21st Century Dam Design — Advances and Adaptations

31st Annual USSD Conference
San Diego, California, April 11-15, 2011
On the Cover

Artist's rendition of San Vicente Dam after completion of the dam raise project to increase local storage and provide a more flexible conveyance system for use during emergencies such as earthquakes that could curtail the region’s imported water supplies. The existing 220-foot-high dam, owned by the City of San Diego, will be raised by 117 feet to increase reservoir storage capacity by 152,000 acre-feet. The project will be the tallest dam raise in the United States and tallest roller compacted concrete dam raise in the world.

U.S. Society on Dams

Vision

To be the nation's leading organization of professionals dedicated to advancing the role of dams for the benefit of society.

Mission — USSD is dedicated to:

- Advancing the knowledge of dam engineering, construction, planning, operation, performance, rehabilitation, decommissioning, maintenance, security and safety;
- Fostering dam technology for socially, environmentally and financially sustainable water resources systems;
- Providing public awareness of the role of dams in the management of the nation's water resources;
- Enhancing practices to meet current and future challenges on dams; and
- Representing the United States as an active member of the International Commission on Large Dams (ICOLD).

The information contained in this publication regarding commercial projects or firms may not be used for advertising or promotional purposes and may not be construed as an endorsement of any product or from by the United States Society on Dams. USSD accepts no responsibility for the statements made or the opinions expressed in this publication.

Copyright © 2011 U.S. Society on Dams
Printed in the United States of America
Library of Congress Control Number: 2011924673

U.S. Society on Dams
1616 Seventeenth Street, #483
Denver, CO 80202
Telephone: 303-628-5430
Fax: 303-628-5431
E-mail: stephens@ussdams.org
Internet: www.ussdams.org
LAKE TOWNSEND DAM REPLACEMENT — CONSTRUCTION UPDATE, GREENSBORO, NC

Robert Cannon, P.G.1
Gerald Robblee, P.E.1
Jerry Gardner, RPR1
Allan Williams, P.E.2

Tillman Marshall1
Frederic Snider, P.G.1
Melinda King, P.E.2
Andrew R. Downs, P.E.3

ABSTRACT

Lake Townsend Dam impounds the primary water supply for the City of Greensboro, North Carolina. The original 44-year old concrete gated spillway is suffering from severe deterioration due to alkali silica reactivity (ASR) and has inadequate hydraulic capacity. After an analysis of repair and replacement options, the selected alternative consists of a new spillway designed with a hydraulic capacity similar to the existing spillway and allowance for embankment overtopping during high flows. The new replacement dam is being constructed immediately downstream of the existing dam. The new spillway consists of a reinforced concrete, seven cycle, 300-ft wide labyrinth with a weir height of 20 feet. Articulating concrete blocks (ACB) will be used to armor the earthen embankments. Underwater demolition of the existing spillway and portions of the embankments will be completed after commissioning of the new dam.

Construction of the new replacement dam began in spring 2009, with an estimated duration of 30 months. Planned commissioning is in November, 2011. The Contractor has faced multiple challenges during construction. Working downstream of a full, operational reservoir entailed additional risk. Diversion of flood flows up to about 10,000 cfs was required. Extensive dewatering was necessary, as soft alluvial clays and loose alluvial sand had to be excavated in the floodplain below the footprint of the new dam. The foundation excavation exceeded 30 feet in the deepest parts. Foundation preparation also required removal of part of the downstream slope of the original embankment. Geotechnical instrumentation was installed to allow performance monitoring of the remaining embankment during foundation excavation and dewatering. Borrow area soils were too wet to achieve the stringent compaction requirements needed for the labyrinth spillway foundation. Several alternatives were tested, and ultimately, cement was added to the site soils at 5% by weight. This cement-modified soil, or CMS, provided numerous benefits during construction.

This paper is a follow-up to a paper entitled Lake Townsend Dam Replacement Project, Greensboro, NC, presented at the April, 2009 Annual USSD conference in Nashville, Tennessee (Cannon et al. 2009). The earlier paper provides a summary of the site investigations and alternatives assessment.

