Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Kozisek Lake Dam Improvements
Fayetteville, Fayette County, Georgia
PGC Project No. 119193

Prepared For:

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December 17, 2019

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Attention: Mr. Marty Walden. P.E.
President

Subject: **Report of Subsurface Exploration and Geotechnical Engineering Evaluation**
Kozisek Lake Dam Improvements
Neely Road
Fayetteville, Fayette County, Georgia
PGC Project No. 119193

Dear Marty:

Piedmont Geotechnical Consultants, LLC and the undersigned are pleased to provide this report of our subsurface exploration and geotechnical engineering evaluation for the referenced project. The field study and this report were accomplished in general accordance with PGC Proposal No. P19179, dated April 10, 2019. The purpose of this geotechnical evaluation was to obtain sufficient subsurface data within the area of the dam in order to formulate recommendations to address the geotechnical aspects for design and construction of the dam improvements needed to address current deficiencies and to satisfy the requirements of the Georgia Safe Dams Program (GSDP) for a safe dam. The following paragraphs describe our understanding of the project, evaluation procedures used, our findings, and geotechnical engineering conclusions and recommendations.

PROJECT INFORMATION

Kozisek Lake Dam is an existing earthen embankment dam located along a tributary of Morning Creek in Fayetteville, Fayette County, Georgia. The dam is owned by Mr. John Kozisek and is located on a parcel of property to the south of Neely Road and to the west of Longview Road. No records of the original dam design or construction were available to PGC; however, we understand the original earth structure was constructed in the 1960’s. The dam is currently classified as a

Category I structure by the Georgia Department of Natural Resources, Safe Dams Program. A dam is classified as a Category I structure if the GSDP determines that the failure of the dam would result in a probable loss of life.

The dam is approximately 1,000 feet long with a crest width of about 12 feet and a maximum height of approximately 27 feet. The dam impounds a lake having a surface area of approximately 7.6 acres at the historic normal pool elevation and a drainage basin of approximately 413 acres (about 0.65 sq. mi.). Currently a low level 8-inch ductile iron or steel pipe with a slide gate acts as the Primary Spillway and is currently keeping the dam in essentially a drained condition. The Auxiliary (emergency) Spillway consists of three (3) 24-inch CMP culverts, crossing under Neely Road near the right abutment. There does not appear to be an existing internal seepage control drain system. The downstream slope of the embankment is steep and both slopes are unmaintained. Available topographic data indicates the upstream slope has a configuration ranging from 2.7(H): 1(V) to 3(H): 1(V) and the downstream slope ranges from 1.4(H): 1(V) to 1.6(H): 1(V).

The dam has several noted deficiencies including a steep downstream slope, unsuitable vegetation, and uncontrolled seepage when the historical normal pool is present. These deficiencies require improvements to satisfy current GSDP rules and guidelines. We understand that the proposed rehabilitation design is to include a flattened downstream slope, adjusting the crest to an uniform elevation of 954 feet, widening the crest to a minimum of 14 feet, extending the upstream slope further upstream into the lakebed, installing a new primary siphon drain and spillway, lowering the normal pool elevation to 838.5 feet, installing a new seepage collection system, and installing a new box culvert Auxiliary Spillway to replace the three corrugated metal pipes that cross under Neely Road. We understand the Kozisek Lake Dam and the Margaret Phillips Lake dam are being evaluated and designed to function in sequence.

EVALUATION PROCEDURES

To evaluate the dam's internal composition and the underlying foundation conditions, six (6) soil test borings were drilled along the crest of the dam to depths ranging from 25 feet to 55 feet below the existing crest elevation and three (3) soil test borings were drilled near the downstream toe of the dam in the south (east bound) travel lane of Neely Road. Three (3) offset borings were also drilled along the crest in order to obtain undisturbed soil samples (UD tubes) for possible laboratory testing. Twelve (12) hand auger borings and one (1) offset hand auger boring were performed near the upstream toe of the dam to evaluate subgrade conditions beneath the planned slope modifications. The soil test borings and hand auger borings were located in the field by measuring distances and estimating directions from identifiable site features. Therefore, their locations as shown on Figures 1 and 2: Site and Boring Location Plan in the Appendix should be considered approximate.

Borings B-1 and B-6 through B-9 were advanced by twisting continuous hollow stem auger flights into the ground. Borings B-2 through B-5 were advanced using mud-rotary drilling methods. At selected intervals, Standard Penetration Resistance Testing (SPT) was performed in general accordance with ASTM Standard D-1586, and soil samples were collected for visual classification. The results of the penetration tests, when properly evaluated, provide an indication of the relative consistency of the soil being sampled, the potential for difficult excavation, and the soil's ability to support loads. A more detailed description of the drilling and sampling process is included in the Appendix of this report. All mechanical borings and their offsets not converted to observation wells were filled with a bentonite/cement grout to the existing ground surface upon completion of drilling activities. The borings drilled in Neely Road were patched with concrete to restore the road surface. Fayette County assisted with traffic control signage and barriers.

Soil samples recovered during the drilling process were classified in the field 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 and groundwater conditions, are shown on the Soil Boring Records in the Appendix. Elevations on the Soil Boring Records were interpolated from the topographical contours on the plan provided to us and should be considered approximate. Figures 4 through 8: Subsurface Profiles in the Appendix present our linear (2D) interpretation of the subsurface conditions between selected borings along selected alignments.

The hand auger borings were performed by manually rotating a sharpened steel bucket auger into the ground. The soils encountered during the augering process were classified in general accordance with the Unified Soil Classification System (USCS) by our engineers. Please refer to the Summary of Hand Auger Borings in the Appendix of this report.

A limited laboratory testing program was conducted on soil samples taken from the soil test borings. A total of eight (8) UD tube samples were collected at the offset borings along the crest and archived for possible testing. Three (3) bulk soil samples were collected from the soil cuttings at selected borings along the crest. Three (3) standard Proctor compaction tests (ASTM D698) were performed on the bulk samples. Four (4) #200 wash tests (ASTM D1140-14) and six (6) in-situ moisture content determination tests (ASTM 2216) were performed on samples recovered from the standard penetration testing for general evaluation of the existing embankment materials for reuse. A Summary of Laboratory Test Results and the individual test reports can be found in the Appendix.

Three (3) temporary groundwater monitoring wells were installed and developed along the crest of the dam in borings B-3, B-4, and B-5. The wells consisted of 2-inch diameter PVC pipe with the bottom 10 feet being slotted to allow water into the well. The PVC pipe was inserted into the open borehole and sand was placed around and to just above the slotted section of the pipe, followed by a minimum of 12 inches of bentonite chips, and the remainder of the open borehole was filled with grout to the existing ground surface. The PVC pipe/well was manually bailed until the water return is clean.

SITE OBSERVATIONS

During the course of this field study spanning from late 2012 to August of 2019, PGC engineers John C. Herron, P.E. and H. Craig Robinson, P.E. visited the site and performed detailed observation of the dam’s external condition. While on-site during these various times, their following observations were noted. Physical directions are referenced while facing downstream (lake to your back).

1. The alignment of the embankment includes mostly straight segments except for a slightly curved portion near the right abutment that makes up about a third (+/-) of the length of the dam. The dam is approximately 1000 feet in length with a crest width of approximately 12 feet and a maximum height of approximately 27 feet. Neely Road follows closely the alignment of the dam to the north, separated by a ditch along the toe of the dam.

2. The upstream slope is generally relatively flat (about 3(H):1(V)) and is overgrown with small to medium trees and underbrush. The upstream slope steepens slightly near the crest of the dam to about a 2(H):1(V) slope, possibly due to cutting down or filling to raise the crest with the excess spilling towards the upstream. The downstream slope is much steeper than the upstream slope (about 1.5(H):1(V)) and is thickly vegetated by small to medium trees and brush. The crest is free from any significant vegetation and appears to be mowed regularly.

