upstream of Lake Townsend were found to overtop utilizing the modeled 3/4 PMP, likely causing failure of these structures and increased inflow to Lake Townsend. Because the failure of the upstream dams would not cause flooding of any known habitable structures and the added cost of rehabilitation or replacement of Lake Townsend dam is not considered significant, the City elected to consider the failure of the upstream dams as part of the Lake Townsend Dam project. Failure of these three dams was incorporated into the hydrologic and hydraulic model, resulting in a computed peak total inflow to Lake Townsend of 213,000 cfs for the 3/4 PMP. This inflow hydrograph was used for the final design of the Lake Townsend project.

**Auxiliary Spillway Evaluation**

The existing auxiliary spillway is a 200 ft wide excavated earthen channel located about 1000 feet north of the dam. The control section is a three-foot deep concrete cutoff wall with a crest elevation three feet above normal pool, or EL 718.5. The hydraulic capacity of the auxiliary spillway is estimated to be about 9,600 cfs at the dam crest (EL 725.5). If the principal spillway gates are operated to maintain normal pool until they are completely opened, it is estimated that the auxiliary spillway would flow for storm events larger than the 100-year flood.

Based on a seismic refraction survey of the existing auxiliary spillway, the depth to bedrock in this area ranges from about 20 feet to more than 35 feet below the ground surface. A preliminary stability and integrity analysis of the auxiliary spillway was completed using the Natural Resource Conservation Service (NRCS) SITES computer program. The SITES analysis indicates that erosional failure of the auxiliary spillway is likely during the SDS.

**Initial Alternatives Screening**

To address the ASR, spillway capacity, and stability concerns at Lake Townsend Dam, concepts for rehabilitating the existing structure along with several spillway replacement alternatives were developed. Rehabilitation of the Existing Spillway: In-place rehabilitation of the existing ASR damaged spillway structure within the project constraints would likely include:

- construction of an extensive cofferdam system across the entire spillway;
- demolition of the piers, bridge, and other superstructure;
- removal of several feet from the top of the existing ogee spillway to remove damaged concrete;
- concrete resurfacing of the entire ogee section;
- construction of new piers, gates and superstructure;
- replacement of the energy dissipater with a larger structure to accommodate increased unit discharge;
- installation of a membrane or other impervious barrier on the upstream face of the spillway to reduce further damage from ASR;
- internal stabilization of the structure by anchoring and/or grouting; and
- modifications to increase spillway capacity by increasing gate size, lowering the spillway crest, providing overtopping protection, or raising the dam.

These measures do not consider rehabilitation of the existing training walls, intake, pump station, and other concrete structures which exhibit less damage but will likely continue to deteriorate due to ASR.

**Spillway Replacement Alternatives:** Based on the site constraints, project goals, anticipated “order of magnitude” construction costs, and permitting considerations, four spillway replacement alternatives were developed for evaluation. For Alternatives 1, 3, and 4, a dike will be constructed through the auxiliary spillway to prevent flow and erosional failure of this spillway during the SDS.

**Alternative 1 - Single Spillway, Maintain Current Top of Dam Elevation**

For this alternative, a new spillway will be sized to pass the SDS without overtopping of the embankment. The structure could be a labyrinth weir, gated spillway, or a combination of these structures. Preliminary routings were performed to size both a gated and labyrinth spillway. Based on these routings, a 20 ft high labyrinth spillway having a width of 560 ft (14 cycles) would be required to pass the SDS. For a gated structure having a width similar to the existing spillway (280 ft), the gates would need to be 16 feet high to pass the SDS.

**Alternative 2 - Two Spillways, Maintain Top of Dam**

Alternative 2 includes replacement of the spillway with a new principal spillway (labyrinth, gated, or combination) and an auxiliary labyrinth spillway located in the existing auxiliary spillway to pass the SDS without overtopping the embankment. Routings were performed to size both structures. A combination gated/labyrinth structure with a length of 280 ft was assumed. The labyrinth portion consists of five, 40-ft wide cycles and a weir height of 20 ft. The gated portion is 80 ft wide and the gates are 20 feet high. The auxiliary labyrinth spillway would have a total width of 200 ft to match the existing auxiliary spillway width. This labyrinth would consist of a 12.5-ft high weir and 10 cycles.