1 Schnabel Engineering, 11A Oak Branch, Greensboro, NC 27407, rcannon@schnabel-eng.com, tmarshal@schnabel-eng.com, grobblee@schnabel-eng.com, fsnider@schnabel-eng.com, jgardner@schnabel-eng.com
2 City of Greensboro, 2602 S. Elm-Eugene Street, Greensboro, NC 27406, melinda.king@greensboro-nc.gov, allan.williams@greensboro-nc.gov
3 Crowder Construction Company, 6409 Brookshire Blvd. Charlotte, NC 28216, adowns@crowdercc.com
PROJECT DESCRIPTION

Lake Townsend Dam is located on Reedy Fork Creek in Guilford County, North Carolina, about 10 miles northeast of downtown Greensboro. Lake Townsend is a 1,635-acre impoundment with a storage capacity of 6,330 million gallons at normal pool. The City of Greensboro Water Department serves a population of approximately 250,000 people, with Lake Townsend providing approximately 70 percent of the City’s raw water storage capacity. The drainage area at the dam is 105 square miles.

The existing Lake Townsend Dam is an earth embankment with a gated concrete spillway. The spillway consists of a concrete, 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. A 200-ft wide earthen emergency spillway is located at the north abutment.

The existing dam was constructed in 1966/1967 when testing of concrete aggregates for ASR was not common. Today, the existing concrete in the spillway is exhibiting expansion, cracking, and deterioration due to ASR, as shown in Figures 1 and 2.

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. As such, the Spillway Design Flood (SDF) is the ¾ Probable Maximum Precipitation (PMP). The existing dam was designed and built prior to implementation of North Carolina dam safety laws. 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 ¾ PMP inflow greater than 115,000 cfs and noting that the project does not have adequate capacity to pass this inflow without overtopping of the embankment (Thomas, 1980; Hazen and Sawyer, 1988; Simons, 1989).

Therefore, the primary objectives for the new dam were to address 1) the ASR concrete deterioration through spillway repair or replacement and 2) the spillway’s inadequate discharge capacity per NC DENR dam safety criteria. Because Lake Townsend is the
City’s primary water supply, it was imperative that the reservoir be maintained to provide uninterrupted water supply throughout construction.

The selected alternative was the design and construction of new labyrinth spillway immediately downstream of the existing dam, as illustrated on Figure 3. Interested Contractors were required to submit a qualifications package and only pre-qualified Contractors were issued bid documents. Crowder Construction Company of Charlotte, NC was the selected bidder, and construction began in the spring of 2009.

Figure 3. Layout of new dam and labyrinth spillway. Once the new dam is completed, the existing spillway and excess embankment will be removed.

STREAM DIVERSION AND CONSTRUCTION SEQUENCING

Maintaining a nearly full reservoir during construction of the new dam resulted in a three stage diversion scheme. The Stage 1 diversion plan is shown on Figure 4. A steel diversion wall was designed to divert flows up to 10,000 cfs to the north of the entire labyrinth spillway, thereby allowing the spillway to be built in one stage.

The North Embankment would be constructed in Stage 2, when flood flows would be diverted through gaps in the labyrinth walls. During Stage 3 the gaps would be closed, the space between the two dams flooded, and part of the existing dam removed. Removal will include underwater demolition and debris removal.
DIVERSION WALL DESIGN, CONSTRUCTION, & PERFORMANCE

The Stage-1 diversion wall was designed to meet the following criteria:

- Provide a physical and hydraulic barrier between the Stage 1 diversion channel and the Stage 1 excavation and construction area.
- Support the lateral loads placed on the wall including lateral soil and water loads, compaction induced loads, and lateral loads due to the weight of the spillway structure and Stage 2 water loads.
- Be stiff enough that deformation due to structure induced loads does not adversely affect the future performance of the wall or the spillway structure.

The diversion wall was connected to the existing spillway structure and was to be connected to the contractor’s downstream cofferdam. The structural design of the cofferdam was performed by the design engineer (Schnabel Engineering). The Contractor was responsible to review the diversion wall design and augment its hydraulic barrier properties to meet the specifications requirements for control of water and excavation dewatering.

