Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Margaret Phillips Lake Dam Improvements
Fayetteville, Fayette County, Georgia
PGC Project No. 118337

Prepared For:

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December 3, 2019

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

Subject: **Report of Subsurface Exploration and Geotechnical Engineering Evaluation**
Margaret Phillips Lake Dam Improvements
Longview Road
Fayetteville, Fayette County, Georgia
PGC Project No. 118337

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. Our understanding of this project is based on the information provided by Walden, Ashworth and Associates, Inc. (WA&A). The field study and this report were accomplished in general accordance with PGC Proposal No. P18513, dated October 8, 2018. The purpose of this geotechnical evaluation was to obtain sufficient subsurface data within accessible limits of the dam in order for our engineers to formulate geotechnical recommendations for design and construction of the planned dam improvements needed to address the 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 our geotechnical engineering conclusions and recommendations. Piedmont Geotechnical Consultants, LLC engineers have provided verbal recommendations and assistance to WA&A during the development of plans/details prior to the issuance of this written report.

PROJECT INFORMATION

Margaret Phillips Lake Dam is an existing earthen dam, approximately 600 to 700 feet in length, 18 to 20 feet in height, with a 20 to 25 feet wide crest. Longview Road, a two-lane asphalt paved road, is located along the crest of the dam and generally runs near the middle of the embankment. The dam is currently classified as a Category I structure by the Georgia Safe Dams Program. The dam is estimated to be at least 55 to 60 years old. The embankment dam has several noted deficiencies including inadequate/failed spillway system, steep downstream slope, unsuitable vegetation and uncontrolled seepage conditions that require improvements to satisfy current GSDP rules and guidelines. No documentation about the history or the design/construction of this dam was available. The existing low-level pipe and control structure are considered inoperable. As such, the normal pool has been elevated +/- 1 to 2 feet and now flows through the two emergency spillway pipes located at the left end of the dam. There is no known internal drain system. A photo summary depicting current conditions is attached with this report.

We understand that the proposed dam improvements will include a new 30" diameter Low-Level Drainpipe with intake and outlet structures, a new 20 feet wide, concrete, single-cycle Labyrinth Service Spillway, 3(H):1(V) upstream and downstream slopes, 34 feet wide crest with 19 feet wide, 2-lane asphalt road, riprap wave protection strip along the upstream slope and a new bridge over the spillway channel. Relocation of utilities from the dam is planned.

EVALUATION PROCEDURES

To evaluate the embankment dam's internal composition and the underlying foundation conditions, eight (8) soil test borings, designated B-1 through B-7 and B-2A, were drilled to depths of 9 feet to 35 feet below the existing crest of the dam. To evaluate the subsurface conditions along the toe of the downstream slope, eight (8) hand auger borings, designated HA-1 through HA-6, HA-1A, and HA-6A, were performed to depths of 1.6 to 11 feet below the existing grades. Due to shallow obstructions, multiple hand auger borings were sometimes attempted within a few horizontal feet of each other. The boring locations, as shown on Figure 1: Site and Boring Location Plan in the Appendix, should be considered approximate.

Soil Test Borings

Prior to drilling, Fayette County Transportation placed barricades and signage to close off all traffic across the dam during the field exploration operations. Drilling, sampling, and Standard Penetration Testing were performed in general accordance with ASTM D-1586. All eight borings were advanced using mud-rotary drilling techniques, which involved pumping a thickened 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 automatic mechanical 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 value expressed as the number of blows versus the depth of penetration; e.g., 100/3 inches, 50/1 inch, etc. All drilling operations were monitored in the field by the project engineer. All mechanical borings were grout filled upon completion of drilling and the roadway surface was repaired.

Soil samples recovered during the drilling process were classified in the field 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 are shown on the Soil Boring Records in the Appendix.

Hand Auger Borings

The hand auger borings were advanced by manually twisting a sharpened steel auger bucket into the ground. The soils encountered and returned to the surface during the augering process were classified by the engineers in general accordance with the Unified Soil Classification System (USCS). Due to the low consistency granular soils and high groundwater conditions encountered, problems with these soils collapsing/flowing into the open borehole were experienced. Where practical, a 4-inch diameter PVC pipe was manually driven to help keep the hole open so its depth could be advanced. We refer the reader to the Summary of Hand Auger Borings for more detailed information about the conditions encountered at each boring location.