3. During our 2012 site visit, we observed seepage and flowing water along portions of the downstream toe of slope along Neeley Road.

4. The original Primary Spillway reportedly consisted of an 18-inch CMP riser connected to an 8-inch low-level DIP. The riser no longer exists, so water now flows freely through the existing low-level pipe. The upstream end of pipe is located near the center of the dam. The riser location and the downstream end of pipe were not observed. We suspect this pipe crosses through the dam and discharges to an area just upstream of an 18-inch reinforced concrete pipe (RCP) culvert that crosses under Neely Road. This low-level pipe is fitted with a slide gate in the open position. This open pipe without the riser pipe keeps the lake in an essentially drained condition during normal flows.

5. The original Auxiliary (emergency) Spillway consists of three (3) 24-inch CMP culverts beneath Neely Road at the right end of the dam. These pipes outlet into a partially lined rock channel that drains along the north side of Neely Road to the low point. A single-family residence is located near the Auxilary Spillway on the north side of Neely Road.

6. An unlined drainage ditch exists just downstream of the toe of the dam, between Neely Road and the dam. Neely Road is an asphalt road that follows the alignment of the dam and is approximately 20 feet wide with varying shoulders. Downstream of Neely Road is a low-lying wetlands area and the upper end of the Margaret Phillips Lake.

AREA GEOLOGY

The site is located in the Piedmont Physiographic Province of Georgia. The residual soils in the Piedmont are the result of the chemical and physical weathering of the underlying parent rock. The weathering profile usually results in fine grained clayey silts and silty clays near the surface, where weathering is more advanced. With depth, sandy silts and silty sands are found, often containing mica. Below the residual soils, partially weathered rock is often found as a transition above relatively unweathered rock. In local practice, partially weathered rock is arbitrarily defined as residual soils with Standard Penetration Resistances in excess of 100 blows per foot (50 blows per 6 inches), and which can be penetrated by a power auger. The natural weathering profile can be altered by water caused erosion/deposition or man-made activities.

SUBSURFACE CONDITIONS

The conditions described in the following paragraphs, and those shown in the Appendix, have been based on our interpolation of the soil boring data using generally accepted principles and practices of geotechnical engineering. However, conditions in this geology may vary intermediate of the tested locations, and even more so on previously developed property. Although individual test borings are representative of the subsurface conditions at the precise boring locations on the day drilled, they are not necessarily indicative of the subsurface conditions at other locations or other times. The nature and extent of variation between the borings may not become evident until the course of construction. If such variations are then noted, it will be necessary to reevaluate the recommendations of this report after on-site observation of the conditions.

Soil Test Borings Along Crest of Dam

Six (6) soil test borings (designated B-1 through B-6) and three offset borings were performed along the crest, at the approximate center line, of the dam to depths ranging from 35 to 53.5 feet below existing grades. All borings initially encountered fill materials to depths ranging from 14 to 26 feet below existing grades with SPT results ranging from 5 to 25 blows per foot (BPF). It was also noted at some SPT intervals that rock fragments were present in the soil samples recovered which may have amplified the SPT values. It is our opinion the fill soils encountered appear to be poorly to moderately well compacted. Several samples recovered in the fill encountered trace to moderate organics (topsoil and/or woody material). The lower consistency fill materials were typically encountered within the lower portions of the embankment, about 15 feet below the crest. The samples recovered in the fill were classified as silty sands (SM), sandy clayey silts (MH), sandy silts (ML), or sandy clays (SC).

Alluvium was encountered beneath the fill in borings B-3 and B-4 and materials described as Possible Alluvium was encountered beneath the fill in boring B-2 to depths of 2 to 10 feet below the existing fills. Alluvium is a term used to describe soil materials which have been eroded and deposited via water. The SPT results taken with in the Alluvium/Possible Alluvium ranged from 4 to 19 BPF. Several samples recovered in the alluvium encountered trace to moderate organics (topsoil and/or woody material) and contained rock fragments which may have amplified the SPT values. The samples recovered in the Alluvium/Possible Alluvium were classified as either silty clayey sands (SC) or sandy silty clays (CL).

Residuum was encountered beneath either the fill or Alluvium with SPT results ranging from 5 to 76 BPF and was predominately classified as silty sands (SM). Underlying the Residuum, partially weathered rock (PWR) was encountered in borings B-4 and B-5 with initial contact at depths of 42 feet and 37 feet, respectively, and was sampled as very dense silty sands (SM). All borings were drilled to their predetermined termination depths. Materials causing refusal to the drilling process were not encountered to the depths drilled.

Stabilized groundwater levels were measured at elevations of about 829 feet to 832 feet.

Soil Test Borings Along Neely Road

Three (3) soil test borings (designated B-7 through B-9) were drilled through the asphalt pavements along the south lane of Neely Road (near the outside edge of pavement) to depths ranging from 20 to 30 feet. The borings encountered approximately 5 inches of asphalt underlain by 4 to 5 inches of graded aggregate base (GAB). Previously placed fill was encountered beneath the GAB to depths ranging from 3 to 7 feet below existing grades with SPT results ranging from 8 to 10 BPF. Therefore, the fill soils supporting the road appear to be moderately well compacted. Slight organics (topsoil with one instance of roots) were encountered in about half of the samples recovered in the fill.

Alluvium was encountered beneath the fill in borings B-7 and B-8 and Possible Alluvium was encountered beneath the fill at boring B-9 and extended to depths of 2 to 16 feet below the bottom of the fill. The SPT results taken in the Alluvium/Possible Alluvium ranged from 3 to 19 BPF. A few samples recovered in the Alluvium encountered trace to moderate organics (topsoil and/or woody material) and contained rock fragments which can possibly amplify the SPT results. The Alluvium was classified as either poorly graded sands (SP), clayey sands (SC), or sandy clays (CL).

Residuum was encountered beneath the Alluvium/Possible Alluvium with SPT results ranging from 7 to 59 BPF and was classified as silty sands (SM). Partially weathered rock was encountered at borings B-8 and B-9 with initial contact at depths of 27 feet and 16 feet, respectively, and was sampled as very dense silty sands (SM). Refusal materials were not encountered in any of these borings to the depths drilled.

Stabilized groundwater levels were measured at elevations of about 828 feet to 832 feet.

Hand Auger Borings Along Upstream Toe of Dam

Twelve (12) hand auger borings (designated HA-1 through HA-12) and one offset boring (designated HA-9A) were performed near/just upstream of the upstream toe of dam. Fill associated with the dam was encountered in hand auger borings HA-5 through HA-8, HA-9A, and HA-10 through HA-12 at the surface (except at HA-9A where the fill was encountered beneath 2 inches of Alluvium) to depths ranging from 6 to 24 inches below existing ground surface and was classified as silty clays (CL or CL-ML), clayey silts (ML), poorly graded sands (SP), or silty sands (SM).

Alluvium was encountered in all hand auger borings either at the surface or beneath the fill to depths ranging from 3 to 96 inches below existing ground surface. The Alluvium was classified as either silty clays (CL or CH-MH), sandy clays (CH-SC), clayey silts (ML or MH), silty sands (SM), clayey sands (CL), poorly graded sands (SP), or well graded sands (SW).

Residuum was encountered in hand auger borings HA-1 through HA-3, HA-8, HA-10 and HA-11 beneath the Alluvium while all other hand auger borings did not encounter Residuum as the Alluvium was not able to be fully penetrated by the hand auger process. Where encountered, the Residuum was classified as silty sands (SM) or sandy silts (ML).

Stabilized groundwater levels were measured at depths ranging from 4 inches to 36 inches below existing ground surface and levels were likely influenced by the proximity to flowing/standing water at the time of this study.

