Fayette
COUNTY

"WHERE QUALITY
IS A LIFESTYLE"

PURCHASING DEPARTMENT
140 STONEWALL AVENUE WEST, STE 204
FAYETTEVILLE, GEORGIA 30214
PHONE: 770-305-5420
www.fayettecountyga.gov

July 21, 2016

**Subject: #1143-B, Emerald Lake Dam Rehabilitation – Addendum #1**

Dear Gentlemen/Ladies:

Included herein is additional information and clarification for the above referenced invitation for bids. Please consider all of the information when preparing your bid. Please acknowledge receipt of this addendum by signing in the space provided and returning it with your bid.

1. **Bidding** – Contractors shall price and bid the project as defined in the plans and specifications. Alternate bids, other than those specifically requested in the bid schedule, will not be accepted.

   Upon award of contract to the lowest, qualified bidder, the Contractor may submit a Request for Information (RFI) for consideration of alternative construction methods. All RFIs will be evaluated by the Engineer and Fayette County and a response will be provided, in writing, to the Contractor. In some cases an Item of Cost (IOC) may be requested.

2. **Utility Relocation** – Permanent utility lines shall be relocated by the Utility Companies or their designated subcontractor at no cost to the Contractor.

   The Contractor is responsible for scheduling and coordinating all work. This includes ensuring new lines are adequately protected from construction activities. The Contractor is also responsible for installation of temporary utilities, if needed; repairs to damaged utilities resulting from Contractor activities; and additional relocations required as a result of poor planning and/or scheduling by the Contractor.

   A revised sheet C-104 is provided with this Addendum showing all utilities placed below (i.e., downstream of) the outlet. The lines have also been moved outside the "undercut limits for spillway."

3. **Fayette County Staging Area** – Fayette County's Road Department can provide approximately 1.5 acres for storage of waste/excess material as noted in the specifications. If the Contractor elects to dispose of material at this location they shall provide (and maintain thru the duration of the project) one row of Type C silt fence around the staging area(s). The Contractor shall also be responsible for pushing-up and stockpiling the material to maximize storage within the allocated area. Material left idle for 14 days or more shall be stabilized with temporary grassing (Ds2). All material left on site at the close of the project shall become the property of Fayette County.

   The Road Department will provide and maintain the construction entrance to the Staging Area.

4. **Emerald Lake Drive and Other County Roads** – For roadways outside the project limits, the Fayette County Road Department will be responsible for addressing potholes or other road damage resulting from normal wear and tear associated with truck traffic.

5. **Stand Pipe and Valve** – The existing structure has a standpipe and valve. Their condition is unknown. The Contractor may use this as part of their water management plan but no allowances shall be provided if any part of the standpipe or valve are unusable or fail during operation. Fayette County makes no promises that it can be used to drain the lake.

6. **Geotechnical Report** – Included with this Addendum is a copy of the project's geotechnical report. This is provided for informational purposes only.

7. **Decorative Light Pole Storage** – Once removed by the Contractor the poles may be stored at the Public Works facility located at 115 McDonough Road, Fayetteville, GA 30214. The Contractor shall be responsible for delivering and unloading the poles to Public Works. Once unloaded, however, all responsibility for storage and care shall become the responsibility of Fayette County. Alternatively, the poles may be stored on site and remain the responsibly of the Contractor for the duration of the project.

8. **Pricing Sheet –** A revised pricing sheet is attached which includes a line item for C33 Sand material and placement.

Received by________________________ Company________________________

**Note: If this addendum is not returned to the Fayette County Purchasing Department or if it is returned not signed, all responders shall still be responsible for the requirements of this addendum and the specifications or changes herein.**

The opening date for this invitation for bids has not changed. The opening date will be 3:00pm, Tuesday, August 2, 2016. Bids must be received in the Purchasing Department at the address above in Suite 204 on or before the opening date and time.

## PRICING SHEET

BIDDER agrees to perform all the work described in the CONTRACT DOCUMENTS, within the time set forth therein, and for the following unit prices or lump sum values:

BIDS shall include sales tax and all other applicable taxes and fees.

| BID SCHEDULE | | | | | |
|---|---|---|---|---|---|
| **No.** | **Item Description** | **Unit** | **Est. Quant.** | **Unit Price** | **Total Price** |
| | BASE BID | | | | |
| 1 | Completion of all work indicated in the Plans and Specification, with exception of items noted below. | LS | 1 | $ | $ |
| 2 | Rock Excavation (Section 01 22 00 - 1.2) | CY | 10 | $ | $ |
| 3 | Earth Work - Additional Alluvial Excavation and Backfilling (Section 01 22 00 - 1.3 and Sheet S-307) | CY | 1,000 | $ | $ |
| 4 | Additional Rip Rap (Section 31 37 00) | ton | 30 | $ | $ |
| 5 | Additional C33 Sand – material and placement | CY | 50 | $ | $ |
| 6 | Allowance per Special Conditions (see Specifications). | LS | 1 | $100,000 | $100,000 |
| | **Base Bid Total** | | | | $________ |
| | ALTERNATE BID ITEMS | | | | |
| 7 | Optional upgrade to industrial-grade ornamental steel fence (Sheet S-300) | LS | 1 | $ | $ |
| 8 | Optional Ashlar Stone Finish to parapet walls. Both walls, both sides. (Sheet S-310) | LS | 1 | $ | $ |

BIDDER hereby agrees to commence WORK under this contract on or before a date to be specified in the NOTICE TO PROCEED and to fully complete the PROJECT within 430 consecutive calendar days thereafter. BIDDER further agrees to pay as liquidated damages, the sum of $250.00 for each consecutive day thereafter as provided in the Terms and Conditions.

**GDOT Prequalification/Certification No.** ________________________________

**COMPANY NAME:** ________________________________

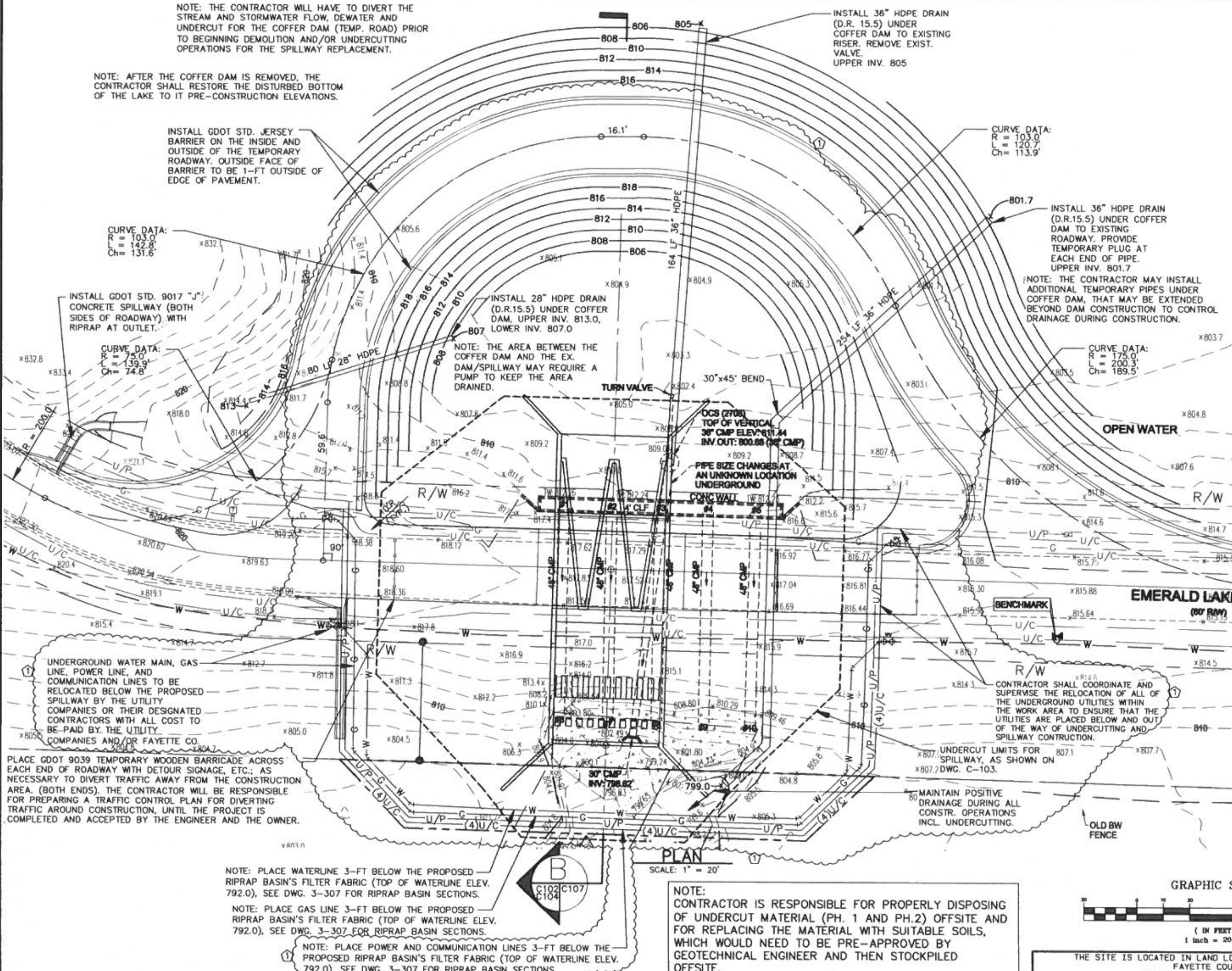
Thank you for your attention to this matter.

Sincerely,

Ted L. Burgess
Director of Purchasing

TLB/tcb

Attachment




PLACE GDOT 9039 TEMPORARY WOODEN BARRICADE ACROSS EACH END OF ROADWAY WITH DETOUR SIGNAGE, ETC.; AS NECESSARY TO DIVERT TRAFFIC AWAY FROM THE CONSTRUCTION AREA. (BOTH ENDS). THE CONTRACTOR WILL BE RESPONSIBLE FOR PREPARING A TRAFFIC CONTROL PLAN FOR DIVERTING TRAFFIC AROUND CONSTRUCTION, UNTIL THE PROJECT IS COMPLETED AND ACCEPTED BY THE ENGINEER AND THE OWNER.

NOTE: PLACE WATERLINE 3-FT BELOW THE PROPOSED RIPRAP BASIN'S FILTER FABRIC (TOP OF WATERLINE ELEV. 792.0), SEE DWG. 3-307 FOR RIPRAP BASIN SECTIONS.

NOTE: PLACE GAS LINE 3-FT BELOW THE PROPOSED RIPRAP BASIN'S FILTER FABRIC (TOP OF WATERLINE ELEV. 792.0), SEE DWG. 3-307 FOR RIPRAP BASIN SECTIONS.

1 NOTE: PLACE POWER AND COMMUNICATION LINES 3-FT BELOW THE PROPOSED RIPRAP BASIN'S FILTER FABRIC (TOP OF WATERLINE ELEV. 792.0), SEE DWG. 3-307 FOR RIPRAP BASIN SECTIONS.

NOTE:
CONTRACTOR IS RESPONSIBLE FOR PROPERLY DISPOSING OF UNDERCUT MATERIAL (PH. 1 AND PH.2) OFFSITE AND FOR REPLACING THE MATERIAL WITH SUITABLE SOILS, WHICH WOULD NEED TO BE PRE-APPROVED BY GEOTECHNICAL ENGINEER AND THEN STOCKPILED OFFSITE.