To meet the structural design criteria and simplify construction, the diversion wall was designed as a cantilever wall without lateral bracing. The wall was designed using traditional methods for cantilever wall design and a finite element analysis performed to evaluate wall deformations at each construction stage. The major stages of construction analyzed included:
• Stage I excavation and dewatering south of the wall,
• Fill placement against the south side of the wall,
• Spillway slab construction (south of wall)
• Spillway endwall construction
• Rebound of groundwater after dewatering is completed
• Excavation for Stage II on the north side of the wall
• Activation of the new spillway while the Stage II excavation north of the wall is at its maximum depth (worst case loading)

To control lateral deformations, the diversion wall design included tying the diversion wall to the new spillway slab. The structural connection would restrict horizontal movement of the wall but allow differential vertical movements.

The final design configuration for the diversion wall included 25 soldier piles (H-piles) on 7 to 10 feet centers depending on the depth to rock. Piles were to be embedded 12 feet into rock and grouted in place. Six additional soldier piles were fastened to the concrete stilling basin slab of the existing spillway and diagonally braced. The 1 inch-thick steel plate lagging was to be extended at least five feet below the excavation or to refusal then welded to the H-piles. All steel was specified to be 50 ksi grade. Total wall height ranged from 12 to 20 feet. A profile view of the diversion wall is shown in Figure 5.

Figure 5. Profile of Stage 1 diversion wall showing soldier piles and steel lagging.

The calculated deformation of the top of the diversion wall was 1.6 inches after placing fill. The calculated additional deformation due to weight of structure and water flowing though spillway during Stage II was 0.4 inches and this deformation would occur about at the mid-height location of the wall. The deformation profiles are shown in Figure 6.
The construction of the diversion wall included using a cluster drill with three 8-inch diameter down-the-hole hammers to drill the rock sockets through cased holes. The steel plate lagging was driven into place with a small vibratory hammer. There were areas where the steel plates were not advanced below the excavation subgrade due to rock or where the steel plates were only advanced a few feet below the excavation subgrade. In many of these areas, the contractor either placed a concrete plug at the base of the wall on the diversion channel side of the excavation or attempted to grout the soil/rocks at the base of the steel plates. In November 2009, after heavy rains from the remnants of Hurricane Ida, stream diversion flow was high (estimated to be in excess of 4,000 cfs). After several hours of high differential head between the diversion channel and the excavation, a hydraulic failure (blowout) below the diversion wall occurred. The blowout resulted in the partially completed excavation filling with water.

After the stream flows subsided, the area where the blowout occurred was excavated. The steel plate in this area had extended about 2 to 3 feet below the excavation subgrade and refused on fractured and weathered rock. The area was excavated to sound rock and the weathering profile was observed to be very irregular. Figure 7 shows a photograph of the excavation along the base of the diversion wall that shows the variability in the rock surface. The soil profile and weathered rock in about a 3-foot wide strip on both sides of the diversion wall was excavated to expose hard rock. These strips were cleaned of loose material and were filled with lean concrete to at least 2 feet above the bottom of the steel plate lagging. The flooding and repairs resulted in a two week delay of the construction schedule.
The performance of the stream diversion wall since the remedial work was performed has been excellent. The project has had several storms that required large reservoir releases through the dam that created large differential heads between the diversion channel and the Stage 1 excavation with little leakage. The completed wall is shown on Figure 8.

The Contractor has performed some limited deformation monitoring of the soldier piles and the results of the deformation monitoring suggest that deformations have been small and within the precision and accuracy of the surveys. The better than expected deformation behavior of the diversion wall may be the result of the use of Cement Modified Soil for the spillway foundation.

**STAGE 1 EXCAVATION AND DEWATERING**

The Stage 1 excavation required the removal of fill and alluvial soils from below the footprint of the labyrinth spillway and south earth embankment. This resulted in an excavation about 450 feet by 200 feet at its base and extending about 20 to 25 feet below groundwater and being 25 to 30 feet deep. The Engineer established excavation contours and dewatering requirements based on the following design criteria:

- The reservoir will remain full.
- The calculated factor of safety of the excavation slopes should be 1.3 or higher.
- The excavation and dewatering system should be instrumented.

Borings performed through the embankment did not encounter alluvial soils. However, one of the borings performed at the toe of the embankment encountered about five feet of sandy alluvial soils. Observation wells installed at toe of the dam showed artesian foundation pressures at the toe of the dam. The blanket drain for the existing dam was to be intercepted, so the potential of excavating saturated drain fill was also a concern. The following excavation and dewatering criteria were established during design:
Excavation slopes shall be 2H:1V or flatter.
Dewatering systems shall pre-drain the soils such that the water table is drawn down to at least 10 feet below excavation slopes.
The base of the excavation shall remain dry with water being drawn down three feet below the excavation grade.