Site Reconnaissance

PGC engineers began observing the conditions of this dam during 2012. Prior to and during our 2019 field exploration, detailed site reconnaissance's were performed by engineers from our office to observe the physical condition of the existing dam and surrounding areas potentially impacted by the planned dam improvements. These observations and the information obtained were used in planning and revising the field exploration, identifying areas of special interest and relating site conditions to known and discovered geologic and subsurface conditions.

SITE OBSERVATIONS

During the course of our field studies, spanning from late 2012 to April - May of 2019, PGC engineers Ali B. Rana, E.I.T. and Craig Robinson, P.E. visited the site and performed detailed observation of the dam's external condition. While on-site during these various times, the following observations were noted. Physical directions are referenced while facing downstream (lake to your back).

1. The right +/- half of the embankment is curved while the left +/- half of the embankment is straight. The total dam length along its crest is approximately 600 to 700 feet in length. Longview Road runs the dam crest. The asphalted road width is about 20 feet. The dam crest and shoulder width vary along the dam length. No to minimal shoulder widths exists along the downstream side about mid-dam, while a several feet wide shoulder exists along the upstream side. No significant dips or ruts were observed in the current asphalt surface. We did observe several areas of elongated cracking in the asphalt surface, especially along the downstream (northbound) lane. We are unsure if the cracking is caused by slope movement/creep, poor subgrades beneath the road surface or possible poor quality backfill over buried utilities.

2. The upstream slope is mostly vegetated with low growing grass and small to medium trees and bushes growing along the shoreline. The upstream slope is gently sloping to the shoreline, becoming steeper towards the ends of the dam. The grass covered areas appear to be maintained. There appears to be some wave benching at and just below the current water level which could be remnant damage from when the normal lake level was 1 to 2 feet lower and controlled by the overflow standpipe. UPC markings indicate that both water and gas lines are buried along the upstream crest/slope.

3. The downstream slope is overgrown with medium to large trees, briars and underbrush. The slope face is somewhat irregular and relatively steep. The general slope configuration for much of the dam length is approximately 1(H):1(V). No obvious evidence of significant seepage through the embankment was observed on the downstream slope, but the trees and vegetation could be masking minor flow. Some seepage was observed in 2012 near the Principal Spillway Pipe alignment. There is currently a considerable flow of water along the downstream toe from the spillway. Some of this water is likely seepage related.

4. The original Principal Spillway System is reported to be a 30-inch corrugated metal riser connected to a 12-inch corrugated metal low level pipe through the dam. In December 2012 the lake level was down +/- 2 feet and the riser pipe was visible. During the 2019 study, the top of the riser pipe was still visible, but submerged by about 6 to 12 inches of water from the elevated lake. During the course of this study, no obvious flow was observed at the downstream end of this spillway pipe, which was also submerged. We are unsure of the conditions that caused the 2012 leaking pipe to become clogged/blocked, resulting in the currently elevated lake level.

5. The original Emergency (Auxiliary) Spillway consist of two 24-inch concrete pipes buried shallow near the left end of the dam and under Longview Road. Due to the current condition of the PSP, the lake appears to be draining mostly through these two pipes. There are no trash guards over the pipes. Drainage from these pipes then flows down the left abutment in a meandering, eroded channel to the floodplain and original stream bed. As these flows enter the floodplain, they spread from a single narrow channel to multiple channels. Water flow slows as it ponds/meanders through the floodplain area before getting to the original stream channel and turning downstream. Rock/PWR materials were observed along the channel near the left end of the embankment. No other rock was observed.

6. Utility clearance (UPC) also marked a buried communication cable along the downstream edge of the crest, generally beneath the pavement section. An overhead powerline is present immediately downstream of the dam in a cleared easement. Poles supporting this line are present within the anticipated area for planned embankment renovation and will interfere with construction activities.

7. The floodplain area immediately downstream of the dam was estimated to be about 200 to 250 feet wide, flat and moderate to heavily wooded with trees. Previous development downstream of the dam and placement of fill materials to raise general grades by several feet higher than the general stream/floodplain levels on both sides of the stream channel appear to have resulted in a narrowing of the floodplain width.

8. A single-family residential structure is located at the right downstream end of the dam.

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.