## TEMPORARY COFFER DAM NOTES:

1. THE COFFER DAM & ACCESS ROAD ARRANGEMENT/ALIGNMENT SHOWN ON THIS DRAWING IS CONCEPTUAL ONLY. THE CONTRACTOR IS RESPONSIBLE FOR THE COMPLETE DESIGN OF THE TEMPORARY COFFER DAM INCLUDING LAYOUT AND ALIGNMENT AS NECESSARY TO PROTECT THE WORK THROUGHOUT THE CONSTRUCTION PERIOD AND PROVIDE FOR CONTROL OF TRAFFIC.
2. THE COFFER DAM/ACCESS ROAD MUST BE BUILT TO A MINIMUM ELEVATION OF 818 FEET AND PROVIDE MINIMUM CROSS-SECTION. SEE C-107, DETAIL A, AND C-110 TEMPORARY ASPHALT DRIVE DETAIL.
2. THE CONTRACTOR IS RESPONSIBLE FOR PROTECTING THE WORK FROM DAMAGE RESULTING FROM STORMWATER. ANY DAMAGE RESULTING FROM ANY RAINFALL EVENT OR EVENTS REGARDLESS OF THE AMOUNT OR DURATION OF THE RAINFALL SHALL BE REPAIRED TO THE SATISFACTION OF THE ENGINEER BY THE CONTRACTOR WITHOUT ANY ADDITIONAL COMPENSATION.
4. THE LAKE SHALL GENERALLY BE MAINTAINED BY THE CONTRACTOR IN A DRAINED CONDITION BY KEEPING THE LOW LEVEL DRAINS FULLY OPERATIONAL. THE WATER LEVEL IN THE LAKE WILL RISE SUBSTANTIALLY DUE TO RAINFALL EVENTS. SEE THE TABLE ON THIS DRAWING FOR PROJECTED WATER SURFACE ELEVATIONS ASSOCIATED WITH VARIOUS STORM EVENTS.
5. THE REQUIREMENT TO PERFORM ALL CONSTRUCTION "IN THE DRY" SHALL NOT BE OBVIATED BY AN ELEVATED WATER LEVEL IN THE RESERVOIR.

NOTE:
TEMPORARY ROADWAY AND DRAINAGE MUST BE IN PLACE PRIOR TO ANY DEMOLITION AND/OR EXCAVATION AND UNDERCUTTING FOR THE LABYRINTH WEIR CONSTRUCTION.

| STORM | INCOMING FLOW (CFS) | EXISTING OUTFLOW (CFS) | EXISTING W.S. ELEV. (FT) | TEMP. DRAIN FLOW (CFS) | TEMP. DRAIN W.S. ELEV. (FT) | PROPOSED OUTFLOW (CFS) | PROPOSED W.S. ELEV. (FT) |
|---|---|---|---|---|---|---|---|
| 2 YEAR | 547 | 425 | 813.5 | 124 | 809.3 | 495 | 813.1 |
| 5 YEAR | 859 | 588 | 814.2 | 144 | 811.9 | 790 | 813.4 |
| 10 YEAR | 976 | 616 | 814.6 | 152 | 812.8 | 900 | 813.5 |
| 25 YEAR | 1,470 | 1,095 | 815.7 ✡ | 359 | 815.5 ✡ ▽ | 1,356 | 814.0 |
| 50 YEAR | 1,654 | 1,324 | 815.9 ✡ | 580 | 815.7 ✡ ▽ | 1,519 | 814.2 |
| 100 YEAR | 1,935 | 1,658 | 816.2 ✡ | 944 | 816.0 ✡ ▽ | 1,772 | 814.5 |
| 1/4 PMP | 3,124 | 2,907 | 816.8 ✡ | N/A | N/A | 2,717 | 815.8 |

✡ DENOTES OVERTOPPPING DAM/ROAD

NOTE: PROPOSED FLOWS ARE BASED ON THE LABYRINTH AND WALL BEING CONSTRUCTED. EXISTING FLOWS ARE BASED ON THE EXISTING BOX AND 5-48" PIPES AND THE EXISTING 30" LOW LEVEL (CMP) DRAIN. TEMPORARY FLOWS ARE BASED ON 30" CMP LOW LEVEL DRAIN AND 36" HDPE DRAIN THRU COFFER DAM AND DAM/ROAD

NOTE:
THE TEMPORARY LOW LEVEL (30" HDPE) PIPE EXTENSION AND THE TEMPORARY 36' (HDPE) PIPE PROVIDES STORMWATER CONTROL FOR STORMS UP TO THE 10-YR STORM EVENT WITH THE LAKE FULLY DRAINED (DURING CONSTRUCTION). THE CONTRACTOR IS RESPONSIBLE FOR COFFER DAM DESIGN INCLUDING TEMPORARY DRAINAGE DESIGN FOR ALL STORMS EXCEEDING THE 10-YR STORM EVENT. THIS PROJECT MAY REQUIRE THE USE OF PUMPS IN ORDER TO PREVENT THE DAM FROM OVERTOPPING DURING STORM EVENTS.

NOTE:
TEMPORARY COFFER DAM TO PROTECT WORK AREA DURING CONSTRUCTION. COFFER DAM SHALL BE DESIGNED BY GA PE. DESIGN CALCULATIONS AND SHOP DWG SHALL BE SUBMITTED TO THE ENGINEER FOR REVIEW. TEMPORARY COFFER DAM SHALL BE REMOVED AFTER CONSTRUCTION IS COMPLETED.

GRAPHIC SCALE
( IN FEET )
1 inch = 20 ft.

811 Know what's below. Call before you dig.

PRIMARY PERMITTEE
OWNER:
FAYETTE COUNTY
BOARD OF COMMISSIONERS
140 STONEWALL AVENUE WEST
FAYETTEVILLE, GEORGIA 30214
CONTACT: MR. PHIL MALLON
PHONE: (770) 313-9855

24 HR. CONTACT PERSON
PHIL MALLON
(770) 313-9855

FOR BID PURPOSES ONLY
NOT FOR CONSTRUCTION

THE SITE IS LOCATED IN LAND LOTS 73 & 74 IN DISTRICT 5, FAYETTE COUNTY, GA
LOCATED ON EMERALD LAKE DRIVE.

| NO. | REVISIONS | MADE | CKD. | DATE |
|---|---|---|---|---|
| 0 | 1 REVISED UTILITY RELOCATION LAYOUT, REMOVED TEMPORARY UTILITIES FROM TEMP. COFFER DAM/ROAD, REV. UTILITY NOTES | JH | MLW | 7/20/16 |

| | | |
|---|---|---|
| SCALE: | 1" = 20' | DATE |
| DESIGN BY: | JMH | 02/16 |
| DRAWN BY: | JMH | 02/16 |
| CHECKED BY: | MLW | 02/16 |
| SUPV. BY: | | |

MARTIN L. WALDEN, PE
LEVEL 2 CERTIFICATION # 45843
EXPIRATION DATE: 09/25/18

# EMERALD LAKE DAM REHAB
## DAM IMPROVEMENTS
### TEMPORARY ROADWAY PLAN
## FAYETTE COUNTY

WALDEN, ASHWORTH & ASSOCIATES, INC.
CONSULTING ENGINEERS
P.O. BOX 6482
MARIETTA, GEORGIA 30065
(770) 956 - 7879

| JOB NO. | DRAWING NUMBER | REV. NO. |
|---|---|---|
| 3400500 | 34005-C-104 | 0 |

# *Emerald Lake Dam*
# *Emerald Lake Drive*
# *Fayetteville, Fayette County, Georgia*
# *PGC Project No. 114019*

Prepared For:

***Walden, Ashworth and Associates, Inc.***

*P.O. Box 6462*
*Marietta, Georgia 30065*

Prepared By:


**PIEDMONT**
GEOTECHNICAL CONSULTANTS, INC. PGC

3000 Northfield Place, Suite 1100
Roswell, Georgia 30076


January 5, 2015

3000 Northfield Place * Suite 1100 * Roswell, GA 30076
(770) 752-9205 * FAX (770) 752-0890

January 5, 2015

**Walden, Ashworth and Associates, Inc.**
P.O. Box 6462
Marietta, Georgia 30065

Attention: Mr. Marty Walden. P.E.

Subject: **Report of Subsurface Exploration and Geotechnical Engineering Evaluation**
Emerald Lake Dam
Emerald Lake Drive
Fayetteville, Fayette County, Georgia
PGC Project No. 114019

Dear Marty:

Piedmont Geotechnical Consultants, Inc. has completed our geotechnical engineering evaluation of the subject project, and is reporting our findings herein. The study was outlined in our proposal No. P12546 to you dated December 19, 2012. The following paragraphs describe our understanding of the project, evaluation procedures used, our findings, and geotechnical engineering conclusions and recommendations.

## PROJECT INFORMATION

Emerald Lake Dam is an existing earthen dam, approximately 600 feet long and a maximum of 20 feet in height. The dam and lake were constructed between 1982 and 1993, likely during the development of the Woodlands Subdivision. The dam also serves as the only access road to the single family homes located south of the lake. Based on the information provided, the dam's spillway was re-constructed about 1997, when five (5) 48-inch diameter corrugated metal pipes and straight concrete weir wall were installed near the south end of the embankment, generally aligning with the original stream channel. The concrete weir appears to be set at the same elevation as the original corrugated riser pipe and all of these pipes appear to flow when the lake is at its normal level. It is our understanding the dam is currently classified as a Category II structure by the Georgia Safe Dams Program. Based on our recent discussions with Mr. Tom Woosley, P.E., Program Manager, the dam is scheduled to be re-evaluated by the State to determine if a classification change is required. Review of aerial photographs show a residential development immediately downstream of the dam with houses in reasonable proximity to the stream channel.

We understand that the proposed rehabilitation design is to include flattened upstream and downstream slopes, a new labyrinth weir spillway and stilling basin, a seepage control drain system and a bridge over the spillway. We also understand that a short wall will be constructed along the upstream crest to effectively raise the top of dam and provide the required storage for the design storm event.

## EVALUATION PROCEDURES

To evaluate the embankment dam's internal composition and the underlying foundation conditions, primarily for the proposed spillway design and construction, four (4) soil test borings were drilled to depths of 25 to 30 feet below the existing crest of the dam. Observation wells were installed in three of the borings with screen depths ranging from 20 to 30 feet below the existing grade. To evaluate the subsurface conditions intermediate of the 48 inch corrugated metal pipes four hand auger borings with portable dynamic cone penetrometer testing were performed to depths of 9 to 15 feet below the existing grades. The boring locations, as shown on Figure 1: Site and Boring Location Plan in the Appendix, should be considered approximate.

Drilling, sampling, and Standard Penetration Testing were performed in general accordance with ASTM D-1586. All four borings were advanced using mud-rotary drilling techniques, which involved pumping a thick bentonite/water drilling fluid through the hollow steel drilling rods and tri-cone rotary bit. As the bit was rotated and forced downward, the drilling fluid circulates the soil cuttings from around the bit to the surface where the heavier soil solids settle out in a large collection tub. Because the drilling fluid is heavier than water and a positive head difference is maintained, the fluid stabilizes the open borehole and minimizes collapse due to water intrusion. At regular intervals, in all soil test borings, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-barrel sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is designated the "Standard Penetration Resistance". The penetration resistance, when properly evaluated, is an index of the soil's strength, density and ability to support loads. Because the sampler may be damaged by driving it one foot into very hard or dense soils, the sampler may only be driven a few inches into such materials and the penetration resistance is expressed as the number of blows versus the depth of penetration; e.g., 100/3 inches, 50/1 inch, etc.