We refer the readers to the Soil Boring Records and Summary of Hand Auger Borings included with this report. These documents provide a more detailed presentation of the materials encountered at depths and their respective Unified Soil Classifications (USCS), SPT values, and other notable observations during the drilling operations and soil stratification. Also, please find Figures: 4 through 8 depicting subsurface profiles which represent a linear array of specific boring data on or near the selected line. We note that the interpretation of data between actual boring locations is very subjective and results in an averaging or straight-line interpretation of data using our best engineering judgment. We note that the transitions between different soil strata are generally less distinct than depicted on the Soil Boring Records and Subsurface Profiles. While these profiles are useful in predicting the subsurface conditions between boring data, the profile may not accurately represent actual subsurface conditions. Groundwater levels noted on the Soil Boring Records, Summary of Hand Auger Borings, and Subsurface Profiles are subject to climatic and seasonal changes and variations in lake levels. As such, groundwater levels may vary from the levels presented in this report.

CONCLUSIONS AND RECOMMENDATIONS

The following paragraphs describe our geotechnical engineering conclusions and recommendations based upon our interpretation of the boring data, our site observations and our understanding of the planned improvements of this dam to correct apparent deficiencies, which include significant spillway modifications. We understand that Walden, Ashworth and Associates, Inc. is designing a new 12-inch ductile iron siphon to replace the existing broken Primary Spillway pipe. The three existing CMP Auxiliary (emergency) Spillway pipes will be replaced with a new double 6-foot wide x 3-feet high box culvert. The design will also include improvements to the embankment dam and an internal drainage system. PGC has provided consultation and verbal recommendations to you prior to issuing this report. We remain available to assist with the development of plans and technical specifications. No borrow source has been identified, evaluated and approved prior to the issuance of this report. A suitable borrow source and a disposal site will be needed for this project.

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

If additional problems that are not currently evident are observed during the course of the ongoing design history of this project and prior to construction, we should be contacted so that we can evaluate the current conditions of the dam and provide additional input, if needed. We recommend that engineers and technicians of our staff monitor and evaluate this dam during construction to assure that the recommendations contained in this report and as incorporated in the final plans and specifications are properly implemented.

GENERAL ASSESSMENT

After completion of our field studies and engineering evaluation of the information collected, our general impression is that this embankment dam is in poor condition. The dam has numerous deficiencies related to the spillway system, the steep downstream slope configuration, negatively impacting slope stability, embankment maintenance and performance monitoring, and potential/apparent uncontrolled seepage that should be addressed by the planned engineered improvements.

As previously stated, the downstream slope is overly steep, irregular and is likely experiencing uncontrolled seepage along the toe and in the area surrounding the existing spillway pipe. Unsuitable trees and underbrush exist on the upstream and downstream slope. It is our opinion the trees and brush should be removed, the slopes flattened, and a suitable grass established and maintained. An internal drain system should be incorporated into the embankment modifications for seepage collection.

The existing Principal Spillway Pipe allows the current pool level to remain at an elevation of about 830 feet, which is an essentially drained condition. The historical normal pool elevation was at about 849 feet. The Auxiliary Spillway system needs to be upgraded. The existing pipes should be removed and replaced.

Based on our evaluation of the mechanical soil test boring data obtained beneath the dam crest, the quality and composition of the existing embankment fill materials vary somewhat, both horizontally and vertically, within the embankment section. Standard Penetration Testing data and samples obtained indicate random fill quality and soil compaction with varying amounts of intermixed organics, rock, and sandy soils. The fill consistency in some borings decreases with increasing depth, while the fill consistency in other remains fairly consistent with depth. The lowest consistency fill materials were encountered in borings B-3 and B-4 at depths ranging from approximately 17 feet to 26 feet below the ground surface, or approximately 10 feet or less above the transition from the embankment fill to the underlying Alluvium. These conditions suggest that a less compacted/thickened soil “bridging” layer might have been placed above the Alluvium due to poor and/or at unstable ground conditions that might have existed at the time of construction. The borings also indicate that a considerable zone of Alluvium was left under the embankment footprint within the floodplain limits. These same alluvial materials were encountered in boings B-7, B-8 and B-9 beneath Neely Road. The alluvial soils contained varying amounts of sand and where encountered were deepest in borings B-3, B-4 and B-8. These materials often included organics. No evidence of a man-made keyway, typically constructed about middle of the embankment to help control seepage, through the upper, more permeable material, was encountered during drilling. In order for the seepage collection system to function as planned, the alluvial materials located along the downstream toe area will need to be removed and replaced. The Alluvium/Possible Alluvium appears to exist between approximate elevations 820 feet and 837 feet beneath the existing dam crest and approximate elevations 815 feet and 831 feet beneath Neely Road and along the downstream toe of the embankments. Stabilized groundwater levels encountered at the wells set at borings B-3 through B-5 were measured to be at an elevation of approximately 829 to 832 feet with the lake essentially drained.

The hand auger borings performed along the upstream toe of the existing dam and in the lakebed, which is within the area where the new upstream slope projection will extend, encountered variable depth, material type, and consistency of alluvial materials. In general, hand auger borings HA-4 through HA-7, HA-9, HA-9A, and HA-12 encountered the deepest alluvial deposits to depths exceeding 8 feet. All of these hand auger borings were abandoned prior to fully penetrating the Alluvium due to unstable ground conditions and excessive groundwater flow which caused the borehole to collapse. Several of the borings were probed past the termination depth with a 9.5-foot rod. While probing below the termination depth in borings HA-6 and HA-7, the materials at 9 and 8.5 feet, respectively, felt as though they were firming up, possibly indicating the transition between the Alluvium and Residuum, but the materials in boring HA-5 probed softly to a depth of 9.5 feet (full length of rod) and never firmed up. The upper materials typically consisted of very low consistency silts/clays transitioning to somewhat sandier soils and clean sands with gravel at depth. Groundwater was typically encountered between 0.5 feet and 3 feet below the ground surface.

Even with the deficiencies and marginal to poor subsurface conditions noted previously, we are of the opinion the entire dam does not need to be removed and replaced in order to create a uniform/stable embankment dam. However, complete undercutting of the existing fills and underlying alluvial materials and replacement with new structural fill placed in accordance with subsequent sections of this report are recommended in the areas supporting the recommended internal seepage collection system along the downstream. In addition, we recommend all alluvial materials underlying the recommended upstream slope modifications be undercut and replaced with new structural fill materials. Some additional undercutting/replacement and/or stone stabilization is expected beneath and downstream of Neely Road where the new siphon pipe and energy dissipation structure will be located. We understand Neely Road must remain as located and operational during construction. Therefore, all embankment modifications will extend from the south edge of Neely Road in an upstream direction.

Positive dewatering and stream diversion operations will be required during construction so that the below grade activities can be accomplished in the dry. Maintaining the lake in a drained condition throughout construction will have a direct positive impact on the dewatering and below grade construction efforts.

The actual amount and location of embankment/foundation seepage was difficult to determine due to the lake currently being in an essentially drained condition. When the lake was initially inspected in 2012, we recall the downstream toe area along Neely Road being wet. In general accordance with GSDP guidelines, we recommend a seepage collection system consisting of an all aggregate Toe/Foundation/Blanket drain connected to a full height (to normal pool) Chimney Drain be incorporated into the planned dam improvements.

We envision the following major construction tasks will be required for implementation of the planned dam improvements. This list is somewhat abbreviated and may not be an all-inclusive lists of the required tasks. These are not presented in any order of importance or sequencing, and while listed separately will often overlap with other tasks.