SUBSURFACE CONDITIONS

The conditions described in the following paragraphs, and those shown in the Appendix, have been based on interpolation of the results of the previously described 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

Eight (8) soil test borings (designated B-1 through B-7, and B-2A) were drilled from the crest of the dam through the asphalt pavement. Borings B-2, B-2A and B-4 were generally drilled along the upstream edge of asphalt. Borings B-3 and B-5 were generally drilled along the downstream edge of asphalt. Borings B-1, B-6 and B-7 were drilled along the approximate embankment/road centerline. The boring locations were adjusted to avoid the existing utilities and drainpipe. Underlying the asphalt pavements, the borings encountered man-made fill materials, Alluvium, Residuum, partially weathered rock (PWR) and refusal materials (presumed rock).

Asphalt thickness ranged from approximately 5 to 9 inches in the eight boreholes. Boring B-4 encountered about 9 inches while the remaining borings encountered about 5 to 6 inches. Minimal thickness of Graded Aggregate Base (GAB) was encountered below the asphalt. Underlying the pavement section, fill materials classified as silty medium to fine sands (SM), fine sandy clayey silts (ML-MH), and sandy silty clays (CL-CH) were encountered. Fill depths ranged from 11 feet to 18 feet in the borings. The fill materials appear to have been placed randomly within the embankment, with no obvious attempt to zone the embankment. Some of the sandier fill materials were encountered near the base/bottom of the fill zone. Fill materials contained varying amounts of rock fragments. Standard penetration test (SPT) values ranged from 1 to 46 blows per foot (bpf), but typically were less than 12 bpf. The higher SPT values exhibited in borings B-5 and B-6 were in soils that contained rock pieces which likely amplified the SPT value. We are of the opinion that when SPT values are less than 8 bpf, the lower SPT values are often indicative of under compacted fill soils. During the drilling of boring B-2, a void/rod drop was encountered from 7 feet to 9 feet and circulation of drilling fluids was lost. Drilling operations were suspended and boring B-2 was abandoned. We are suspect that the void encountered in boring B-2 could be a buried pipe.

Beneath the previously placed fill materials in borings B-2A, B-3, B-4, B-5 and B-6, approximately 2 to 13 feet of Alluvium was encountered. Alluvium is a term used to describe soil materials which have been eroded and deposited by water. The Alluvium was described as silty sands (SM), clayey silty sands (SM-SC) and coarse to fine sands (SP). SPT values ranged from 3 to 11 bpf, with the higher values often being somewhat sandier. Several alluvial samples were described as containing organics. Circulation of drilling fluids was temporarily lost in boring B-4 between 16 to 18 feet, suggesting an open graded zone such as gravels or large organics, or a possible water filled void. Circulation was recovered once the anomaly was filled or became blocked by cuttings.

Underlying the previously placed fill in borings B-1 and B-7 and the Alluvium in borings B-2A, B-4 and B-6, a relatively thin zone of Residuum, soils weathered from the underlying parent rock, was encountered. Residuum was encountered at depths ranging from 11 to 27 feet below the existing ground surface and typically was only about 1.5 to 5 feet thick before PWR/refusal materials were encountered. Residual soils were classified as sandy silts (ML) or silty sands (SM) and exhibited SPT values ranging from 7 to 58 bpf. The higher SPT values often contained less weathered rocky materials.

Partially weathered rock (PWR) materials were encountered in 7 of 8 borings (excluding boring B-2) at depths ranging from 14.5 to 31 feet below the ground surface. Where fully penetrated the PWR ranged from 2.5 to 5.5 feet thick. PWR materials are described as very hard or very dense residual materials that retain the relic structure of the parent rock but can be further penetrated using standard drilling methods. Where sampled, PWR materials were described as very dense silty sands (SM), sometimes containing mica.

Refusal to the drilling methods was encountered in borings B-1, B-2A, B-3 and B-7 at depths ranging from 17 to 25 feet below the surface. Refusal levels were presumed to represent the top of mass rock, but refusal can occur on large rock boulders, lens, or pinnacles. The nature and integrity of the refusal materials can not be determined unless cored using specialty rock cutting tools. This type of work was beyond our scope of work and deemed not necessary for this project.