Soil samples recovered during the drilling process were classified by a geotechnical engineer using visual/manual procedures in general accordance with the Unified Soil Classification System (USCS). Detailed descriptions of the materials encountered at each boring location, along with a graphical representation of the Standard Penetration Test results, are shown on the Soil Boring Records in the Appendix.

Three (3) of the soil test borings were converted to temporary observation wells. Once each boring reached the planned termination depth, the borehole was over reamed with a larger tri-cone rotary bit for installation of the well materials. The purpose of these wells was to determine the stabilized water levels (phreatic surface) within the embankment. In each open borehole a 5 or 10 foot section of 0.010 inch slotted, 2 inch diameter, PVC well screen attached to solid PVC riser pipe was placed to the desired well depth. The screened portion of the well was backfilled using filter sand to approximately 1 to 2 feet above the screen height, followed by 1 foot of bentonite chips. The remaining depth was backfilled with a bentonite/cement grout. A flush-mounted (manhole type) well cover was set in the asphalt at the location of boring B-1. The two wells installed in the downstream road shoulder were protected with a standup type metal riser.

The hand auger borings were advanced by manually twisting a sharpened steel auger bucket into the ground. The soils encountered during the augering process were classified in general accordance with the Unified Soil Classification System (USCS). At regular intervals, the auger was removed, and the soil consistency was measured with a dynamic cone penetrometer (DCP). The conical point is first seated to penetrate any loose cuttings and then driven an additional 1-¾ inches with blows of a 15-pound hammer falling 20 inches. The number of hammer blows required to achieve this penetration is recorded as an index to the soil's strength and consistency. Please refer to the Summary of Hand Auger Borings in the Appendix.

## SITE OBSERVATIONS

During the course of this field study, performed late February to early March 2014, PGC engineers Jonathan Sharpe, E.I.T. and Craig Robinson, P.E. visited the site and performed detailed observation of the dam's external condition. While on-site at the various times, their following observations were noted. Physical directions are referenced while facing downstream (lake to your back).

1. The embankment dam appears to be statically stable, with moderately steep slopes and very wide crest section (approximately 50 to 60 feet with 2 lane roadway). The crest of the dam is straight, sloped slightly upstream and approximately 600 feet long. The overall height of the dam is estimated to be 20 feet. The dam serves as the only access road to the single family homes located south of the lake. No dips or ruts were observed in the asphalt that would indicate poor shallow subgrade conditions, though there are some low spots/dips in the shoulder on both the upstream and downstream sides of the road that appear to coincide with the spillway pipes.

2. The upstream slope is generally vegetated with low growing grass, varies from 1.5(H):1(V) to 2.5(H):1(V) configuration and is somewhat irregular. Rip-rap wave protection is present at and above the normal lake level along the upstream slope.

3. The downstream slope is overgrown with small trees, briars and underbrush. The slope face is somewhat irregular and moderately steep. The general configuration is approximately 2(H):1(V) in the general area of the existing spillway. Beyond the spillway the slope is flatter. Above the right-most spillway pipe, signs of loss of ground were observed. Other irregularities on the slope (beneath and surrounding the pipes) appeared to be caused by erosion over the years from around the pipes.

4. The primary spillway is a 36 inch corrugated metal riser connected to a 30 inch corrugated metal low level pipe (measured at the downstream end). The flow observed at the downstream end of the low level pipe visually appears to be greater than the amount of water entering the riser pipe. During the course of this study, the riser pipe was observed to be leaning several degrees to one side. It is suspected the weight of ice predominantly on one side of the riser pipe over-stressed the pipe and caused damage to the structure. Due to the leaning pipe, the normal pool level has subsequently been lowered. At the beginning of the drilling process the lake was approximately 6 inches below the suspected normal pool. During subsequent visits to the site after the drilling, the lake appeared to be closer to 12 inches below normal pool. We understand that Fayette County has temporarily supported the leaning riser with cables anchored to the shoreline and have blocked flow through the right spillway pipe. We suspect that the low level CMP and riser are part of the original construction and pre-date the secondary spillway system. However, it is possible that the low level pipe has been extended downstream.

5. The existing secondary spillway consists of five 48 inch corrugated metal pipes. The pipes are about 15 years old, about the middle of their designed life. The five spillway pipes were visually inspected. Each pipe exhibits some separation and misalignment at most of the joints. Approximately 35 to 40 feet from the downstream end of the pipes there is a noticeable vertical displacement (settlement) in each pipe. This displacement has resulted in water ponding which appears to be potential seepage since the lake level was lower than the weir elevation and was not flowing through the spillway pipes on the day of our visit. The first joint (from downstream to upstream) of the right most pipe has separated significantly. A hole has formed near the downstream edge of crest at the tree line that appears to be a drop out from soil loss over the southernmost pipe. This hole generally aligns with the joint separation.

6. Flows from the spillway pipes have caused considerable damage to the grouted riprap rock apron. Evidence of seepage around the corrugated metal pipes and soil erosion/piping were observed. No evidence of a seepage control drain system was observed.

7. Several buried utilities (water, power, gas, and communications) were marked prior to our field work and are shown to be buried on both sides of the roadway.

8. The floodplain downstream of the dam is generally flat, broad and moderate to heavily wooded. The stream channel is 10 to 15 feet wide (much narrower than the current spillway system). Approximately 200 feet downstream of the embankment a second stream channel enters the main channel from the left. The stream channel is incised about 8 to 10 feet below the general floodplain level. Orange stained seepage was observed in the stream channel approximately 20 to 30 feet downstream of the low level pipe.

## FINDINGS

The soil test borings were performed along the crest of the dam, two in the upstream edge of asphalt and two in the downstream shoulder, near the tree line. All borings encountered fill materials which were either underlain by alluvium followed by residuum in borings B-2 and B-4, or just residuum in borings B-1 and B-3. All borings were terminated in residual soils. The four hand auger borings were performed between the existing spillway pipes, two upstream of the road and two downstream of the road. Previously placed fill was encountered at each location, with refusal on the weir/headwall structure footing encountered in the two upstream hand auger borings. Each hand auger boring was terminated or refused in the fill materials.

### Soil Test Borings

Soil test borings B-1 and B-3 were performed on the upstream edge of pavement and encountered 3 to 4.5 inches of asphalt and 15 to 16.5 inches of basestone. Beneath the layer of asphalt and basestone the borings encountered previously placed fill to depths of 26 and 19 feet below the ground surface, respectively. Borings B-2 and B-4 were performed in the downstream road shoulder. Previously placed fill was encountered to depths of 16 and 10 feet below the ground surface at these locations, respectively. The fill was classified as silty sands (SM), clayey sands (SC), silty clays (CL), clayey and sandy silts (ML) and appears to have been placed randomly within the embankment, with no obvious attempt to zone the embankment and necessarily use more clayey soils in the central portion of the embankment. Standard penetration test values ranged from 1 to 14 blows per foot (bpf), but typically were less than 10 bpf. Generally, the upper 5 feet of the embankment exhibited higher SPT values; however, the SPT values within the fill zone typically decreased with depth. SPT values less than 8 bpf are often indicative of under compacted soils.

Beneath the previously placed fill in borings B-2 and B-4, 9 to 10 feet of alluvial soils were encountered. The alluvium was described as silty sands (SM) or clayey sands (SC). The alluvium encountered in boring B-4 was of very low consistency (SPT values less than 4 bpf), while alluvium in boring B-2 was sandier and exhibited a higher consistency (SPT values of 10 bpf or greater).

Underlying the previously placed fill in borings B-1 and B-3 and the alluvium in borings B-2 and B-4, residual soils were encountered. The residuum was encountered at depths ranging from 19 to 26 feet below the existing ground surface. Residual soils were classified as silty sands (SM) with SPT values ranging from 7 to 53 bpf.

Stabilized groundwater levels were measured in the three observation wells, at the locations of B-1, B-2 and B-4, at 11 to 15 feet below the ground surface. The phreatic surface depicted by the groundwater levels suggest little to about 4 feet of drop upstream to downstream through the embankment. Groundwater levels fluctuate with changes in climate, rainfall and pool levels and may be different at other times.

**Hand Auger Borings**

Hand auger borings HA-1 and HA-2 were performed on the upstream side of the road, between the edge of pavement and the concrete structure. Boring HA-1 was drilled between the 2nd and 3rd (as numbered from left to right looking downstream) and boring HA-2 was drilled between the 4th and 5th pipes. The upper 2 to 4 feet of fill was found to be moderately well compacted. Below about 4 feet the fill consistency generally decreased in with increased depth. Both borings refused at a depth of 9 feet below the existing ground surface. The soils encountered at the refusal level were very wet. We believe this refusal is on the top of the footing for the concrete weir/headwall structure.

Hand auger borings HA-3 and HA-4 were performed on the downstream side of the road. Boring HA-3 was performed between the 1st and 2nd pipes, approximately 15 feet downstream of the pavement. Boring HA-4 was performed between the 3rd and 4th pipes, about 12 inches downstream of the pavement. Each of these borings encountered fill materials to their termination depths. Boring HA-3 encountered fill with heavy organics beginning at 8.5 feet and continuing to its termination depth of 10 feet. The consistency of the fill encountered in each of these borings was generally poorly to moderately well compacted and was highly variable.

## CONCLUSIONS AND RECOMMENDATIONS

The following paragraphs describe our geotechnical engineering conclusions and recommendations concerning the planned renovations of this dam to correct apparent geotechnical deficiencies as well as proposed spillway modifications. We understand that Walden, Ashworth and Associates, Inc. has designed a labyrinth weir spillway and new stilling basin structure to replace the existing spillway pipes and weir. The design will also incorporate a bridge over the spillway structure. There remain some logistical issues to be resolved as the design is progressed, primarily related to stream diversion, undercutting and replacement, and traffic control.

These conclusions and recommendations presented in this report are based strictly on the subsurface data available to us and our observations of surface features at the dam site combined with past experience on similar projects. No other warranty, expressed or implied, is made. These conclusions and recommendations are provided for the sole use of Walden, Ashworth and Associates, Inc. and their client for the renovation of the Emerald Lake Dam.

If additional problems that are not currently evident are observed in the course of the ongoing history of this project, we should be contacted for additional input. We recommend that engineers and technicians of our staff monitor any remediation to this dam during construction to assure that the recommendations contained in this report and in the final plans and specifications are properly implemented, and to provide an opportunity to further examine this structure.

## GENERAL

Our general impression is that this embankment dam is in fair condition, but has deficiencies primarily related to the spillway system and seepage control that should be addressed. The downstream slope is overly steep, irregular and is experiencing uncontrolled seepage at the toe and beneath the existing spillway pipes. Although steeper than desired, we did not observe any unstable slope areas.

Based on our evaluation of the boring data collected during this study, the quality and composition of the existing embankment fill materials appear random within the embankment section and have zones that generally appear to be under-compacted. The upper few feet of fill generally appears to be somewhat better compacted than the deeper fills. The consistency of the fills decrease with increasing depth. The lowest consistency fill materials were encountered in borings B-1 and B-2 at a depth of 5 feet or less above the transition from fill to residuum or fill to alluvium.