**Alternative 3 - Single Spillway, Armor Embankment**

For this alternative, the principal spillway would be replaced with a spillway having hydraulic capacity similar to the existing spillway. The embankment would be armored to protect against failure due to overtopping. Common armoring materials used for this application are articulating concrete blocks (ACB) and roller compacted concrete (RCC). For both options, we preliminarily sized a combined structure with a 6 cycle, 20 ft high labyrinth weir and a gate roughly 35 ft wide and 15 ft high. The computed overtopping depth was 4.7 ft.

**Alternative 4 - Single Spillway, Raise Top of Dam**

The principal spillway configuration for Alternative 3 was also used for Alternative 4; however, the top of dam would be raised as necessary to provide additional flood storage and prevent overtopping for the SDS. Raising the dam could be accomplished using earthfill and/or a parapet. A raise of about 6 ft would be necessary.

Alternative 3 was selected as the spillway replacement approach for Lake Townsend Dam. This alternative is believed to be the least costly, has little environmental impact, and provides passage of the SDS, including the failure of upstream dams for a nominal increase in cost.
PHASE 2
DETAILED INVESTIGATIONS AND ALTERNATIVES EVALUATION

Phase 2 included more detailed hydraulic analysis, geotechnical analysis based on the Round 1 Investigation program, and evaluation of alternatives within the framework of Alternative 3, selected in Phase 1.

Round 1 Geotechnical Investigation

The Round 1 geotechnical investigation was performed during Phase 1 and 2 of the project and was intended to collect sufficient subsurface information to evaluate the relative feasibility of design alternatives. The Round 1 investigation included:

- Review of borings logs and subsurface profiles from the field investigations performed as part of the original dam design;
- Four seismic refraction survey lines and four test borings to estimate the depth to rock and the characteristics of the soils in the auxiliary spillway;
- Seven test borings located in the reservoir upstream of the spillway to provide subsurface information for evaluation of cofferdam requirements;
- Two concrete cores in the spillway and four test borings in the stilling basin to evaluate uplift pressures within and below the spillway and stilling basin;
- Three test borings immediately downstream of the earth embankments to provide preliminary data to evaluate the requirements for new earth embankments and spillways;
- Sixteen test pits to collect information and samples for laboratory testing to evaluate the feasibility of on-site borrow areas; and
- Preliminary laboratory testing to characterize the index, strength, and compressibility properties of potential foundation and borrow soils.

Significant results and conclusions of the Round 1 field investigation were as follows:

- The existing auxiliary spillway is a channel excavated in fine grained, erodible soils. The depth to rock varies from about 20 feet to 35 feet.
- The existing seepage protection and collection measures appear to effectively reduce uplift pressures below the dam and stilling basin.
- The presence of soft and loose alluvial soils downstream of the existing dam within the historic floodplain was confirmed.

Tailwater Modeling

The flow depth in Reedy Fork Creek downstream of the dam (referred to as tailwater) affects the discharge capacity, energy dissipation, and structural design of the proposed replacement spillway. A hydraulic model downstream of Lake Townsend was developed using HEC-RAS and the results are shown in Figure 4. The results indicate that there is potential for submergence of the new spillway for extreme floods, reducing discharge capacity. Submergence has a more significant effect on a gated spillway than a fixed crest weir.

Spillway Hydraulics and Routing

The submergence of the spillway for large flows suggests that a fixed crest labyrinth weir is more appropriate than a gated structure for the replacement spillway at Lake Townsend Dam. There are also fewer operational and maintenance issues for a labyrinth.