- Identification and approval of an offsite borrow source. Identification of a disposal site for unsuitable/undercut materials.
- Draining of the lake and maintaining a drained/lowered pool level condition during construction. A minimal pool for water quality may be considered. The presence and location of any standing pool could have negative impact on the dewatering and construction efforts.
- Construction of a temporary coffer dam(s) to protect the upstream embankment improvements. Diversion of stream flows through the construction site.
- Installation of temporary dewatering systems, both upstream and downstream of the existing embankment.
- Removal of all unsuitable vegetation on the embankments and impacted areas.
- Grout abandonment of the existing spillway pipe.
- Prescribed excavation of downstream slope of existing embankment, undercutting and replacement. Stockpile and conditioning of existing fill materials.
- Installation of foundation drain will need to be installed in vertical segments due to depth of undercutting and backfill.
- Breach and remove existing Auxiliary Spillway Pipes.
- Installation of new Primary Siphon Spillway and Auxiliary Box Culvert Spillway, including riprap armoring.
- Earthwork activities to construct upstream and downstream slopes.
- Construction of the upstream wave protection.
- Final grading of slopes and permanent grassing.
- Installation of instrumentation.
- Replacement of asphalt/stripping.
- Installation of Guardrails, signage and redirect traffic.

TEMPORARY GROUNDWATER AND SURFACE WATER CONTROL

Positive groundwater and surface water control will be necessary during construction. Inadequate control of groundwater and surface water will adversely impact subgrade preparation and other activities that will take place in conjunction with this project. These activities will include, but are not limited to, undercutting of the alluvial and existing fill soils, grout abandonment of existing spillway pipes, initial fill placement, and foundation drain construction. The site and boundary limitations, sequencing of construction, depth of undercutting and rate of construction should all be considered when determining methods and plans for dewatering and stream control. It is possible the existing low-level drainpipe can be initially used to help control stream flow, but eventually pumping will be required once the existing pipe is grouted. We anticipate considerable pool functions are probable given the small diameter of the existing pipe. Pumping will become necessary and continue until at least the siphon is made functional. Development of dewatering and surface water control plans and the successful implementation of the accepted plans are considered critical to the successful completion of this project. Often these efforts are performed in stages/phases to account for varying construction tasks, site limitations and access.

The contractor should be advised of the hydrologic characteristics of the watershed basin and the hydraulic capacity of the existing pipe and new 12-inch siphon so that they may be able to determine what storm event/water level to provide protection of their subsequent work, the height and placement of temporary coffer dams, pumping capacity, and other associated erosion/sediment control features. Redundancy in pumping capacity and/or backup pump equipment should be considered. Project specifications should require that the contractor submit a detailed diversion plan with all anticipated phases for the engineer's review and approval prior to implementation.

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. Limited explorations have been performed upstream and downstream of the existing embankment. Based on the hand auger boring data, the ambient groundwater conditions upstream of the dam are essentially the same as the current ground surface to about -2 ft below the ground surface between elevations 830 to 833 feet with the lake essentially drained. Based on the mechanical borings, the ambient groundwater level beneath the existing embankment ranges from approximately elevation 829 to 832 feet and beneath Neely Road from approximately elevation 828 to 830 feet. As such, we anticipate extensive dewatering efforts will be required to lower the current groundwater levels by as much as 8 to 20 feet below the general floodplain level. Actual groundwater conditions will also depend on the time of the year, prevailing weather patterns at the time of actual construction and the time of construction lake levels. It is our opinion that Kozisek Lake should remain essentially drained and Margaret Phillips Lake lowered as much as possible during construction. We note that cooperation between contracts and contractors can significantly impact the requirements for stream diversion and dewatering efforts for both projects.

Some of the difficulties in dealing with the groundwater are directly impacted by the depth below prevailing groundwater levels, soil type and consistency, and the time required for the particular element of construction to be accomplished. Based on our previous experience in dam construction, we anticipate a vacuum well-point system, possibly multiple and overlapping systems, will be required to satisfactorily lower the groundwater to the depths discussed in order to accomplish the undercutting and backfilling, the construction of the lower Foundation Drain component, and the new siphon. Given the magnitude of dewatering anticipated, it is our opinion a functioning well-point system(s) would be considered the primary dewatering effort. Even with properly functioning well-point system(s), dewatering operations may require supplemental dewatering operations using cased wells, shallow sumps and focused pumping.

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 be the contractor’s responsibility, and that these documents establish a performance criterion for our assessment of the effectiveness of the dewatering system actually installed. Typically, the performance criteria require 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 in locations selected by the geotechnical engineer and to target areas where groundwater is anticipated to be problematic. In addition, the contractor should be made aware that adjustments to the dewatering system may be necessary if areas of deeper excavation for undercutting or drain construction 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 3 to 5 feet above the prevailing stabilized groundwater levels. Due to the project's proximity to residences, the use of "quiet" pumps and other noise buffering devices should be anticipated. 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, comments and 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 also be properly abandoned or incorporated into permanent construction so as to not negatively impact the dam's performance (post construction) during operating pool levels.

EMBANKMENT MODIFICATIONS/CONSTRUCTION

Our observations and the topographic survey of existing conditions provided indicates that the downstream slope of this dam is very steep, irregular, and poorly unmaintained. The upstream slope is flatter and also poorly maintained.

Within the available limits of construction and to the greatest extent practical, we recommend that the renovation plans include modifying the overall geometry/configuration of this dam to a uniform configuration. We recommend the upstream and downstream slopes be flattened, as needed, to create a uniform embankment cross-section/configuration using 3(H):1(V) or flatter slopes by primarily adding additional earth or by cutting when necessary. We understand the downstream toe of the dam will remain essentially at the same location and follow Neely Road. The centerline of Neely Road will remain essentially the same and the roadway section widened as needed to provide a minimum asphalt pavement width of 19 feet with 7.5 feet of shoulder to each side. We also understand that the crest of the dam will be widened to a minimum width of 14 feet at elevation of 854.0 feet. Therefore, the majority of the embankment dam earthwork improvements must occur in the upstream direction, with a significant amount of the existing embankment being cut down and a substantial amount of new earthwork occurring within the lakebed.

During the course of this study, Walden, Ashworth and Associates, Inc. provided for our use the Terramark Land Surveying, Inc. topographic survey of the existing embankment as well as their preliminary grading plans which included the planned crest widening and anticipated slope modifications. Four transverse profiles were developed at our request which presented both existing and proposed embankment topography. Using this information, PGC determined the amount of remedial shaping/excavation needed to create the desired embankment configuration, to provide suitable cover to the recommended internal drainage system, and to take into account a minimum equipment working space for small conventional earthmoving equipment.

To accomplish a uniform embankment configuration/geometry, we recommend a significant portion of the existing downstream slope be removed along most of the dam length to provide a uniform shaped slope on which to construct the Chimney Drain and then sufficient earth cover to protect it. Since the downstream slope is irregular along its length, we recommend the downstream slope shaping/excavation be uniform about the new dam centerline. As such, we recommend the downstream slope of the existing embankment be uniformly shaped/excavated to a uniform 1.5(H):1(V) configuration with the upper or top limit of the excavation placed a maximum of 15 feet downstream of the proposed embankment centerline at elevation 847 feet. Removal of this portion of the existing embankment will allow for the construction of the new 3(H):1(V) downstream slope and allow that a minimum 3 feet (vertical) cover over the Chimney Drain, and a minimum lift (horizontal) width of 12 feet equipment working space beyond the Chimney Drain at all locations. The upstream slope of the existing dam should be shaped (beginning generally near the toe of slope) downward on a 3(H):1(V) or slightly steeper slope to the approved residual subgrade within the limits of the planned slope modifications.

Minimal shaping/excavation along the upstream slope below elevation 847 feet to the existing toe of slope will also be required. The existing upstream slope is much flatter than the existing downstream slope, so the recommended shaping/excavations into the existing upstream slope are considered less drastic and will generally occur almost entirely near the existing toe of slope.