The joints of the existing spillway pipes have been visually observed to have separated, which has created surface depressions and significant ground loss above the right most pipe and if not corrected will allow the internal erosion condition to worsen at this or develop at other locations. The downstream end of each pipe was also severely undermined several feet. We suspect the apparent settlement of all five spillway pipes (located about 35 to 40 feet upstream of the outlet end) and the presence of alluvial soils in borings B-2 and B-4 delineates two episodes of construction; the latter of which was the placement of a downstream zone of fill over the existing materials to widen the existing dam crest. The upstream slope above the current normal pool level is moderately steep and slightly irregular. Given the current condition of the embankment, based on our observations and subsurface data collected, we are of the opinion the entire dam does not need to be fully upgraded, but rather only the area directly impacted by and surrounding the spillway modifications. We note that the current vegetation could be masking a seepage problem that may not become evident until after this episode of construction is complete and the lake re-impounded. In lieu of modifying the entire dam with a complete seepage collection system, we recommend addressing any subsequent seepage concerns if they develop. At this time, we only recommend extending the seepage collection system a prescribed distance right and left of the proposed spillway structure.

We understand the spillway upgrades include slip-lining the existing low level pipe and installing a new riser pipe/or gate structure as opposed to complete replacement. This allows for the existing pipe to remain in use as a means to keep the lake level down during construction, but its alignment, elevation and condition may cause some logistical issues during the breach excavation and undercutting process.

Within the limits of construction, the downstream slope should be flattened to create a more stable, maintainable, configuration. The recommended embankment modifications should include the placement of an internal drainage blanket/partial chimney drain and pipe to help control the development of the phreatic surface in the modified embankment zone. The upstream slope above normal pool should be flattened and protected by a suitable form of wave protection.

We envision the construction activities required to rehabilitate this embankment will include the following:

- Draining of the lake and maintaining a drained/lowered condition during construction
- Slip-lining of the existing low level pipe (if it is to remain)
- Construction of a temporary roadway/coffer dam
- Installation of a temporary dewatering system
- Removal of all unsuitable vegetation on the embankment
- Undercutting and replacement of unsuitable soils beneath the footprint of the new spillway, including the temporary routing or support of utilities
- Removal of existing spillway pipes, subgrade preparation and backfilling of the excavation
- Installation of seepage collection drains
- Spillway and bridge construction
- Fill placement to create flattened downstream slope
- Construction of the upstream crest wall
- Final grading of slopes and permanent grassing
- Replacement of asphalt

## TEMPORARY GROUNDWATER AND SURFACE WATER CONTROL

Groundwater conditions at the time of construction will pose considerable challenges during subgrade preparation and other construction activities that will take place in and near the floodplain. No explorations have been performed downstream of the embankment, and as such, no groundwater data has been collected for this area. However, based on our observations, ambient groundwater conditions downstream of the dam are likely within 3 to 5 feet of the general floodplain level (804-805 feet), or about elevation 800 to 802 feet. As such, we anticipate dewatering efforts will be required to lower the current groundwater level by as much as 10 to 12 feet, or about 15 to 17 feet below the general floodplain level. Actual groundwater conditions will depend on the time of the year and prevailing weather patterns at the time of actual construction and the lake level; the contractor may experience higher groundwater levels than those presented and discussed in this report. Draining of the lake in advance of beginning construction may help with the dewatering efforts.

Groundwater and surface water control is necessary during construction since this water will likely adversely impact subgrade preparation and other activities that will take place in conjunction with the renovation of this dam. These activities will include, but are not limited to, undercutting of the alluvial soils, initial backfilling of undercut areas, removal of existing spillway pipes, toe and foundation drain construction, excavation and construction of any required plunge pools and the stilling basin associated with the labyrinth spillway structure, and the lower elevation portions of turndown construction associated with this spillway. In addition, depending on the contractor's selected plan for constructing the foundation drain portion of the internal drainage system, even deeper temporary excavations may be needed. It is also possible that deeper than anticipated zones of alluvium may be encountered, which would result in some additional excavation depth and dewatering efforts.

Some of the difficulties in dealing with the groundwater are directly impacted by the time required for the particular element of construction, and the speed at which the contractor can execute the work. For example, undercutting the breach area in relatively small strips or sections could allow for immediate backfilling of the approved areas with sufficient speed to possibly prevent any significant problems from a slight amount of groundwater seepage. Similarly, the lower portions of the toe/foundation drain can possibly be backfilled with sufficient speed (assuming satisfactory execution and appropriate advance planning by the contractor) to place drain materials above the water problem before difficulties occur. The most significant difficulties may relate to such structures as the spillway stilling basin, which will require that the subgrade be prepared, and considerable fairly slow additional construction take place to actually construct this structure. This type of construction will be more difficult to successfully implement if groundwater control is marginal. Based on our previous experience in dam construction, we anticipate a vacuum well-point system will be required to satisfactorily lower the groundwater in order to accomplish the undercutting and backfilling, construction of the lower drain segments and stilling basin construction.

The dewatering techniques utilized on this project should be the sole responsibility of the contractor. We recommend that the contract documents clearly indicate that the design and implementation of the dewatering system is the contractor's responsibility, and that these documents establish performance criteria for assessing the effectiveness of the dewatering system actually installed. Typically, the performance criteria requires that the dewatering system successfully lower the prevailing groundwater levels at least 3 feet below the lowest anticipated subgrade levels in advance of excavation. This is typically confirmed by shallow observation wells spread around the area where groundwater is anticipated to be problematic. Weather conditions during construction can also have an impact on the effectiveness of the dewatering system selected. In addition, the contractor should be made aware that adjustments to the dewatering system may be needed if areas of deeper excavation are required based on the conditions actually exposed during construction.

The dewatering system implemented should function continuously 24 hours a day, 7 days a week until the excavations are properly backfilled, or structures are placed to at least 2 to 3 feet above the ambient stabilized groundwater levels. The project specifications should require that the contractor submit a detailed dewatering plan for the engineer's review and approval prior to implementation. These plans should be provided early in the overall construction process to allow adequate time for review, re-submittals if necessary, and implementation of the plans in a timely fashion so as not to impact the contractor's schedule. Any dewatering system implemented must be properly abandoned.

As indicated previously, the contractor will also be required to deal with surface water diversion during construction. The difficulty of handling diversion will be directly impacted by the weather conditions prevalent during construction, and the contractor's selected sequencing of operations. It is anticipated that the lake will need to be essentially drained, or lowered as low as possible to help minimize the dewatering requirements, and also to provide for reasonable storage from rainfall events during the period of time that the partial excavation through the dam is created for constructing the labyrinth spillway. We envision that this drawdown would be controlled by the low-level pipe supplemented by pumping, if needed. We understand that the existing low level pipe is to be slip-lined and remain in place. The removal of the five existing spillway pipes is planned to be sequenced in such a way that two or three may remain in place, while the new spillway in being constructed, to aid in diversion and control of surface water during this critical time. Project specifications should require that the contractor submit a detailed diversion plan for the engineer's review prior to implementation. The diversion plan should also consider the traffic diversion schemes and their impact on stream diversion.

## EMBANKMENT MODIFICATIONS/CONSTRUCTION

Our observations and the survey provided indicates that the downstream slope of this dam is irregular and steeper than the typical 3(H):1(V) slopes typically utilized to create stable earth dams in this area. Within the limits of construction and to the greatest extend practical we recommend that the renovation plans include modifying the overall geometry/configuration of this dam to a uniform configuration. The downstream slope should be flattened to create a uniform configuration of 3(H):1(V) or flatter by adding additional earth to extend the slope farther downstream. If possible, the crest should be sloped towards the lake to minimize surface flows across the longer downstream slope section. Should the slope be projected farther downstream, we recommend all existing fills and alluvial soils be undercut and replaced beneath the footprint of the new 3(H):1(V) slope projection. The downstream limits of the undercutting should extend at least to where the projected 3(H):1(V) slope contacts the approved residual subgrade. The temporary upstream or leading edge of the undercutting should be created no steeper 1.5(H):1(V). Beneath the new labyrinth spillway footprint all of the existing fills and alluvial soils will need to be undercut to residuum extending a minimum of 5 feet beyond the structure at the residual subgrade level and sloped up on a 1.5(H):1(V) slope or flatter to the existing crest. Based on the results of the soil test borings drilled along the existing crest, undercut depths are expected to extend at least 19 to 26 feet below the dam crest, or about elevation 791 to 798 feet. We anticipate the depth/level of undercutting could increase in the downstream direction and around the original stream channel. Some slope flattening of the upstream slope above the normal pool elevation may be desired to create a more uniform slope that can be maintained and to create a uniform section of wave protection. All final subgrade preparation should be made with a smooth blade or straight edge on an excavator bucket.

Beyond the required undercut for the new spillway structure, partial to total undercutting and replacement may be required to provide suitable subgrade conditions capable of supporting drain construction and the new fill for the downstream slope extension. Actual undercut limits will be determined at the time of construction. This will be discussed further in the Internal Drains section of this report.

Prior to beginning construction, a source(s) of suitable embankment fill materials needs to be located and approved by the geotechnical engineer. Most of the existing fill encountered during this investigation appears visually suitable for re-use as structural fill, though much of this material will require drying before re-use. For the purposes of this project, we have used the terminology "select" and "common" to represent different classes of soil materials and their general placement within the embankment. "Select" fill should be used to backfill in all undercut areas up to the pad/platform grades and for the excavation created by the removal of the existing spillway. In the dam breach, "select" fill should be used to at least 2 feet above normal pool. All other areas can be backfilled using "common" soils. "Select" fills are defined as soil materials having USCS designations CL, ML and SC and "common" soils can be all of the "select" designations plus SM. All SC and SM materials are required to have at least 30% passing the #200 sieve and a Plasticity Index of at least 5.

The entire embankment should be stripped of all vegetation, stumps and associated roots. All fill materials placed should consist of clean soils, free of deleterious materials and rock fragments larger than approximately three inches in diameter. The compacted soil should generally have a maximum dry density (ASTM D-698) of at least 90 pcf. We recommend that all fill placed be compacted to a minimum of 95 percent of the soil's standard Proctor maximum dry density at a moisture content ranging from the soil's optimum moisture content to above the optimum moisture content. The fill materials should be placed in essentially horizontal lifts across as much of the embankment footprint as possible at any given time to prevent the formation of an interface between zones of fill placed at different times in a near-vertical upstream-downstream orientation. The soil should be placed in relatively thin lifts and uniformly well compacted with self-propelled sheepsfoot rollers. No previous fill lifts should be left in a smooth condition, such as results from rubber-tired rolling, at the time of placement of subsequent fill lifts. Should a smooth condition result, it will be necessary to lightly scarify each fill lift to assure adequate bonding with the overlying lift prior to subsequent fill placement. In addition, during breaks in the grading activities, should the exposed subgrade become overly dry or overly wet, it may be necessary to blade off these materials, or to scarify and re-compact these materials in-place, with appropriate moisture adjustment, prior to the addition of subsequent fill layers.

In areas where existing slopes or temporary fill slopes (that are not covered with Chimney Drain) are steeper than 5(H):1(V), benching of the slope soils will be necessary in areas not impacted by drains to adequately tie in the new fill. Where fill is placed around outlet conduits, it will be necessary to maintain the level of fill approximately equal on both sides of the conduit to prevent possible lateral displacement and damage to the structure. In addition, adjacent to conduits, immediately behind walls, and near similar structures, thinner fill lifts and portable compaction equipment such as hand tamps will be required.

The existing riser for the low level pipe has partially failed and is leaning several degrees. We anticipate that a new riser/gate structure will be part of the rehabilitation plan and that minor undercutting and subgrade preparation will be required for proper support of the new structure. Once slip-lined, the low level pipe will also need to be extended to outlet beyond the new slope projection. It may be prudent to slip-line the existing low level pipe early in construction sequence to avoid the pipe leaking and causing subgrade or slope stability issues within the breach. The new section of pipe should be partially encased in a concrete cradle and will likely also require undercutting to create a suitable fill subgrade to support the new pipe and cradle.