The proposed spillway was designed to generally match the hydraulic capacity of the existing spillways with the reservoir level at top of dam (EL 725.5), which is approximately 80,000 cfs. A seven cycle labyrinth with a weir height of 20 ft, cycle width of 42 ft, and cycle depth of 80 ft has a capacity of 82,700 cfs for the same reservoir elevation. Tailwater begins to submerge the weir (i.e. reaches EL 715.5) at about 87,000 cfs. Spillway rating curves for the existing and proposed spillways are presented in Figure 5.

The results of the hydrologic and hydraulic analysis for the existing and proposed spillways are presented in Table 2.

Both the existing and proposed spillways have discharge capacity equal to about 60 percent of the PMP (assuming no failure of upstream dams) with the reservoir at top of dam (EL 725.5).

Stream Diversion Capacity

Maintaining the reservoir during construction is critical to this project. To accommodate the construction sequence of the proposed new spillway and embankments downstream of the dam, it was necessary to establish the minimum capacity of a stream diversion...
through the proposed construction site. The number of gates to be used during a given phase of construction will affect the diversion capacity and the ease of construction. Fewer gates would likely facilitate less costly construction but increase the risk of flooding of the construction site.

The existing spillway will accommodate installation of stop logs or bulkheads upstream of the gates to EL 718.0, 2.5 feet above normal pool. For a full gate opening, each spillway bay has a capacity of about 3,500 cfs for a reservoir EL 718.0.

The selection of diversion capacity was based on historical stream flow along with estimates of storm return period using various sources. Peak annual flows at the USGS gage on Reedy Fork near Gibsonville (131 square mile drainage area) are shown in Figure 6.

Based on cost, constructability issues, and risk, the City selected a scheme in which three spillway bays will be used for stream diversion during various stages of construction. This provides diversion capacity of more than 10,000 cfs.

Evaluation of Alternatives

Three alternatives were further developed and evaluated within the framework of Alternative 3 developed in Phase 1.

Alternative 3A: This alternative includes construction of the new spillway as close to the existing spillway as considered technically feasible while still using the existing dam and spillway as a cofferdam. This alternative includes permanent cellular sheet pile structures (cells) constructed adjacent to existing structures, just north of the spillway and just south of the pump station. These cells would provide a transition from the existing embankment to the proposed spillway. The upstream edge of the proposed spillway would be located within the existing stilling basin, about 18 ft upstream of the end sill. The existing embankment would be armored with Articulating Concrete Block (ACB) to prevent failure due to overtopping. The new spillway would be constructed in two phases to provide for diversion during construction.

Alternative 3B: For this alternative, the new spillway would be constructed further downstream than for Alternative 3A. The upstream edge of the new spillway would be about 40 ft downstream of the existing stilling basin. Earth embankments would be constructed to connect the new spillway to the abutments. The embankments would be armored with ACB to prevent failure due to overtopping. The new spillway would be constructed in two phases to accommodate diversion during the construction period.

Alternative 3C: This alternative is similar to Alternative 3B except that the proposed spillway would be located farther south to allow diversion through the north end of the existing spillway and construction of the labyrinth replacement spillway in one phase.

Comparison of Final Alternatives: Factors considered in the comparison of the three alternatives included: cost, diversion and

Table 2: 3/4 PMP Results (includes modeled failure of all three upstream dams)

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Figure 5: Spillway Discharge for Existing and Proposed Spillways

Figure 6: Stream Gage Annual Peaks (Courtesy of USGS)
sequencing, construction complexity, construction schedule, site impacts, rehabilitation versus replacement, foundation conditions, design and construction flexibility, and risk to water supply.

Conceptual layouts were developed for these alternatives to serve as the basis for quantity and cost estimates. The construction staging of each alternative was considered in these layouts. Cost estimates indicated similar costs for each alternative with the calculated differences in cost within the margin of error; therefore, cost was not considered a major factor in the selection of a preferred alternative.