In all areas where remedial slope shaping/excavation operations for drain installation and/or the new 3(H):1(V) filled slope configuration extends beyond the existing dam footprint, complete undercutting of existing fills within the prescribed slope zones and all underlying alluvial soils down to the residual subgrade should be performed to provide a stable subgrade to support placement of the new earthwork. As a general recommendation, all Alluvium should be undercut and removed to expose the underlying residual subgrade. Based on the hand auger boring data along the upstream slope, undercut depths ranging from approximately 0 to 2 feet at hand auger borings HA-1 and HA-10 to possibly in excess of 10 to 12 feet near hand auger borings HA-5, HA-6 and HA-7should be anticipated. Based on the results from the mechanical boring data, we anticipate undercut excavations ranging from essentially nothing to in excess of 19 feet are possible to fully remove the underlying Alluvium. Areas to be undercut must be adequately dewatered in advance of beginning the undercutting operations. The contractor must demonstrate to the engineers that the areas to be undercut are sufficiently dewatered before they will be allowed to begin the remedial undercutting operations. Actual construction conditions may be encountered which could require excavations to be extended deeper than anticipated. Where possible, undercutting should be extended to the fullest horizontal limits as defined by extending the 3(H):1(V) finished slope projection to the approved subgrade level with a reasonable exit slope back to existing grades. Neely Road will limit the extent of undercutting in some downstream areas. Where Neely Road conflicts with the prescribed limits of undercutting, we recommend the excavation be terminated based on a 1(H):1(V) slope projected from the edge of pavement down to the approved residual subgrade. Traffic control barriers will be necessary. Given the access limitations and the potential dewatering challenges, we suggest as much as practical the undercutting operations begin near the ends and progress in manageable pieces or strips, so that the exposed residual subgrades can be covered as quickly as possible, and allowing subsequent undercutting/backfilling operations to be staged from the previous area. We refer the reader to the Summary of Anticipated Undercutting Conditions included in the report Appendix.

During the undercutting and backfilling operations downstream of the existing dam, it will be necessary to perform earthwork and some drain construction operations almost concurrently. To fully construct the drain system components, it will be necessary for the earthwork operations to be suspended at a temporary pad level so that the recommended Blanket Drain and Rock Toe Berm/Toe Ditch and the lower portion of the Chimney Drain can be constructed. We recommend the temporary pad level be determined based upon the lowest subgrade level for the Toe Ditch located along the south side of Neely Road, which will vary as Neely Road exists in both a vertical/horizontal curve alignment. The temporary fill pad level should be graded to slope up gradient from the Toe Ditch towards the prepared 1.5(H):1(V) existing embankment slope on a 1-2% slope. The completed temporary fill pad level will be determined the top elevation for the Toe/Foundation Drain trench and will be the subgrade supporting the Blanket Drain and Rock Toe Berm/Toe Ditch. This temporary fill pad should be completed in its entirety upstream to downstream so that the drain system can be completed, and subsequent earthwork started to provide cover and protection. The contractor will need to provide additional measures to prevent contamination of the exposed Rock Toe Berm/Toe Ditch during subsequent earthwork operations.

Prior to beginning any earthwork operations, the entire embankment should be stripped of all vegetation, stumps and associated roots. The prescribed excavations will likely remove most of the stumps/root system from the downstream slope. The upstream slope should be thoroughly stripped and grubbed before beginning earthwork. The geotechnical engineer should evaluate and approve all exposed subgrades prior to beginning subsequent work.

All undercutting and subgrade preparation operations should be witnessed by the geotechnical engineer. Actual undercut limits will be determined at the time of construction by the geotechnical engineer. All final subgrade preparation should be made with a smooth blade or straight edge on an excavator bucket to remove all loosened/disturbed materials.

For the purposes of this project, we have used the terminology "select" and "common" to represent two different classes of soil materials and their general placement within the embankment. As a minimum, "select" soils should be used to fill all undercut areas back to at least original grades and/or to the recommended downstream pad/platform grades supporting drains, and the upstream slope/embankment fills up to elevation 843 feet. All other areas can be backfilled using "common" soils. "Select" soils are defined as earth materials having USCS designations CL, ML and SC and "common" soils can be all 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.

Prior to beginning construction, a source(s) of suitable embankment fill materials will need to be located and approved by the geotechnical engineer. Most of the soil materials encountered in the mechanical soil test borings drilled during this investigation and described as "fill" visually appear suitable for re-use as "common" structural fill; however, much of this material will require mechanical manipulation and moisture conditioning (drying) before re-use. For moisture conditioning to be efficiently accomplished, a well-drained and sufficiently large enough area away from the dam footprint will need to be set aside so that these excavated materials can be thinly spread and manipulated with tractor pulled disc harrows or dozers. The upstream left and right shorelines may be suitable for these type operations, if accessible. We note that our assessment of the existing fill materials is based on limited testing of SPT samples recovered and compared to laboratory Proctors developed from bulk samples taken from borings B-1, B-4A and B-6. We expect actual conditions within the embankment to vary based on the laboratory test results. In-situ soil moistures range from approximately 2 to 12 percent over the Proctor's optimum moisture contents. We refer the reader to the Summary of Laboratory Test Results and the individual test reports in the Appendix. The determination of suitability of the existing fill materials should be made by the geotechnical engineering at the time of construction. Materials described as Alluvium are not suitable for re-use in the dam. Excess or unsuitable soils cannot be wasted onsite.

All fill materials placed should consist of clean soils, free of deleterious materials and rock fragments larger than 3 inches in diameter. The compacted soil should 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 or above the soil's optimum moisture content. Fill materials placed within Neely Road should be placed at a more restricted moisture content range of +/- 1 percent of the soil's optimum moisture content and at a minimum of 98 percent of the soil's maximum dry density within the upper 2 feet below pavements for improved support of the pavement section. Due to the limited space available at the dam site, moisture conditioning of fill materials will likely need to be conducted away from the dam site at the borrow site prior to placement in the dam footprint.

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 temporary fill surfaces. When temporary fill slopes are unavoidable, they should be constructed no steeper than 5(H):1(V). The new fill materials 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 or truck hauling, 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 become necessary to blade off these materials, or to scarify, moisture condition, and re-compact these materials in-place, prior to the placement of subsequent fill layers.

In areas where existing or man-made excavation slopes or temporary fill slopes are steeper than 5(H):1(V), mechanical benching into the soils along the slope surface will be necessary for all areas not covered by drain aggregates to adequately bond the new fill to the underlying surface. Where fill is placed around conduits, it will be necessary to maintain the level of fill approximately equal on both sides of the conduit during placement to prevent possible lateral displacement and/or 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, or vibratory pad foot trench rollers will be required.

During the earthwork/fill placement operations, we recommend the upstream and downstream embankment surface be sufficiently overbuilt so that the final slope surfaces can be cut/trimmed to a final grade (pre-topsoil placement) that is well compacted. The final constructed structural fill embankment should result in minimum 3(H):1(V) slopes, upstream and downstream, and a minimum crest width of 14 feet. If possible, the crest should be sloped with a minimum 1-2% cross-slope grade towards the lake (down towards the upstream direction) to minimize surface flows across the longer downstream slope section. If wave protection is needed, we recommend it be installed into an excavated notch after the fill embankment section has been raised to full width at least 1 to 2 feet higher than the armored section. Excavated soil materials from wave protection construction can be used in the embankment.

SIPHON SPILLWAY

It is our understanding that a new 12-inch diameter ductile iron siphon pipe will be installed as the Primary Spillway, which would eliminate the need to excavate entirely through the embankment to its base to install a new low-level pipe. At the upstream side of the dam, an inlet section using a perforated/screened pipe segment (trash rack) will be placed with the conduit attached through small concrete pedestals to the upstream face of the dam or buried at a shallow depth beneath the slope face. The conduit will then extend across the crest of the dam at the new normal pool level and is then buried at a shallow depth on the downstream slope. The conduit will then turn to run beneath Neely Road to the discharge location in a concrete impact structure. A full concrete encasement will be placed where the conduit extends through the dam in a nearly horizontal position near the normal pool level. Concrete collars at each joint and soil backfill will be utilized on the downstream face where the pipe is buried at shallow depth. Since the potential for seepage along this conduit system is relatively minor, especially considering the significant horizontal distances involved for this relatively small embankment, no separate filter collar is recommended. However, where the siphon will cross the Chimney Drain, we recommend the Chimney Drain section be raised to at least 2 feet above the pipe, extending +/- 5 feet to each side of the pipe. If the siphon intersects the coarser drain materials, care should be taken be ensure the integrity of the drain is not compromised while providing necessary drainage to intercept any seepage at this penetration.