## SPILLWAY MODIFICATIONS

At this time, we understand you are currently planning to replace the existing spillway with a new labyrinth weir spillway and stilling basin. The proposed spillway is 40 feet wide and will be placed towards the right side of the existing spillway system. The roadway across the crest of the dam serves as access to the south end of the development. As such, a bridge will be placed over the spillway and the labyrinth walls will be used for bridge support.

Excavations into the dam to support any new concrete structures will encounter subgrade conditions which will require total remedial undercutting and replacement with new structural fill to provide uniform subgrade support. The width of the undercut zone should be equal the width of the spillway plus 5 feet on each side at the approved residual subgrade (+/- 791 to 798 feet) and include excavating slopes through the embankment/alluvial materials no steeper than 1.5(H):1(V). Based on sections drawn showing this breach configuration, the left two or three spillway pipes and the low level pipe will be potentially affected by the breach excavation. We understand that the low level pipe is to remain and that it is preferable to temporarily leave the two left spillway pipes in place. To achieve this, the new spillway may need to be shifted to the right some distance, to allow for the left two pipes to remain in place with the required excavation and the excavation at the low level pipe steepened or the low level pipe may need to be temporarily shored. Once the new spillway has been constructed, we understand that the remaining spillway pipes will be removed in a subsequent excavation. Due to the noticeable settlement approximately 40 feet upstream from the downstream end of the pipes we anticipate some additional subgrade remediation efforts may be needed to create a suitable subgrade to place the new fill on. The actual limits and extent of this remediation cannot be accurately determined until the time of construction. Due to the poor condition of the inverts of the existing spillway pipes, we expect the subgrade, when exposed, to be saturated and will require minor remediation to prepare a stable subgrade for fill placement. As mentioned previously, it may be prudent to slip-line the existing low level pipe prior to beginning the undercut/breach excavation to minimize seepage into the undercut excavation which is expected to be at least 5 to 7 feet deeper. Since it is planned to leave this pipe in a functional condition and to serve as a means for keeping the lake in a lowered condition, routine or near constant flow through the pipe is expected. As such, if the pipe leaks it may impact subgrade preparation and/or slope stability of the breach excavation.

As indicated previously in this report, it will be necessary to remove and replace materials beneath the spillway footprint. Dewatering within this area will be necessary. A positive dewatering effort using vacuum well-points will be needed to dewater the excavation until fill placement is at least 3 feet above the prevailing water level. In addition, we anticipate that it may be most appropriate to mass excavate the unsuitable materials, backfill, and then carefully excavate to the desired subgrade levels for the labyrinth spillway system within these newly compacted fill materials.

In conjunction with this spillway, several turndowns, and the associated underdrain system will be designed into this structure. Typically, turndowns are required at the upstream and downstream limits of the spillway base slab that supports the labyrinth weir walls, at each of the chute slab sections, and at the downstream end of the stilling basin, as a minimum. In addition, underdrain systems are normally included downstream of the labyrinth weir walls, which generally corresponds to the downstream end section of the main slab, the entire sloped chute, and the stilling basin. Blanket drains beneath each of the slab sections created by the turndowns, with collector pipe drains adjacent to the upstream side of each turndown are typically utilized. For this application, we recommend that a total aggregate system be utilized for the underdrain system. Generally, no filter fabric should be included. Fine filters at a minimum recommended thickness of 9 inches may consist of layers of ASTM C-33 sand (standard gradation) and washed No. 89 stone. The coarse filters may be comprised of washed No. 57 stone; we recommend a minimum thickness of 12 inches for all areas with underdrain, except that the minimum thickness should be increased to at least 18 inches beneath the stilling basin and the lowest sloped section that supports the chute blocks. Perforated PVC pipe collectors a minimum of 6 inches in diameter and routed through the sidewalls of the spillway system are typically utilized to outlet the collected under seepage, and to provide hydrostatic pressure relief during flood events.

All exposed subgrade materials should be in a compact condition at the time of underdrain and/or concrete slab construction. All disturbed materials should either be compacted in place to the required density, with appropriate moisture conditioning, if needed, or the material should be removed and replaced with new structural fill. For minor depths of over-excavation needed to remove disturbed materials, an option may be to utilize additional aggregates for the underdrain system and/or additional concrete where slabs-on-grade are created to replace the over-excavated materials. This should be left to the discretion of the contractor, based on discussions with the engineers, and should be provided at no additional cost to the project.

## INTERNAL DRAINS

The following recommendations concerning seepage control are based on limited field data obtained during this exploration, and our past experience with similar projects. It is our opinion that any observed and potential seepage should be adequately controlled. Seepage left uncontrolled tends to worsen with time, and can cause internal erosion of the embankment and foundation soils. In addition, and often a more prevalent issue, is that uncontrolled seepage presents maintenance difficulties due to the general soft ground conditions that result, as well as having a negative impact on overall slope stability. Based on our observations at the site, the phreatic levels measured in the observation wells in combination with the wide embankment section, we are of the opinion the existing embankment dam is not currently experiencing a wide spread seepage problem, but rather the area of suspected seepage is more concentrated to around the existing spillway system and in the apparent original stream valley. As such, we are not recommending the entire embankment be modified to include a seepage collection system, as would be required on Category I dams, at this time. We note that most of the downstream slope and areas immediately downstream of the dam contain many trees that should be removed as part of this planned construction so that suitable grassing can be established and maintained. These trees may be masking an existing seepage condition that may not become apparent until sometime after the tree removal and re-impoundment of the lake. Should an uncontrolled seepage condition develop, additional modifications to the dam may be needed and recommended at the appropriate time.

The general seepage control modifications recommended for this dam include a toe/foundation drain placed generally parallel to the downstream toe of the existing embankment slope, combined with a partial height chimney drain placed on the prepared existing downstream slope face or in a vertical excavated trench up to at least elevation 809 feet. The proposed drain should extend laterally at least 100 feet left and at least 40 feet right of the new spillway or to the maximum practical extent. Beyond the limits of the breach required for the new spillway and prior to drain construction, the existing downstream slope of the dam should be thoroughly stripped and generally configured to a 2(H):1(V) configuration beginning at the downstream edge of the right-of-way limits, with the surface compacted reasonably well. As discussed in a previous section of this report, remedial subgrade preparation is required along the downstream toe of the existing dam and will likely be completed either prior to or in conjunction with constructing portions of this recommended drainage system. Once the slope has been cut to the 2(H):1(V) configuration as discussed, the exposed subgrades can then be evaluated. Based on our visual observations of the existing conditions downstream of the left two spillway pipes, we anticipate that some undercutting may be required where the drains will be constructed. For areas where undercutting is required, the 2(H):1(V) slope should be extended to residual soils and should extend downstream the distance of the new 3(H):1(V) slope projection. Due to the anticipated depths of undercutting at the breach area along this drain alignment, especially in the stream valley, it may be necessary to construct the toe/foundation drain in the deeper undercut areas in multiple lifts to prevent having to create an overly deep excavation for this purpose after backfilling has been completed. Once the toe/foundation drain is completed, lower portions of the chimney drain can be constructed on the prepared downstream slope face, and additional fill materials placed to cover the drain system and create the extended downstream toe configuration previously recommended. The spillway underdrain system and the embankment seepage collection system should be separated and with discrete outlets; however, the internal drainage system should provide sufficient overlap as to prevent the inclusion of a section of embankment without seepage collection.

The toe drain alignment will generally follow the toe of the excavated 2(H):1(V) temporary slope at a created pad grade of approximately elevation 804 feet (where undercutting is required) or higher in areas where less undercutting is required. However, from a practical construction standpoint and to lessen the risk of slope failure/trench collapse, we recommend the design provide for an approximately 5-foot minimum horizontal offset to the trench alignment from the 2(H):1(V) temporary toe so that it can be excavated from the general initial recommended backfill grade at the downstream toe of the dam without need for cutting further into the existing embankment. Where the chimney drain is placed above the toe drain, a horizontal section of chimney will be required to connect the chimney to the offset toe drain. Where drain construction can commence without undercutting, the drain will still generally follow the toe of the 2(H):1(V) slope where it meets the existing floodplain elevation. In this situation, the drain will be mostly a toe/foundation drain system. The drain should still extend 2 feet into residual soils or a maximum depth of 7.5 feet. For all of the drainage system components described, we anticipate that our staff will continue to work closely with WAA to assist with the details that will be prepared as part of their construction documents. This report section is intended to provide a general overview of the drainage system components and requirements.

Along portions of the alignment of the toe drain, generally confined toward the ends of this system in the abutment areas, the base of the minimum 4-foot tall toe drain may penetrate into the residual foundation materials. However, along most of this alignment where removal of unsuitable materials and new backfill materials would be placed, it will be necessary for a foundation drain extension to be placed below the toe drain to penetrate these materials and extend at least 2 feet into the underlying residual soil profile, or to refusal, whichever comes first. We would recommend against the contractor attempting placing this entire system in one operation due to potential instability of the side slopes of the trench relative to the rate of construction possible. Therefore, it will likely be necessary along much of this alignment to construct this toe/foundation system in at least two or more lifts. In such case, the lower portion of the drain (foundation drain) could be partially constructed after partial backfilling of the undercut area, with the remaining portion of the foundation drain and the toe drain constructed by excavating through the upper backfill materials and cleaning off the surface of the previously placed foundation drain lift(s). It has been our experience that a maximum trench depth of about 4 feet limits the risk of trench instability and can aid with general safety considerations. Careful control of the alignment of the drain system to assure that the individual lifts line up appropriately would be required. In addition, a straight edge blade on the backhoe bucket is required to adequately clean the previously placed foundation drain lift when re-exposed without the need for laborers to enter the excavation for final cleanup. By limiting the lifts to about 2 feet, it should be possible for the material to be placed in the trench to a level that would allow small vibratory sleds to compact the initially placed aggregates, and then to fill the trench and again compact the remaining aggregates in reasonable lifts. There should never be a need for individuals to be in a trench that is too deep utilizing this approach. We recommend that the foundation drain portion of this system consist of ASTM C-33 concrete sand (standard gradation). Where the foundation drain component is required, the filter fabric wrapped toe drain should be constructed with the filter fabric draped into both sides of the trench in an "open bottom" configuration prior to completing placement of the foundation drain aggregates. The filter fabric should be embedded at least 18 inches into the upper portion of foundation drain. Therefore, considering that the overall toe drain component has a recommended height of 4 feet, this would require that the uppermost portion of this drain system have an actual trench depth of about 5.5 feet, with the top of the lowest lift of foundation drain prior to constructing the toe drain maintained at least 18 inches below the base of the toe drain to allow the fabric to be adequately embedded into the remaining foundation drain aggregate after excavation through the upper backfill materials.

The top of this system would have an open configuration for the filter fabric to allow direct contact between the recommended chimney drain and the top of the toe drain. To provide the proper filter transition, it will be necessary to provide a layer of No. 89 stone at least 9 inches and up to 12 inches thick at the top of the 4-foot deep toe drain section. Another similar layer of No. 89 stone 9 to 12 inches thick would be required to separate the foundation drain sand from the No. 57 stone at the bottom of the toe drain.