A section through the existing embankment and excavation surface is shown on Figure 9.

The excavation and dewatering systems were monitored with a series of inclinometers, vibrating wire piezometers, and observation wells. The observation wells installed during the design phase were maintained and read until they were either excavated or until the final excavation grade was reached and the performance of the dewatering system had been confirmed. New instrumentation included four inclinometer casings installed in the downstream slope of the existing dam and two inclinometer casings installed downstream of the dam in what would become the downstream slope of the excavation for the new spillway and embankment. The effectiveness of the dewatering system has been monitored with the vibrating wire piezometers in embankment fill, alluvial soils, residual soils and rock foundation materials.

The Contractor was required to design, install, operate and maintain the dewatering system. A preliminary concept for dewatering developed by the Engineer included:

- a two-stage dewatering system of closely spaced wells or well points,
- a series of groundwater collection trenches at the toe of the excavation slopes and in the center of the excavation, and
- a series of wells at the north end of the excavation near the Stage 1 diversion wall and downstream cofferdam to reduce upward gradients adjacent to the diversion wall and downstream cofferdam.

Figure 10 shows the excavation contours as designed, the location of geotechnical instrumentation, and the dewatering concept developed by the Engineer.
The contractor followed the engineer’s concept except for deleting the dewatering wells along the Stage 1 diversion wall, and not connecting the dewatering manifolds across the far end of the excavation. The final dewatering plan included a two-stage well point system, dewatering trenches at the toe of the excavation and local drilled sumps as needed. The well points were installed in drilled or jetted holes. An 8-inch diameter jet rig was used to install the downstream excavation slope Stage 1 well points. The Stage 2 well point installed in the downstream slope and the well points installed in the embankment dam were installed in 8-inch diameter drilled holes. The annular spaces between the well points and the drill holes were filled with filter sand and a surface seal of bentonite was placed at the ground surface. Figure 11 shows the installed well points and manifold along the downstream slope of the existing embankment.

Figure 10. Plan of Dewatering Concept and Geotechnical Instrumentation

Figure 11. Dewatering well points and manifold on existing embankment.
The well points were effective in dewatering the sandy alluvium. However, after about 10 to 15 feet of excavation, the center of the excavation still had relatively high groundwater levels and continued excavation could not be performed in accordance with the specifications. The Contractor requested, and received approval, to excavate a temporary dewatering trench in the center of the excavation to enhance pre-draining of excavation soils, as shown on Figure 12. The bottom of the trench drain was about 1 to 2 feet above the final excavation grade so as not to affect the final foundation surface.

The dewatering trench allowed the remainder of the excavation area to drain, allowing the planned upstream and downstream toe drains to be installed. The upstream toe drain was installed with a trenching machine and the downstream toe drain was installed by open excavation.

The toe drains lowered water levels generally to about 2 to 3 feet below final excavation grade except in a few areas where localized seeps were observed. These areas were dewatered by installing filtered sumps at each seep using a 2-foot diameter auger mounted to a mini-excavator. After a final foundation subgrade clean-up and few feet of spillway foundation fill was placed and compacted, the local sumps were abandoned by grouting them in-place.

**SPILLWAY FOUNDATION DESIGN AND CONSTRUCTION**

The subsurface conditions below the proposed labyrinth spillway consisted of fill overlying loose and soft alluvial soils, dense residual soil, and rock. The fill soils and alluvial soils were not suitable for spillway support due to settlement and seepage concerns. The design of the foundation required the fill soils and alluvial soils to be excavated and replaced with compacted fill. Silty sands (SM) and Sandy Silts (ML) were specified as suitable fill soils for the spillway foundation.

In order to control settlement, the spillway foundation fill was specified to be compacted to 95% of its maximum dry density as determined by ASTM D1557 (Modified Proctor). Settlement analyses showed that by increasing the compaction criteria from 95% of the standard proctor maximum dry density to 95% of the modified proctor maximum dry density, the settlement of the spillway slab would be reduced from slightly more than 1-1/2 inch to about ¾ inch. The endwalls of the spillway support 35 feet of fill and were estimated to settle up to about 1-1/2 inches (about twice the settlement of the spillway)
with the spillway foundation fill compacted to 95% of its modified Proctor maximum dry density.