The backfill across the crest of the dam will be critical; we recommend that the more select clayey materials be utilized. These materials should be placed wet of optimum, in thin lifts, and well compacted. Sloping back the sides of the excavation will be required to provide adequate bonding with the existing embankment materials.

ABANDONMENT OF EXISTING LOW-LEVEL CONDUIT

Based on our observations, there is one +/- 8-inch diameter low level pipe visible at the upstream toe of the existing embankment. The downstream end of the pipe has not been observed. The condition of the pipe is unknown. We recommend the existing low-level drainpipe be abandoned in place utilizing low pressure grouting techniques in lieu of excavation and removal. This would require that both ends of the conduit be exposed, the interior of the pipe cleaned out and pressure washed to remove any sediment and other infill materials, and verification that the pipe is reasonably intact and that pressure grouting is an acceptable method. If these conditions are satisfied, our experience would indicate that it would likely be more cost effective to grout this pipe in place, rather than to utilize direct removal. Where both ends of the conduit are exposed, grout is typically pumped through a bulkhead from the downstream end, and allowed to migrate the entire length of the pipe to the upstream end, which is also either bulkheaded or utilizes the existing slide gate, if in place, to contain the flow. We understand there could be an open riser section connected to this pipe. If a bulkhead is used, temporary venting will be required. If the existing gate can be utilized, it should be possible to leave the gate partially open until the grout flow is observed, and then to shut the gate. Once clean grout is seen from the vent at the upstream end, the vent is closed, and additional pressure applied to the grout to help assure that any voids around the conduit are filled. The downstream end of this conduit should also be further protected with a simple filter collar, which can likely be incorporated into the Chimney Drain. If it is determined that this low-level pipe is not reasonably intact, and that grouting would likely not be sufficient, complete removal would be necessary. If removal is required the embankment would need to be breached, the pipe removed, and the breach backfilled with “select” fill. The breach width at the pipe elevation would need to be wide enough to operate typical compaction equipment (minimum 15 feet) and the side slopes of the breach would be required to be sloped up to the crest no steeper than a 1.5(H):1(V) configuration. Should remediation of the subgrade be required to place fill, undercutting would be necessary, thus deepening and subsequently widening the breach. The removal of this pipe would result in a significant portion of the embankment being removed. Dewatering would likely be required while the dam is breached, undercut, and backfilled.

REMOVAL OF EXISTING AUXILIARY SPILLWAY PIPES

The three existing CMP Auxiliary Spillway pipes located near the right end of the dam should be removed as part of the planned dam modifications. A common excavation should be used to remove these pipes. The excavation should extend to the widest dimension of the outside pipes plus 5 feet and include a minimum 1.5(H): 1(V) entrance/exit slope. While no geotechnical data is available for this area, we anticipate some remedial undercutting and subgrade preparation will be necessary. For planning purposes, we recommend the excavation to remove the pipes extend a minimum of 25 feet wide down to a minimum elevation 847 - 848 feet. A deeper and wider excavation may be required based on the actual conditions encountered. The resulting excavation should be backfilled with select fill materials.

INTERNAL EMBANKMENT DRAINS

The following recommendations concerning the embankment and foundation seepage collection system are based the 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 collected and its exit from the embankment controlled. Seepage left uncontrolled tends to worsen with time and can cause internal erosion of the embankment and foundation soils. In addition, uncontrolled seepage presents maintenance difficulties due to the general softening of ground and could negatively impact overall slope stability. For all of the drainage system components described herein, PGC will continue to work closely with WA&A to assist with the development of details that will be incorporated into the construction documents. This report section is intended to provide a general overview of the drainage system components and requirements.

General widespread embankment seepage was not observed during our field study. Some wet areas were observed in close proximity to the existing storm drain beneath Neely Road, located about mid-dam. Historical observation of seepage has been documented. The impoundment has been in an essentially drained condition since our involvement began in late 2012. Even with the lake being drained these last several years, groundwater levels remain essentially level with the original floodplain both upstream and downstream of the embankment. Furthermore, the borings encountered sandy Alluvium beneath a significant portion of the existing embankment and the proposed modified embankment footprint that by their nature of layered deposition tend to be much more permeable than the embankment fills and underlying residual materials. The conditions observed and encountered in the borings exhibit properties indicative of seepage. We anticipate the inherent seepage conditions and quantities to increase with the re-impoundment of the lake to the sustained permanent new normal pool. In keeping with GSDP guidelines and good engineering practice, we recommend the entire embankment be modified to include a comprehensive seepage collection system, as would be typically required on Category I dams in Georgia. The planned embankment improvements and proposed geotechnical studies were discussed with the Georgia Safe Dams Program prior to the field studies and extensive slope modifications with an internal seepage collection system was considered mandatory modifications to the project, thus allowing Piedmont Geotechnical Consultants, LLC to forgo extensive laboratory testing and detailed slope stability evaluations and submittal of a formal Engineered Calculations Report for this project. Seepage collection and surface drainage for Kozisek Lake Dam is somewhat complicated by Neely Road, located immediately downstream of the current embankment, and the presence of the swampy, low-lying, headwaters for Margaret Phillips Lake located just north of Neely Road.

The recommended seepage collection system for this dam includes a combined Foundation/Chimney/Blanket Drain placed generally parallel and along the downstream toe of the recommended 1.5(H):1(V) shaped embankment slope at the prepared pad level or on approved residual materials. The proposed drain should extend laterally (right to left) to the fullest limit possible as defined by the new normal pool elevation of 838.5 feet as projected to the downstream residual abutment grade. In lieu of perforated pipe embedded in the drain system with discrete outlets, we recommend the drain system include a continuous Blanket Drain section placed on the prepared pad grade that is connected to a Rock Toe Berm and a rock lined Toe Ditch. By eliminating the internal piping, the water levels internal to the embankment can be lowered another few feet and can outlet into the Rock Berm/Toe Ditch and then drain to the new storm drainpipe beneath Neely Road. The storm drainpipe beneath Neely Road should be placed as low as possible to prevent ponding of water in the ditch on the dam side, but still provide a positive drainage slope beneath Neely Road.

To collect seepage from beneath the existing embankment, we recommend a 3 feet wide, vertical oriented Toe/Foundation Drain component be constructed internal to the dam. The Toe/Foundation Drain should extend through the lower existing fills and Alluvium allowed to remain and then an additional 2 feet minimum into the underlying residual materials. The Foundation/Toe Drain alignment will generally follow the toe of the excavated and shaped 1.5(H):1(V) temporary slope. The Toe/Foundation Drain will be constructed to the recommended pad grade after undercutting. Due to the anticipated undercut depths downstream of the existing embankment, it will be necessary to construct the vertical oriented Toe/Foundation Drain component through the deeper undercut areas (areas undercut more than about 6 feet below the Toe Ditch subgrade) in multiple lifts to prevent having to create an overly deep and unsafe excavation from the completed the final pad grade. As such, portions of the Foundation Drain will need to be installed concurrently with the undercut/backfill operations. A maximum trench depth of 4 feet is recommended for these interim lifts. The Foundation Drain alignment may also need to be shifted further downstream from the 1.5(H):1(V) toe of slope from about +/- 5 feet to up to as much as +/-15 feet in an effort to minimize digging through too deep a profile of the weaker, potentially less stable, existing embankment fills and underlying alluvial soils. Where the Toe/Foundation Drain construction can commence without need for interim lifts, the drain will generally follow the toe of the 1.5(H):1(V) slope with a 5-feet minimum horizontal offset. In this situation, the Foundation Drain should extend a minimum of 2 feet into residual materials. Where the Toe/Foundation Drain component transitions out of the deeper undercut area and into areas with less than 6 feet of new fill depth to the pad level, we recommend the minimum Toe/Foundation Drain penetration into the underlying residual materials increase up to a maximum depth of 6 feet into the residual soil strata. The geotechnical engineer will evaluate this condition during undercutting operations and provide recommendations at the time of construction.