Solid outlet pipes should be provided for the toe drain collection system at maximum spacing of approximately 200 feet. We currently envision that one outlet on each side of the spillway should be sufficient for this system. All of the perforated drain pipes and outlet pipes should provide for at least a minimal amount of slope for proper drainage. We recommend maximum 22.5 degree bends to be used at all turns and elevation changes so that the entire pipe system could be cleaned from the downstream end. We also recommend cleanouts be placed at the end of each drain segment. Small animal guards and headwalls should be included at the outlet ends at all discharge pipes. The headwalls can be eliminated if the outlet can penetrate the spillway at a suitable elevation. The outlet structures should be constructed so that flows from the pipes can be collected and monitored. To provide for durability, the PVC pipe should transition to a section of ductile iron pipe for the last section that would extend through the headwall and remain exposed. We recommend that headwall construction be such that a minimum 12-inch drop below the pipe can be maintained for monitoring purposes. The outlet pipe should extend at least 2 inches beyond any headwall surface, primarily to accommodate the installation of the small animal guards.

The deeper portions of the toe/foundation drain system will likely require dewatering during construction. It will be necessary for the contractor to excavate a relatively dry trench to allow for proper construction of the drain system. Positive dewatering has already been recommended within the lower portions of the embankment downstream toe area. The drainage aggregate placed should be reasonably well compacted using appropriate equipment. The drainage aggregate for the trench drain construction should be stockpiled on site in appropriate quantities to allow the contractor to immediately place these materials as sections of the trench excavation are completed. Delays could lead to sloughing and extensive repairs.

The filter fabric required in conjunction with the drain construction should consist of a nominal 8 ounce per square yard needle-punched, non-woven polypropylene fabric with an AOS of 80 to 100 intended specifically for this purpose. Recent projects have utilized fabric such as Amoco 4553, Mirafi 180N, Terratex N08 or approved equivalent. The contractor should be required to submit his fabric and aggregate information to us for review prior to implementing them into the construction. It is critical for the fabric to be placed in imminent contact with a relatively undisturbed soil interface to prevent clogging of the fabric. No fabric should be placed in a wet or muddy excavation. The filter system for the drain consists not only of the filter fabric, but also the soil materials immediately adjacent to the fabric, creating a composite system.

The recommended chimney drain should consist of ASTM C-33 sand (standard gradation) placed directly on the downstream face of the existing dam that has been adequately prepared in advance as recommended. The chimney drain should have a uniform width or thickness of 2 feet measured perpendicular to the slope face extending from the toe drain up to at least elevation 809 feet.

With the toe drain alignment offset from the 2(H):1(V) toe of slope, as discussed previously, a short essentially horizontal section of the chimney drain would initially be placed to connect the top of the trench drain with the toe of the existing slope. This section of horizontal drain should have a minimum depth of 2 feet. Where the chimney drain is constructed, the top of the fabric wrapped toe drain should be left with an "open top" configuration by placing the extra filter fabric on the subgrade either side of the trench. At least an 18 to 24 inch section of fabric should be provided to extend the fabric into the interface between the chimney drain materials and the surrounding subgrade. This will separate the top of the No. 89 transition stone layer at least 18 to 24 inches from access to the surrounding soil subgrade. The portion of this extra fabric on at least the upstream side of the toe drain would then be left in a horizontal configuration beneath the connecting portion of the chimney drain. On the downstream side of the toe drain, this fabric could also be left in a horizontal configuration across the interim pad grade elevation, with the connecting horizontal piece of chimney drain extending out over the fabric beyond the downstream edge of the toe drain. We recommend at all locations that the final 3(H):1(V) are flatter downstream slope configuration provide a minimum of 3 feet of cover measured perpendicular from the slope face to any of the internal drainage system components. Since there is some variation in the existing dam downstream face slopes, this particular recommendation should consider the critical location to provide this minimum cover, likely resulting in many of the other areas providing more than this minimum.

Some additional drain details will need to be worked out where the recommended internal drainage system crosses or is in contact with any existing and new conduits or structures associated with this dam.

## LATERAL EARTH PRESSURES

Parameters related to lateral earth pressures are provided for use in designing the earth retaining structures, such as the labyrinth spillway side and wing walls, associated with this project. After these structures are constructed, backfill materials will be placed adjacent to the sidewalls. These walls will need to be designed to resist lateral earth pressures and possibly hydrostatic water pressure upstream of any drain locations. Since these structures are anticipated to be fairly rigid, we recommend that an equivalent fluid pressure for the full at-rest condition be utilized in design of these walls. For a horizontal backfill configuration, drained conditions, and no surcharge loading, an at-rest pressure of 60 pounds per cubic foot may be utilized based on past experience. Hydrostatic forces will increase the total lateral pressures through a reduction of the earth pressure based on buoyancy effects, and the addition of the full water pressure. In locations where sufficient wall movement may occur to use the reduced active earth pressure, our previous experience would suggest that an equivalent fluid pressure for this condition of about 40 pounds per cubic foot may be used. This also assumes a horizontal backfill configuration placed as recommended, drained conditions, and no surcharge loading. Further, heavy compaction equipment should not be allowed immediately behind any wall, unless the wall is designed for the increased lateral stress due to this equipment. All fill materials placed adjacent to the wall should conform to the recommendations for the general embankment fill. Portable hand operated equipment will likely be required immediately adjacent to the wall to provide proper compaction. These areas should be carefully compacted since this is a critical location for potential seepage.

Based on the project information provided and past experience, we recommend as a result of the fully submerged condition, that an allowable passive resistance of approximately 75 pounds per cubic foot as an equivalent fluid pressure be used. This is based on a passive coefficient in the range of 2.77 to 3.0, and a total unit weight of approximately 115 to 120 pounds per cubic foot for the soil. The submerged unit weight combined with a safety factor of about 2 results in the approximately 75 pounds per cubic foot value recommended. In addition to the passive resistance, we recommend a sliding coefficient of 0.35, which includes a safety factor of about 1.5. The buoyancy effects should be accounted for in calculating the normal force at the base of the structure. No other specific information related to lateral earth pressures was requested.

## FOUNDATION RECOMMENDATIONS

Since the new spillway will be supported by new fill, we recommend that the new spillway structure be designed using a maximum bearing pressure of 3,000 psf. The recommended bearing pressure is based on the new structural fill being properly compacted to the requirements stated in this report. We recommend that the crest wall be designed for a maximum bearing pressure of 1,500 psf. The quality of the embankment fill along the alignment of this wall has not been evaluated at this time.

As with any construction, all foundation excavations should be evaluated by a geotechnical engineer, who will verify that the design bearing pressure is available, and that foundations are not immediately underlain by worse conditions. If the engineer finds localized conditions of weak foundation materials an individual footing, it should be undercut or a lower bearing pressure used, depending upon the actual conditions found.

## OTHER DESIGN CONSIDERATIONS

After final grading and proper compaction of the exposed slopes and crest, suitable erosion protection should be provided. Low maintenance grasses are employed most commonly on the downstream slope, the exposed portion of the upstream slope and portions of the crest not under pavements. It has been our experience that on dams where a good vegetative cover is not established early on, problems with erosion resulting in higher long-term maintenance may occur. The "common" soils allowed at this site could be relatively sandy, and are also considered relatively erodible. Vegetative cover is a critical item and should be properly considered. Remedial maintenance and repair of eroded slopes in the future would be difficult. Such erosion can also significantly contaminate such items as the shoreline riprap and the downstream toe ditch, and lead to problems with establishment of unsuitable vegetation in these areas. Consideration might also be given to irrigating the area to at least initially to establish a good stand of grass cover. The crest of the dam should be sloped slightly toward the reservoir.

Riprap wave protection should be considered on the upstream slope face and downstream of the spillway outlets. We recommended that any riprap used be bedded on smaller stone underlain by filter fabric. The fabric used should have the same properties as the fabric discussed in conjunction with the internal drainage system. Based on past experience, the bedding stone would typically consist of a minimum of about 6 inches of crushed stone such as #57 gradation. The individual riprap rock fragments should be dense, sound and resistant to abrasion and should be free from cracks, seams and other defects that would tend to unduly increase their destruction by water and frost action. The riprap rock should also be sized as appropriate for the anticipated velocities and/or wave action.

Consideration should be given to installing groin protection at the contact between the downstream slope and the abutments of the spillway outlets. We recommend this consist of a rock lined toe ditch along the entire length of the downstream contact of the embankment and surrounding areas.

We expect the new/replacement pavement section will be designed to match the existing conditions identified. All pavement subgrades should be proofrolled by the geotechnical engineer with a loaded (20-tons) tandem axle dump truck. Proofrolling the subgrade will identify any unstable or soft conditions which could lead to asphalt failure.

## CLOSURE

We appreciate the opportunity to provide you with this geotechnical engineering evaluation. We remain available to assist you as you develop plans and specifications for remediation of this project, and also to provide the recommended construction phase monitoring services. Should you have any questions concerning this report, or if we may be of additional service to you in any way, please do not hesitate to contact us.

Sincerely,
**Piedmont Geotechnical Consultants, Inc.**

Jonathan P. Sharpe. E.I.T.
Project Engineer

H. Craig Robinson

H. Craig Robinson, P.E.
Senior Registered Engineer

HCR/ew

Attachments: Soil Test Boring Procedures
Correlation with Standard Penetration Test Results
Figure 1: Site and Boring Location Plan
Soil Classification Chart
Soil Test Boring Records (4)
Summary of Hand Auger Borings

# APPENDIX

## SOIL TEST BORING PROCEDURES (ASTM D-1586)

The soil test borings were advanced by twisting continuous auger flights into the ground. At selected intervals, soil samples were obtained by driving a standard 1.4 inch I.D., 2.0 inch O.D., split tube sampler into the ground. The sampler was initially seated six inches to penetrate any loose cuttings created in the boring process. The sampler is then driven an additional 12 inches by blows of a 140 pound "hammer" falling 30 inches. The number of blows required to drive the sampler the final foot is designated the Standard Penetration Resistance.

The samples recovered were sealed in glass jars and were transported to the office where they were classified by an engineer in general accordance with the Unified Soil Classification System (USCS).