The borrow investigation was performed in the fall of 2006 and the summer of 2007. North Carolina was in the middle of an extended dry period in the summer of 2007. Natural moisture contents of potential borrow samples collected in the fall of 2006 were 3% to 7% above their optimum moisture contents. Samples collected in the summer of 2007 were 1% to 5% above their optimum moisture content. Based on the natural moisture contents observed in 2006 and during the drought of 2007, it was expected that compacting the spillway foundation fill to achieve an in-place density higher than 95% of its maximum dry density would require the contractor to dry the soils to within about 1 to 2% of optimum moisture content.

During the fall of 2009, the contractor made several attempts to construct the required spillway foundation test fill to show that their procedures to excavate, handle, and dry the spillway foundation fill would be adequate to achieve the specified in-place density. The contractor had difficulty drying the foundation fill soils excavated from the portion of the borrow area which had moisture contents about 5% to 6% above their optimum moistures. The contractor considered several options to dry the fill soils including:

- Air drying,
- Mixing with hydrated lime, and
- Mixing with quick lime

Quick lime was the most effective of the three procedures attempted, but required extensive mixing, adding an expensive admixture, and did not reduce the compactive effort required to achieve the specified density of the fill. Additional discussions between the Engineer and the Contractor resulted in the contractor exploring the feasibility of adding cement to dry the borrow area soils and make the compacted spillway fill stiffer than it would otherwise be. In this case, the compaction criteria could be reduced to 95% of the standard proctor maximum dry density.

The Contractor performed preliminary mix design testing program for cement modified soil (CMS) to be used as spillway foundation fill. Based on discussions between the Engineer and the contractor’s geotechnical consultant, the cement content was varied from 2% to 6% of the soils dry weight with a final design moisture content of at least 2% above the soil’s optimum moisture content. Test samples were remolded to 95% of the soil’s standard Proctor maximum dry density in the laboratory and cured for 7 days. Both unconfined compression testing and one-dimensional consolidation tests were performed on the samples after the 7-day curing period.

The results of the lab testing indicated the following:

- CMS samples with 2% cement were more compressible than the specified fill
- CMS samples with 4% cement were as compressible the specified fill.
- CMS samples with 6% cement were less compressible than the specified fill.
Based on the results of the laboratory mix design program, the contractor constructed a test fill of CMS using a cement addition rate of 5%. The test fill was constructed in the borrow area and consisted of placing, and compacting three nominally 1-foot-thick lifts of borrow area soils in an area that measured about 50 feet by 200 feet. The cement was spread and mixed using equipment commonly used to mix cement treated aggregate base courses for roadways in North Carolina. The following equipment was used to construct the test fill.

- Cat D-6 bulldozer
- Cement spreader truck (Figure 13)
- CAT RM500 Soil Stabilizer/Mixer (Figure 14)
- CAT 815 Soil Compactor
- Ride-On Single Drum Padded Foot Vibratory Roller

Field testing included measuring the density and moisture of each 12-inch thick loose lift of spillway fill, the cement application rate per square yard, and the density of each lift of CMS after spreading, mixing, and compacting was performed. The Engineer collected two 10-inch Shelby tube samples from each lift of fill for laboratory testing that included unconsolidated undrained triaxial compression (UU) tests, grain size distribution tests, and Atterberg limits tests. The UU tests were performed on samples that were cured for 14 days. The fill soils used to construct the CMS test fill were visually classified as sandy silt with a low plasticity index and were considered to be on the upper end of allowable compressibility for spillway foundation fill. Hand-excavated pits were dug to visually check for uniformity of mixing.

During the design phase of the project, finite element models were developed to evaluate the settlement of the labyrinth spillway and endwalls. Based on the expected soil behavior of the CMS, additional finite element analyses were performed to evaluate the
performance of CMS as spillway foundation fill. The finite element analyses modeled the CMS as a mohr-coulomb material and the elastic modulus of the CMS was varied. The result of the triaxial compression tests were used to estimate an expected range of elastic modulus. Based on the results of the laboratory testing and the finite element analyses, it was concluded that a 5% cement addition rate would produce a CMS foundation with an average elastic modulus of about 1,000 ksf, or higher, and spillway settlement would be about 1/3 of the settlement originally calculated for soil-only fill.