Alternative 3C was the preferred alternative based on the following:

- Diversion and sequencing would be less complex since the spillway can be constructed in a single phase while flow is diverted to the north of the structure. The other alternatives require phased construction of the spillway.
- Alternative 3A is considered more complex and specialty contractors will be required for construction of the sheet pile cells and various excavation support systems.
- The construction of the spillway in a single phase will benefit the construction schedule and cost.
- Alternatives 3B and 3C provide more flexibility than Alternative 3A during design and construction since the location of the structure can be adjusted.
- For Alternative 3C, the depth of excavation and replacement of soils for the spillway structure will be less than for Alternative 3B, reducing the potential for differential settlement.

Alternative 3A is believed to present the most risk to the existing structures and the water supply during construction.

Based on “order of magnitude” estimates, the three alternatives are considered comparable with regard to estimated construction cost; however, there are likely inherent cost savings with Alternative 3C related to sequencing and schedule that cannot be considered in the level of estimate performed. For this reason, Alternative 3C is believed to be the least costly alternative.

PHASE 3

FINAL DESIGN

Phase 3 consisted of the final project design and development of plans and specifications. This included hydraulic, structural, and geotechnical design and final layout of features.

Round 2 Geotechnical Investigation

Round 2 of the geotechnical investigation program was designed to provide sufficient data to perform geotechnical analyses and design during Phase 3 and characterize the subsurface conditions within the limits of proposed construction. Round 2 geotechnical field investigations included:

- Thirteen cone penetration test (CPT) probes to estimate the limits and characteristics of soft and loose alluvial soils;
- Ten test borings and observation wells to provide additional subsurface information within the historic floodplain and allow long term monitoring of groundwater levels;
- Eleven test borings to evaluate the subsurface conditions outside of the historic floodplain and within the limits of the proposed earth dam;
- Nine additional test pits to expand the investigation area of potential on-site borrow areas;
- Field permeability testing that included packer testing in rock and slug tests in observation wells to evaluate the permeability of potentially excavated soils and the dam foundation; and
- Additional laboratory testing to estimate the strength and compressibility of foundation soils and the existing embankment dam.

The results of the Round 1 and Round 2 field investigation were combined and summarized to form the basis for geotechnical analyses performed during Phase 3.

Hydraulic Modeling and Design

The labyrinth configuration was developed in Phase 2 using proven empirical methods for estimating discharge. Downstream of the labyrinth, a stepped chute or series of drops was proposed to help dissipate energy for smaller floods. For large floods, the tailwater submerges the flow downstream of the weir, reducing the required energy dissipation. Computation fluid dynamics (CFD) and a physical model were used to estimate labyrinth discharge and evaluate the effectiveness of various stepped chute configurations. Initially, two-dimensional CFD was used for screening of numerous configurations. Based on the CFD results, four configurations (Figure 7) were evaluated using a physical model.
The final stepped chute configuration was selected based on the model results (Figure 8 includes a graphic from the CFD model and photo from the Physical Model). Configuration 4 will be used downstream of the normal flow weir and in line with the creek; Configuration 3 will be used for the remaining portion of the spillway (downstream of the high flow portion of the labyrinth weir).

A discharge rating for the proposed labyrinth was also developed using a combination of the empirical estimates and numerical and physical model results.

It is estimated that the earth embankments will overtop for storms greater than about 60 percent of the PMP. The computed maximum depth of overtopping for the SDS is 5.2 ft; therefore, the embankment will be protected from erosive failure due to overtopping flow. Articulating concrete block (ACB) systems have been used on numerous projects to protect embankments from failure due to overtopping, and the method was chosen as the most cost-effective means to protect the dam from overtopping flow. The stability of the ACB system was evaluated using methods presented in Articulating Concrete Block Revetment Design - Factor of Safety Method (NCMA TEK 11-12, 2002).

The CFD and physical modeling and the ACB design are discussed in more detail in the ASDSO paper entitled The Hydraulic Design Toolbox: Theory and Modeling for the Lake Townsend Spillway Replacement Project (Paxson et al, 2008).