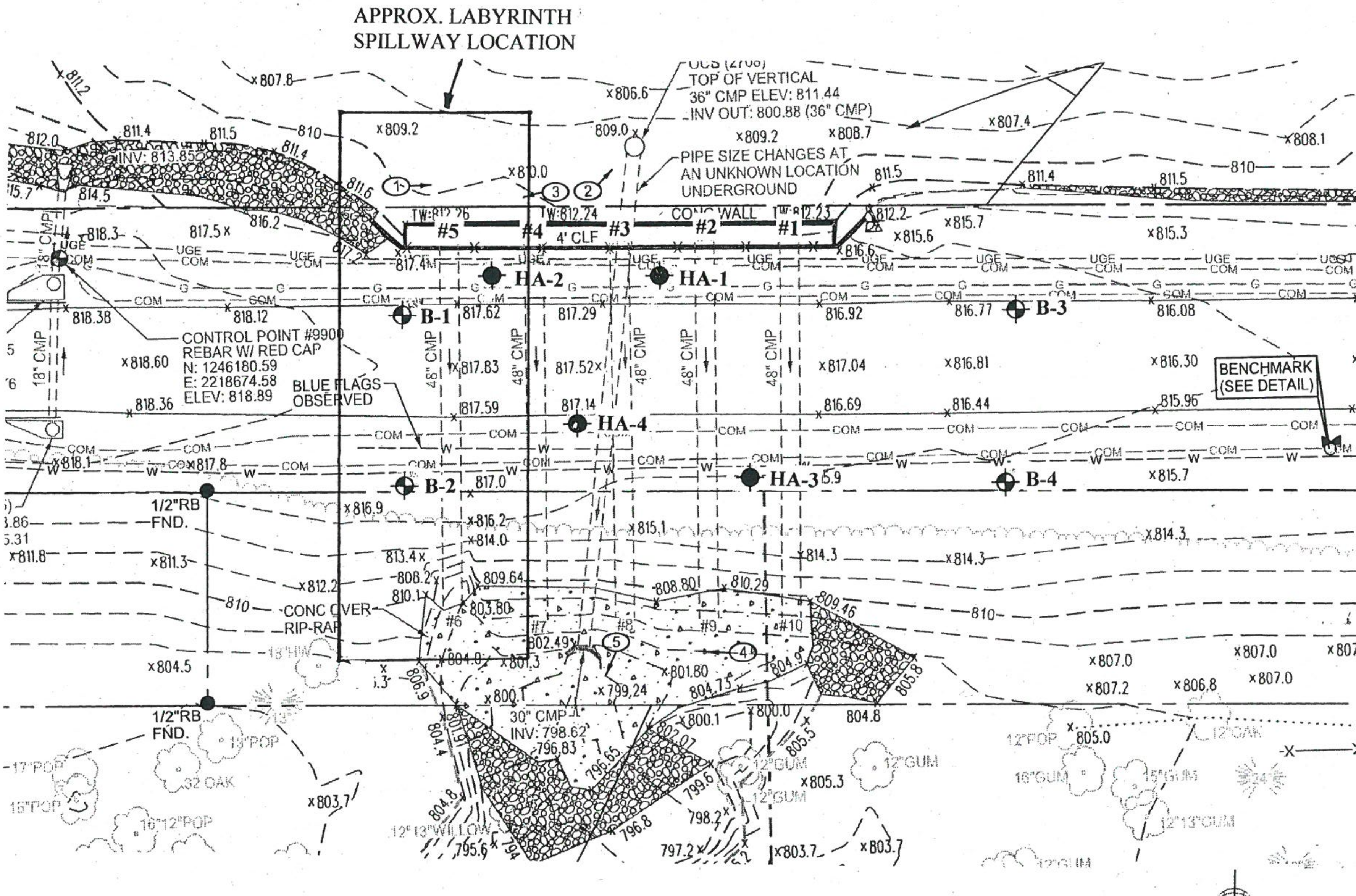
## CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE COMPACTNESS AND CONSISTENCY

### Sand and Gravel

| Standard Penetration Resistance Blows / Foot | Relative Compactness |
|---|---|
| 0 - 4 | Very Loose |
| 5 - 10 | Loose |
| 11 - 30 | Medium Dense |
| 31 - 50 | Dense |
| Over 50 | Very Dense |

### Silt and Clay

| Standard Penetration Resistance Blows / Foot | Relative Compactness |
|---|---|
| 0 - 1 | Very Soft |
| 2 - 4 | Soft |
| 5 - 8 | Firm |
| 9 - 15 | Stiff |
| 16 - 30 | Very Stiff |
| 31 - 50 | Hard |
| Over 50 | Very Hard |




# FIGURE 1: SITE AND BORING LOCATION PLAN

| LEGEND: | | |
|---|---|---|
| SOIL TEST BORINGS<br>HAND AUGER BORINGS<br>SCALE: 1"=30' | PIEDMONT GEOTECHNICAL CONSULTANTS, INC.<br>P.O. BOX 1997<br>ROSWELL, GA 30077 | EMERALD LAKE DAM<br>EMERALD LAKE DRIVE<br>FAYETTEVILLE, GEORGIA<br>PGC PROJECT NO. 114019 |

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS | | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| | | | GRAPH | LETTER | |
| COARSE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | GRAVEL AND GRAVELLY SOILS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br>(LITTLE OR NO FINES) | | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | | | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | GRAVELS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| | SAND AND SANDY SOILS<br>MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | CLEAN SANDS<br>(LITTLE OR NO FINES) | | SW | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | | | SP | POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES |
| | | SANDS WITH FINES<br>(APPRECIABLE AMOUNT OF FINES) | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS<br>MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | | OL | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | OH | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| ALLUVIUM | | | | PT | ALLUVIUM, PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |
| FILL | | | | FILL | MATERIAL PLACED BY MAN |

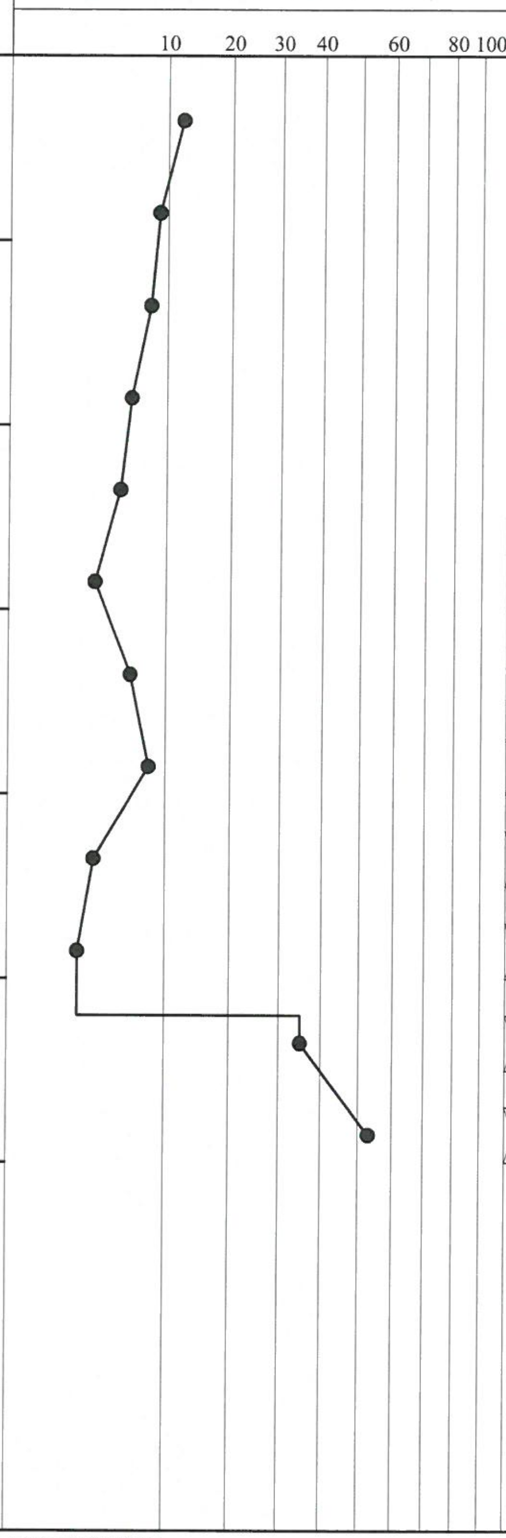
NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

**PIEDMONT**
GEOTECHNICAL CONSULTANTS, INC. PGC

# EMERALD LAKE DAM

| DEPTH (FT) | DESCRIPTION | ELEV. |
|---|---|---|
| | | 817 |
| | ASPHALT: 3 inches | |
| | BASESTONE: 15 inches | |
| | FILL: Medium dense to loose multi-colored clayey silty medium to fine SAND (SM), trace organics, clean topsoil zones | |
| 5 | | 812 |
| 10 | | 807 |
| | Very loose brown clayey silty medium to fine SAND (SM), organics | |
| 15 | | 802 |
| | Loose to very loose gray brown silty medium to fine SAND (SM), trace clay, trace organics | |
| 20 | | 797 |
| 25 | | 792 |
| | RESIDUUM: Dense gray white silty medium to fine SAND (SM) | |
| 30 | | 787 |
| | Boring terminated at 30 feet | |
| | Temporary observation well installed with screen set from 15 to 20 feet. | |
| 35 | | |
| 40 | | |

PENETRATION (BLOWS PER FOOT)




| |
|---|
| 12 |
| 9 |
| 8 |
| 6 |
| 5 |
| 3 |
| 6 |
| 8 |
| 3 |
| 2 |
| 35 |
| 53 |

SOIL BORING RECORD 114019 EMERALD LAKE DAM.GPJ PIEDMONT GEO.GDT 1/5/15

REMARKS: 3/20 - Lake - 12 inches below normal pool.

## SOIL BORING RECORD

| | |
|---|---|
| BORING NUMBER | B-1 |
| DATE DRILLED | 3/10/2014 |
| PROJECT NUMBER | 114019 |
| PAGE | 1 of 1 |

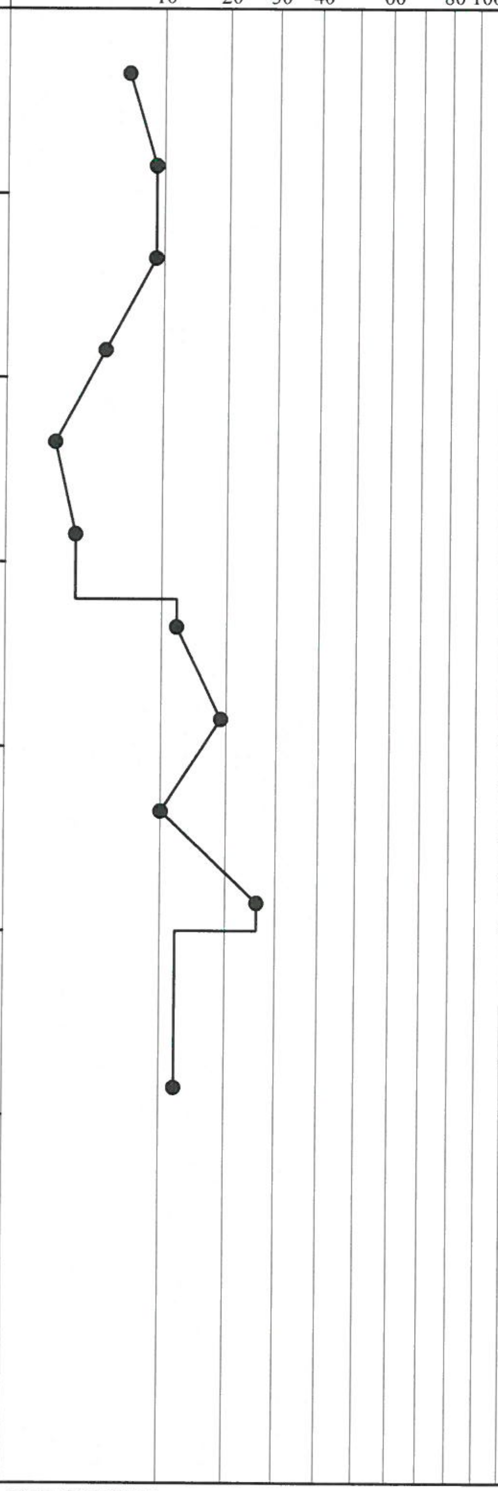
▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

# PIEDMONT
GEOTECHNICAL CONSULTANTS, INC. PGC

# EMERALD LAKE DAM

| DEPTH (FT) | DESCRIPTION | ELEV. |
|---|---|---|
| 0 | FILL: Loose red gray clayey SAND (SC) | 817 |
| 5 | | 812 |
| | Loose brown silty medium to fine SAND (SM) | |
| | Soft gray brown clayey SILT (ML) | |
| 10 | Very soft gray brown medium to fine sandy SILT (ML) | 807 |
| 15 | | 802 |
| | ALLUVIUM: Loose to medium dense gray brown silty fine to coarse SAND (SM), becomes more coarse sand with increasing depth, very wet | |
| 20 | | 797 |
| 25 | RESIDUUM: Medium dense brown tan silty medium to fine SAND (SM), micaceous | 792 |
| 30 | Boring terminated at 30 feet<br>Temporary observation well installed with screen set from 25 to 30 feet. | 787 |
| 35 | | |
| 40 | | |

PENETRATION (BLOWS PER FOOT)




| | |
|---|---|
| 1 | 6 |
| 2 | 9 |
| 3 | 9 |
| 4 | 4 |
| 5 | 1 |
| 6 | 2 |
| 7 | 12 |
| 8 | 19 |
| 9 | 10 |
| 10 | 26 |
| 11 | 12 |



REMARKS: 3/20 - Lake - 12 inches below normal pool.