Where multiple lifts are required, careful control and monitoring of the as-built drain alignment is needed to ensure that the subsequent lifts line up appropriately. Excavation and construction of the Toe/Foundation Drain component should be performed and accomplished in short manageable lengths to allow installation of the aggregate layers to be accomplished while lessening risk of trench collapse. The Foundation/Toe Drain trench should not be allowed to remain open at the end of each shift. Temporary measures to maintain aggregate layer separation and to protect the leading edges of construction should be anticipated to prevent contamination/damage of the drain materials. Contamination/damage of aggregates and to the filter fabric could require their removal and replacement. Positive dewatering during Toe/Foundation Drain construction should be anticipated during excavations below approximate elevation 832 feet based on current groundwater conditions.

As stated previously, the maximum interim Foundation Drain trench depth should be limited to 4 feet. A straight edge blade on the excavator bucket (with no holes between the teeth) is required to adequately clean the approved residual subgrade and/or previously placed Foundation Drain lifts when re-excavated to minimize the need for laborers to enter the excavation for final cleanup. Sand placed to backfill the trench should not exceed 2 feet thickness (loose). By limiting the lifts of sand 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 fine aggregates, and then to fill the trench and again compact the remaining aggregates in reasonable (maximum 2 feet thick) lifts as needed. There should never be a need for individuals to be in a trench that is too deep utilizing this approach. This process is repeated until the Foundation Drain sand reaches 6 feet below the pad level. We recommend that the Foundation Drain portion of this system below the Toe Drain consist entirely of natural sand meeting ASTM C-33 standard gradation (manufactured sand is not allowed). A 50/50 "blend" by volume of ASTM C-33 sand and washed #89 stone can be substituted. Blended fine filter materials (aka "Blend") are recommended in other portions of the seepage collection system where coarse aggregates are included to lessen the number of different aggregate layers and the overall thickness.

Prior to completing the Toe/Foundation Drain component, we recommend the inclusion and embedment of suitable filter fabric materials to provide filtration and separation of the coarse aggregates in the Toe Drain from the surrounding soil materials. The filter fabric should be constructed/draped into both sides of the excavated trench in an "open bottom" configuration. The filter fabric should be embedded 18 to 24 inches into the upper portion of Foundation Drain sand. Therefore, considering that the overall Toe Drain component has a recommended height of 4.0 feet, this would require that the uppermost portion of this drain system have a minimum trench depth of at least 5.5 to 6 feet, with the top of the lowest lift of Foundation Drain sand prior to constructing the Toe Drain component maintained at least 18 to 24 inches below the base of the Toe Drain to allow the fabric to be adequately embedded into the remaining Foundation Drain material (sand or blend).

The top of this Toe/Foundation Drain should also have an open top configuration for the filter fabric to allow direct contact between the recommended Blanket Drain with the top of the completed Toe/Foundation Drain. Proper aggregate filter transition must be maintained throughout the system. As such, no #89 or #57 stone should be placed in direct contact with the soil subgrade. Sufficient quantities of each drainage aggregate for the Foundation/Toe Drain construction should be stockpiled on site to allow the contractor to immediately place these materials as sections of the trench excavation are prepared. Delays could lead to trench sloughing or impacts to subgrades 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 fabrics such as GEOTEX 180EX, Tencate-Mirafi 180N, TerraTex N08, or approved equivalent. The contractor should be required to submit their fabric and aggregate information to the engineer for review and approval prior to implementing them into the construction. It is critical for the fabric to be placed in intimate contact with a relatively undisturbed soil interface to prevent clogging of the fabric. No fabric should be placed in or on a wet or muddy excavation/subgrade. 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. Where laps are required between separate pieces of fabric, the fabric should be overlapped by at least 24 inches. Where flow is anticipated, the fabric should be shingled with the flow. The fabric should be firmly pinned to the soil subgrade prior to aggregate placement. We request the Minimum Average Roll Values (MARV) sheets referencing each fabric roll number to be used at the project be provided to the engineer at the time of delivery to the project.

Once the Toe/Foundation Drain is completed up to the pad grade, the lower portions of the Chimney Drain and associated Blanket Drain/Rock Toe Berm/Toe Ditch can be constructed on the prepared downstream slope and temporary pad surfaces. The aggregate layering is consistent within the Blanket Drain, Rock Toe Berm and Rock Toe Ditch components, and it is our expectation this seepage collection system at the temporary pad level can be constructed in its entirety, full width (upstream/downstream), in manageable strips provided adequate setbacks for each aggregate layer is maintained between adjoining sections.

The recommended Chimney Drain should consist of natural sand meeting ASTM C-33 standard gradation placed directly on the prepared downstream face of the existing dam. The Chimney Drain should have a uniform width or thickness of 2 feet measured perpendicular/normal to the slope face extending from the Blanket Drain component up to at least elevation 839 feet across the full length of dam. The Chimney Drain component will be placed on the prepared and approved 1.5(H):1(V) slope. We anticipate the contractor would only want to place the Chimney Drain sand in manageable lifts of about 4 to 5 feet so that fill placement can follow soon thereafter, to limit exposure to the weather, and to limit the amount of sloughing/over-build between lifts. Moistening of the sand should help reduce the amount of sloughing in the sloped sand column.

We recommend a Blanket Drain placed on the prepared subgrade (undercut and backfilled to the temporary fill pad or cleaned residual materials) to connect the Chimney Drain and Foundation/Toe Drain to the Rock Toe Berm/Toe Ditch. This section of horizontal drain should have a minimum thickness/depth of 2.5 to 3 feet depending on the aggregate layering chosen. Specifically, the Blanket Drain should consist of a minimum 6-inch layer of C33 sand overlain by a minimum 6-inch layer of washed #89 stone followed by a minimum of 12 inches of washed #57 stone. The first two layers are then repeated in reverse order to satisfy proper aggregate filter transition. The two separate 6-inch layers of C33 sand and #89 stone (6 inches each) above and below the #57 stone can be replaced two single 9-inch layers of fine filter "blend" as previously discussed.

To prevent the composite drain system from draining to the lowest point in the prepared subgrade before exiting the embankments and potentially overloading the drain capacity, we recommend an earthen separation be included in the Toe/Foundation Drain trench every 100 to 200 feet apart that is intended to force the seepage collected in system up gradient of these points to pool and flow downstream through all aggregate Strip Drains to the Rock Toe Ditch. The earth separations should be a minimum of 5 feet to a maximum of 10 feet in width. At each separation, we recommend 3 feet wide excavated Strip Drain trench extend downstream and terminate at the Rock Toe Ditch. The Strip Drain trench will vary in depth from about 3.5 feet at the Foundation/Toe Drain to essentially nothing at the Toe Ditch. Materials excavated from the trench can be placed and compacted to the lower side of the trench on the prepared fill subgrade to create a short berm. The Strip Drain shall consist of #57 stone placed in a filter fabric lined trench. The fabric should have a closed bottom to separate the #57 stone from the soil and an open top with 2-foot flaps to each side of the Strip Drain trench to allow the Blanket Drain aggregates to be properly transitioned.

Where the Blanket Drain outlets the embankment slope, we recommend a Rock Toe Berm be constructed in conjunction with a Rock Toe Ditch. Both systems consist of properly layered aggregates as described previously and will include a triangular zone of Type III riprap to provide connection between the Blanket Drain and the Toe Ditch.