**Geotechnical Design**

Soft alluvial soils below the proposed spillway and embankments will be removed and replaced with compacted fill. The excavation is immediately downstream of the existing dam and will require removal of a portion of the existing downstream embankment slope. The excavation will be as much as 30 feet deep and extend to about 25 feet below groundwater levels. Geotechnical analyses performed to evaluate construction issues related to the excavation and the performance of the completed dam included:

- Staged finite element analyses using Plaxis to evaluate excavation stability and settlement of the new dam and concrete spillway;
- Limit equilibrium (slope stability) analyses to evaluate the stability of the new dam and confirm the results of the finite element analyses;
- Evaluation of dewatering requirements and development of dewatering criteria; and
- Seepage analysis for the new earth dam and new concrete spillway foundation.

The stability of the ACB system was evaluated using methods presented in Articulating Concrete Block Revetment Design - Factor of Safety Method (NCMA TEK 11-12, 2002).
A zoned embankment with a chimney and blanket drain was proposed. The design included founding of the concrete spillway on fill compacted to 95% of its maximum dry density as determined by ASTM D1557 (modified Proctor). The specified compaction criteria are more restrictive than typical large earthwork projects in North Carolina; however, using more stringent compaction criteria resulted in a 1/2 inch reduction in estimated settlement of the new concrete spillway.

**Structural Design**

Because the labyrinth will be constructed on compacted fill, the structural design was dictated by seepage considerations, the estimated total settlement of the structure, and differential settlement between concrete placements. Concrete blocks were sized to allow a placement of a reasonable volume of concrete per placement, but limited to reduce thermal effects. Concrete placements will be made in a checkerboard pattern and adjacent placements will not be allowed until after 7 days to allow concrete hydration temperatures to dissipate and the concrete blocks to reach an equilibrium temperature. In addition, a concrete mix using 50% cement and 50% Class F fly ash has been specified to limit heat generation yet achieve a concrete strength of 4000 psi at 56 days. To limit seepage, a sheet-pile cutoff will be constructed beneath the upstream edge of the labyrinth, drain fill and drainage piping will be placed beneath the downstream portion of the labyrinth structure, and all concrete joints will be doweled with waterstop.

The structural design of the labyrinth weir was performed using the STAAD Pro computer program using plate finite elements in a 3-D configuration for each block and labyrinth cycle. The structural plans were developed using a three-dimensional AutoCAD model (see Figure 9). This facilitated the spillway layout and enabled the designers to evaluate conflicts between adjacent structural elements.

**Construction Sequence and Final Project Layout**

Because the reservoir is to be maintained throughout construction, diversion of flows around the construction site is critical to the project. The construction sequence developed as part of the design is generally as follows:

**Stage 1**

1. Install bulkheads in the six southern spillway bays so three northern bays are available for diversion.
2. Construct Stage 1 diversion wall parallel to flow and stream crossing/downstream cofferdam. These structures will also serve as excavation support during Stage 1.
3. Dewater to lower the water table to below the anticipated depth of excavation.
4. Excavate soft alluvial soils beneath the labyrinth spillway and south embankment and replace with structural fill.
5. Construct labyrinth spillway and south embankment.

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Stage 2

1. Install bulkheads in northern spillway bays to divert flows through southern bays and proposed spillway outlet works bay. Portions of the labyrinth weir walls will be only partially constructed to allow passage of larger flows during construction.
2. Construct north embankment.
3. Remove portions of existing embankments above normal pool.

Stage 3

1. Install outlet works and complete construction of labyrinth weir walls.
2. Open existing spillway gates and water up the new spillway and embankments.
3. Remove existing spillway and pump station superstructure.
4. Demolish existing spillway and pump station to ten feet below normal pool.
5. Excavate portions of existing embankment to allow flow into new spillway.

The proposed project layout is shown in Figure 10. Based on the complexity and risk associated with the project, interested contractors were required to submit qualifications and only prequalified contractors were issued bid documents. Construction began in spring 2009.

REFERENCES

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