**BENEFITS OF THE CMS FOUNDATION MATERIALS**

The CMS material has proven to be beneficial to the entire construction process. Even though the site borrow soils were wet of optimum, the characteristics of cement required water to be added during placement. Therefore, borrow material could be used without having to be dried. The addition of the cement also lowered the uncertainty in the placement schedule, as the process became independent of the need for dry weather and independent of the variability of the in-situ moisture content of the borrow soils. Overall, the process became systematic and predictable - ideal characteristics for components of a major construction project.

The CMS had other ancillary benefits. For example, within two weeks of starting the CMS placement a heavy rainfall submerged the site under two feet of water. After pumping out the surface water and stripping the top few inches of saturated material, placement resumed after only two days. Under more normal conditions, the CMS foundation was found to be very resistant to damage by weather. Typical rain showers had no effect on construction activities on top of the CMS material and even heavier amounts of rain usually required only minimal amounts of material to be stripped from the surface. The surface was also resistant to damage by the movement of heavy equipment, so minimal re-surfacing or other maintenance activities were required.

The CMS also provided significant benefits during placement of the concrete spillway slabs. The original design required earth fill to be placed to rough grade elevations and, after concrete was placed, cured and the forms removed, the area was to be backfilled and compacted to final subgrade. Due to the properties of the CMS, the contractor was able to place the fill to final subgrade elevations and excavate neatline vertical faces to serve as form supports. Some of these vertical faces were 6 feet in height, which without the CMS would have required sloping or excavation support. The contractor's engineer determined that the cement in the soil provided adequate support for the excavation. These vertical faces allowed minimizing the amount of formwork support required, and alleviated the need for backfilling and compaction equipment working in the concrete placement area. This aspect alone reduced the complexity of managing the jobsite and reduced the number of pieces of equipment and personnel required.

Settlement survey monuments were installed in many of the spillway slabs as the cement started to cure. Repeated measurements of these monuments has indicated virtually no settlement or heaving as adjacent slabs are poured. Overall, no downside to the use of a CMS foundation has yet to be discovered.
CONCRETE DESIGN

Specifications for the cast in place contract for the labyrinth structure call for a 50% fly ash and 50% cement mixture of 340 lbs/cy each, having a 0.38 water/cement ratio to produce a 5-inch slump, and 3.5% to 6.5% air content. The Contractor was responsible for the initial mix design. The concrete supplier made several test mixes for consideration and recommended a mix of 320 lbs/cy of cement and equal amount of fly ash, a water/cement ratio of .45, a slump range of 6-9 inches with high range water reducer and set retarder, and an air content of 3.5 to 6.5%. This particular mix provided for the following average strengths: 3-day 2490 psi, 7-day 3130 psi, 14-day 4070psi, 28 day 4780 psi, and 56-day 6140 psi. This mix was approved for construction, and concrete performance has been above average in terms of placement, curing temperatures, and finish quality. Figures 15 and 16 present views of spillway construction.

Figure 15. View looking northeast from upstream of new spillway showing southern endwall and partially constructed labyrinth sidewalls
SUMMARY

Challenges to date on the Lake Townsend Dam Replacement Project were primarily related to excavation stability and identification of suitable backfill for the labyrinth spillway. Enhanced dewatering, and monitoring of groundwater levels and slope stability during excavation provided “early warning” of potential substantive ground movements. No movements were identified that posed a risk to the existing dam or foundation. The use of cement-modified soil as foundation backfill resolved issues with borrow materials being too wet to compact effectively. The use of CMS had a number of benefits, including reducing schedule and construction risks by becoming more independent of the weather. Ancillary benefits included providing an excellent, weather resistant foundation surface from which to work, and the ability to carve vertical walls for concrete forming that required no shoring. Construction has proceeded on schedule.
REFERENCES


Simons, Jim, 1985, NC DENR Memorandum to Doug Miller, Subject: Lake Townsend Dam, Lake Brandt Dam, Lake Higgins, Lake Jeanette Dam, September 25, 1989.

Thomas, Nathan O., 1980, Lake Townsend Dam, Guilford County, North Carolina, Hydrologic and Hydraulic Analysis, Analysis requested by Soil and Material Engineering, Inc. in Connection with the Phase I Dam Safety Inspections, April 8, 1980.