We recommend the finished Toe Ditch be trapezoidal shaped and have a minimum 2 feet wide flat bottom, with 2(H):1(V) or flatter side slopes and be at least 2 feet deep, unless runoff requires a larger ditch for capacity. The Riprap rock should be designed (sized) to resist the anticipated flows. Our experience suggests that Type III Riprap rock will suffice for this application, but we defer to WA&A in this matter. We recommend the overall system include a minimum of 6 inches of C-33 sand placed on the approved subgrade, overlain by a minimum of 6 inches of #89 stone, overlain by a minimum of 6 inches of #57 stone and followed by the designed Type III riprap rock section. The aggregate blend may be substituted. If Type I Riprap rock is required, we recommend a minimum of 9 inches of #34 stone be placed between the #57 stone layer and the Type I Riprap rock. We recommend the Toe Ditch component extend a minimum of 25 feet beyond the termination of the Blanket Drain/Rock Toe Berm as defined by the new lowered normal pool level of 838.5 feet.

The contractor should exercise all care possible during the construction of the seepage collection system to prevent contamination of the various rock materials and damage to the installed drain system. The working areas will be surrounding by sloped earth surfaces and runoff with sediments can easily erode the fine filter layers and contaminate the underlying coarse aggregates. The use of sacrificial filter fabric layers in combination with silt fence and other means of re-directing runoff may be necessary depending on the weather conditions at the time of construction. The protection of the drain system during construction should solely be the responsibility of the contractor. When the initial lifts of soil fill materials are placed to provide cover to the drain system, a minimum fill thickness of at least 3 feet should be achieved so that sufficient thickness exist to allow the contractor to perform moisture conditioning of the fill pad without risking potential damage to the underlying drain system. The contractor will need to provide protective measures to prevent loose soil from falling on and into the Rock Toe Berm/Toe Ditch during construction and subsequent erosion sediment until a suitable grass cover is established.

LATERAL EARTH PRESSURES

No offsite borrow source has been located, nor a laboratory study performed. As such, we are providing the following soil parameters related to lateral earth pressures based on our experience in the Piedmont Geology for use in designing any earth retaining structures associated with this project. For a horizontal backfill configuration, drained conditions, and no surcharge loading, an at-rest pressure of 60 pounds per cubic feet 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 movement is allowable, our previous experience would suggest that an equivalent active 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 structures, unless the structure is designed for the increased lateral stress due to this equipment. All fill materials placed adjacent to walls or structures below grade 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

We recommend that all ground supported structures be designed using a maximum soil bearing pressure of 3,000 psf. The recommended bearing pressure is based on the new structural fill being properly compacted to the recommendations stated in this report. Remedial subgrade preparation is anticipated for support of storm drain, siphon and impact structures located beneath and north of Neely Road in areas not specifically undercut and prepared as part of the recommended embankment modifications. Partial undercutting and replacement with stone may be required. Dewatering operations will also be required during construction of these low-lying structures.

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. Vegetative cover is a critical item and should be properly considered. Remedial maintenance and repair of eroded slopes should be prompt as soon as deficient areas are identified. 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. The crest of the dam should be sloped slightly toward the reservoir. Consideration might also be given to using sod/turf grass in lieu of seed and irrigating the area to at least initially establish a good stand of grass. Grass species should be in accordance with approved Georgia Safe Dams Program guidelines. The dam orientation should be considered. Soil chemistry testing is recommended to determine the best grass species for the soil conditions and which amendments might be needed to create a healthy grass cover.

Riprap protection should also be considered on the upstream slope face (wave protection), downstream of the spillway outlets, and possibly along sloping surfaces adjoining concrete structures. We recommended that any Riprap rock used be bedded on smaller stone underlain by a medium weight geotextile filter fabric. The fabric used should have the same properties as the fabric discussed in conjunction with the internal drainage system. For GDOT Type III Riprap rock, the bedding stone would typically consist of a minimum of about 6 inches of crushed stone such as #57 gradation. For GDOT Type I Riprap rock, we recommend a minimum of 9 inches of #34 stone. 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.

We expect the new/replacement pavement section will be designed to match the existing conditions identified and meet minimum typical Fayette County DOT standards. All pavement subgrades should be proofrolled by the geotechnical engineer with a loaded (20-tons) tandem-axle dump truck prior to placement of the Graded Aggregate Base layer. Proofrolling the subgrade will identify any unstable or soft conditions which could lead to premature asphalt pavement failure.

ADDITIONAL SERVICES

The design of an earth dam continues through the construction phase and initial operation of the structure. As such, we recommend that we be allowed to remain involved in this project through the remaining design and construction phases. We are available to continue to assist you in preparing the details of the plans and specifications. In addition, we have recommended throughout this report, and as required by the Georgia Safe Dams Program Engineering Guidelines, a comprehensive field-testing program during construction that will be necessary to assure that the contractor complies with the specifications and that the dam is built in accordance with our recommendations. We would be pleased to discuss these supplementary services with you at the appropriate time. We currently envision that our professional engineering services will be required during foundation preparation of the embankment and spillway systems, including providing dewatering and remedial excavation/undercutting recommendations; initial internal drainage system construction, and periodically during general embankment construction. All earthwork operations should be monitored on a full-time basis by technicians of our firm. We consider these to be geotechnical-related items.

QUALIFICATIONS

Our evaluation of the dam design and construction has been based on our understanding of the site and project information, and the data gathered during this field exploration program. The general subsurface conditions used have been based on interpolation of the subsurface data between the borings. Regardless of the thoroughness of a subsurface exploration, there is always the possibility that conditions between borings will be different from those at the actual boring, that conditions are not as anticipated by the designers, or that the construction process has altered the subgrade conditions. Therefore, experienced geotechnical engineers should observe all phases of the construction to verify that the conditions anticipated in design actually exist. Otherwise, we assume no responsibility for construction compliance with the design concept, specifications or recommendations.

The nature and extent of variations between the borings may not become evident until the course of construction. If variations are then observed, it will be necessary to re-evaluate the recommendations in this report after performing on-site observations during construction and noting the characteristics of any such variation. However, only relatively minor variations that can be readily evaluated and adjusted for during construction are expected.

The design recommendations presented in this report have been developed based on the previously described project information and subsurface conditions. If there is any change in these project criteria, including project location on the site, a review should be made by this office to determine if any modifications to the recommendations will be required. The findings of such a review should be presented in a supplemental report.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices normal to the Piedmont Physiographic/Geologic Province of Georgia. This warranty is in lieu of all other warranties either expressed or implied. This company is not responsible for the conclusions, opinions or recommendations made by others based on these data.

This report was made to determine the geotechnical properties of the site and is not intended to serve as a wetlands survey. No effort was made to define, delineate or designate any areas as wetlands. Any references to low areas, floodplain areas, poorly drained areas, etc. are related to geotechnical engineering applications. Any recommendations regarding drainage are made on the basis that the work can be permitted and performed in accordance with the current laws pertaining to wetland areas.

The scope of services does not include any environmental assessment or evaluation for the presence or absence of hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the site studied. Any statements in this report or on the Soil Boring Records regarding odors, staining or soils, or other unusual conditions observed are strictly for the information of our client. Unless complete environmental information regarding the site is readily available, an environmental assessment is recommended prior to development of this site.

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 to provide the recommended construction phase monitoring services. Should you have any questions concerning this report, or if we can be of additional service to you in any way, please do not hesitate to contact us.

Sincerely,
Piedmont Geotechnical Consultants, LLC

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Attachments:

- Figure 1: Site and Boring Location Plan West
- Figure 2: Site and Boring Location Plan East
- Soil Test Boring Procedures
- Correlation with Standard Penetration Test Results
- Soil Classification Chart
- Soil Test Boring Records (9)
- Summary of Hand Auger Borings
- Figure 3: Preliminary Grading Plan and Profile Locations
- Figure 4 through Figure 8: Subsurface Profiles
- Summary of Anticipated Undercutting Depths
- Summary of Laboratory Test Results
- Report of Natural Moisture Content
- Particle Size Distribution Report
- Compaction Test Report
- Selected Project Photos of Current Conditions