## SOIL BORING RECORD

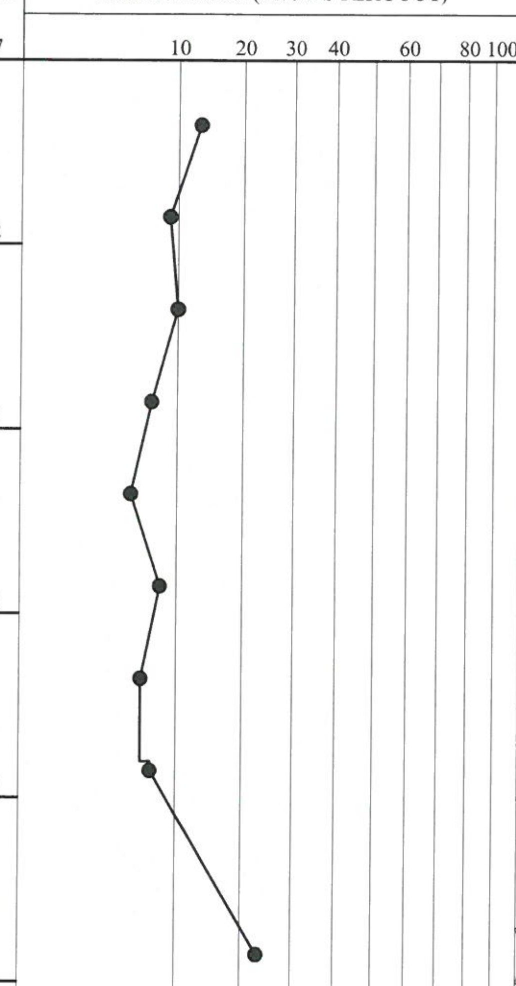
| | |
|---|---|
| BORING NUMBER | B-2 |
| DATE DRILLED | 3/6/2014 |
| PROJECT NUMBER | 114019 |
| PAGE | 1 of 1 |

▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

PIEDMONT GEOTECHNICAL CONSULTANTS, INC. PGC

# EMERALD LAKE DAM

| DEPTH (FT) | DESCRIPTION | ELEV. | PENETRATION (BLOWS PER FOOT) |
|---|---|---|---|
| 0 | ASPHALT: 4.5 inches | 817 | |
| | BASESTONE: 16.5 inches | | |
| | FILL: Medium dense to loose brown red tan clayey silty medium to fine SAND (SM), trace organics | | 13 |
| 5 | | 812 | 9 |
| | | | 10 |
| 10 | | 807 | 7 |
| | Firm brown red medium to fine sandy silty CLAY (CL) | | 5 |
| 15 | | 802 | 8 |
| | | | 6 |
| 20 | RESIDUUM: Loose to medium dense gray white silty medium to fine SAND (SM) | 797 | 7 |
| 25 | | 792 | 23 |
| | Boring terminated at 25 feet | | |
| 30 | | | |
| 35 | | | |
| 40 | | | |




REMARKS: Groundwater level not determined.
Borehole abandoned by filling with grout.

## SOIL BORING RECORD

| | |
|---|---|
| BORING NUMBER | B-3 |
| DATE DRILLED | 3/11/2014 |
| PROJECT NUMBER | 114019 |
| PAGE | 1 of 1 |

▽ Groundwater level at time of boring

▼ Groundwater level - 24 hrs

C Caved depth - 24 hrs

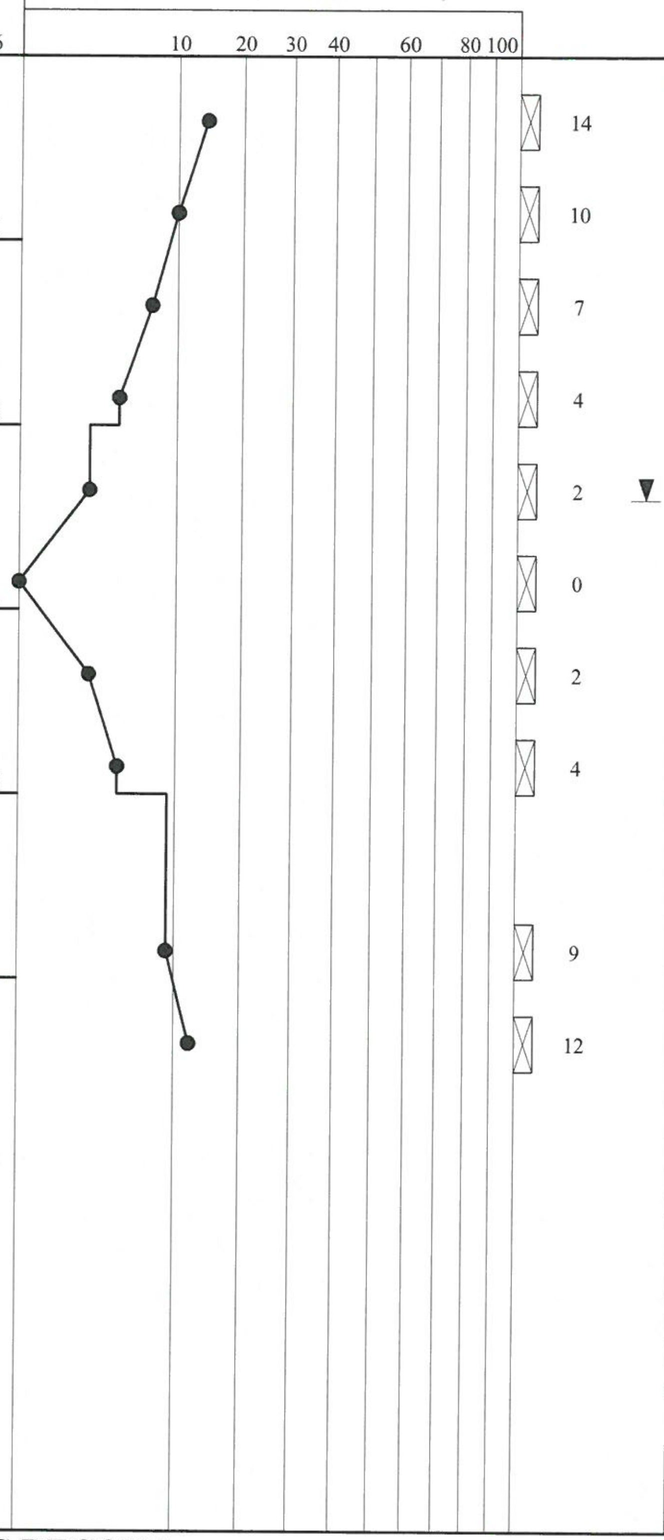
■ Undisturbed sample

**PIEDMONT**
GEOTECHNICAL CONSULTANTS, INC. PGC

# EMERALD LAKE DAM

| DEPTH (FT) | DESCRIPTION | ELEV. 816 |
|---|---|---|
| 0 | FILL: Medium dense to very loose brown tan gray clayey silty medium to fine SAND (SM) | 816 |
| 5 | | 811 |
| 10 | ALLUVIUM: Very loose gray clayey medium to fine SAND (SC), trace organics | 806 |
| 15 | Very loose gray silty SAND (SM) | 801 |
| 20 | RESIDUUM: Loose to medium dense brown tan gray silty medium to fine SAND (SM) | 796 |
| 25 | | 791 |
| 27.5 | Boring terminated at 27.5 feet | |
| 30 | Temporary observation well installed with screen set from 15 to 20 feet. | |
| 35 | | |
| 40 | | |

PENETRATION (BLOWS PER FOOT)




Blows per foot: 14, 10, 7, 4, 2, 0, 2, 4, 9, 12

REMARKS: 3/20 - Lake - 12 inches below normal pool.

## SOIL BORING RECORD

| | |
|---|---|
| BORING NUMBER | B-4 |
| DATE DRILLED | 3/11/2014 |
| PROJECT NUMBER | 114019 |
| PAGE | 1 of 1 |

▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

**SUMMARY OF HAND AUGER BORINGS**
**EMERALD LAKE DAM**
**Emerald Lake Drive**
**Fayetteville, Fayette County, Georgia**
**PGC Project No. 114019**

| Boring No. | Depth (ft) | Description | Dynamic Cone Penetrometer Depth (ft) | Dynamic Cone Penetrometer n (bpi) |
|---|---|---|---|---|
| HA-1 | 0 – 5 | FILL: Brown red silty medium to fine SAND (SM), trace clay, slightly organic at 5 feet | 1<br>2<br>4<br>6<br>8 | 12<br>11<br>15<br>7<br>3 |
| | 5 – 7 | Brown red clayey silty medium to fine SAND (SM), noticeably wetter and softer with depth | | |
| | 7 – 9 | Red brown medium to fine sandy silty CLAY (CL), very wet | | |
| | 9 | Hand auger refusal<br>No groundwater encountered at time of boring | | |
| HA-2 | 0 – 2 | FILL: Brown red silty medium to fine SAND (SM), trace clay | 1<br>2<br>4<br>6<br>8 | 9<br>11<br>5<br>1<br>2 |
| | 2 – 4 | Brown clayey silty medium to fine SAND (SM) | | |
| | 4 – 9 | Red brown clayey silty medium to fine SAND (SM), very wet below 5 feet | | |
| | 9 | Hand auger refusal<br>Time of boring groundwater at 8.8 feet | | |
| HA-3 | 0 – 4.5 | FILL: Brown red silty medium to fine SAND (SM), trace clay, softer at 3.5 feet | 1<br>2<br>4<br>6<br>8<br>10 | 5<br>9<br>10<br>8<br>15<br>14 |
| | 4.5 – 6 | Gray clayey silty medium to fine SAND (SM), possible Alluvium used as fill | | |
| | 6 – 8.5 | Multi-colored clayey silty medium to fine SAND (SM), zone of silty clay material 6 to 12 inches thick | | |
| | 8.5 – 10 | Dark gray silty medium to fine SAND (SM), topsoil, organics | | |
| | 10 | Boring terminated<br>No groundwater encountered at time of boring | | |

| Boring No. | Depth (ft) | Description | Dynamic Cone Penetrometer | |
|---|---|---|---|---|
| | | | Depth (ft) | n (bpi) |
| HA-4 | 0 – .25 | FILL: Brown silty medium to fine SAND (SM) | 2 | 6 |
| | .25 – 1.25 | BASESTONE | 4 | 7 |
| | 1.25 – 5.5 | FILL: Red brown clayey silty medium to fine SAND (SM) | 6 | 9 |
| | | | 8 | 7 |
| | 5.5 – 7.5 | Tan brown clayey silty medium to fine SAND (SM) | 10 | 8 |
| | | | 12 | 8 |
| | 7.5 – 15 | Brown gray medium to fine sandy SILT (ML), more clayey with depth, very wet | | |
| | 15 | Boring terminated | | |
| | | Time of boring groundwater at 15 feet | | |

"**n**" – Number of blows of 15 pound hammer falling 20 inches.