Hand Auger Borings

Eight (8) hand auger borings, designated HA-1/1A, HA-2 through HA-5, and HA-6/6-A were manually advanced along the downstream toe of the dam by the engineers twisting a 3-inch diameter extendable bucket auger. Cuttings retrieved during drilling were classified in the field by the engineers. Hand auger borings were advanced to depths ranging from 20 to 132 inches below the ground surface. With the exception of hand auger boring HA-4, all other borings encountered a thin surficial zone of dark brown organic topsoil from the ground surface to a depth of approximately 4 to 6 inches below grade. Underlying the topsoil in borings HA-1/1A, HA-2 and HA-6/6A and from the ground surface in boring HA-4, fill materials described as very soft silty clays (CL-CH) and loose silty sands (SM) were encountered to depths ranging from 12 to 24 inches below grade. With exception of borings HA-6 and HA-6A which encountered refusal materials immediately below the fill at 20 inches, the remaining borings encountered alluvial materials beneath the topsoil and fill materials to depths ranging from 42 to +132 inches below grade. Alluvial soils were described as clayey and silty sands (SC-SM) and slightly silty to clean sands (SP) with small gravel. The depth of Alluvium and sand content was greatest in closer proximity to the apparent original stream channel. Flowing sand/collapsing hole conditions were encountered during drilling of borings HA-3, HA-4 and HA-5. A larger diameter PVC pipe was driven into the ground as the hole was advanced to aid with preventing hole collapse/filling. Hand auger borings HA-1/1A, HA-3 and HA-4 reached refusal materials that likely were either the base layer of alluvial gravel or the top of dense residual materials. Borings HA-2, HA-5 and HA-6/6A encountered residual soil materials at or above their refusal/termination depths. Where encountered, the residual soil materials were described as micaceous silty sands (SM).

Due to the wet rotary drilling methods used, a determination of groundwater/phreatic surface was not practical in the mechanical soil test borings. The groundwater/phreatic surface were expected to exist within the fill embankment between the lake's current normal pool level and downstream toe of slope. Stabilized groundwater was encountered in all of the hand auger borings from -16 inches below (HA-6/6A) to +24 inches above (HA-4 inside PVC pipe) the surrounding ground surface. The groundwater levels that were measured above the surrounding ground surface are an indication of hydraulic pressures within the foundation materials from near the tip elevation of the PVC casing. Water surfaces inside the PVC pipe that were higher than the surrounding ground surface likely increase the potential that seepage is occurring in these areas, but the condition of seepage is not obvious.

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 and their respective Unified Soil Classifications (USCS), SPT values, and other notable observations during the drilling operations and soil stratification. Also, please find Figures: 2, 3 and 5 depicting subsurface profiles which represent a linear array of specific boring data on or near the selected profile line. We note that the interpretation of data between actual boring locations is very subjective and often is the result of 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.

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 Labyrinth weir spillway and Stilling Basin structure (LSSB) and a new Low-Level Drainpipe (LLDP) to replace the existing spillway pipes. The design will also include improvements to the embankment dam and Longview Road, and include a bridge over the spillway structure. There remain some logistical and sequencing issues to be resolved as the design is finalized, primarily related to stream diversion, dewatering, undercutting and replacement, and temporary/permanent utility placement. 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 made. These conclusions and recommendations are provided for the sole use of Walden, Ashworth and Associates, Inc. and their client for the improvement of the Margaret Phillips 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 impacting stability and that prevents adequate 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 pipes, even though direct observation of these conditions has not been documented. Large trees exist on the downstream slope. Cracking in the asphalt pavements suggest there could be some minor sloughing/creeping/movement of the downstream slope, especially in proximity to the old Principal Spillway Pipe and adjoining floodplain areas. It is our opinion the trees should be removed, the slopes flattened, and a suitable grass established and maintained. A drain system should be incorporated into the embankment modifications for seepage collection.

The existing Principal Spillway Pipe has been damaged and/or plugged and to our knowledge does not currently operate. As such, the current pool level is elevated by 1 to 2 feet and flowing through the two Emergency Spillway Pipes and down the left abutment along the downstream embankment toe. The spillway system needs to be upgraded and the existing pipes removed or abandoned.

The upstream slope is relatively flat, and we have not observed any conditions to indicate that it is performing poorly. Trees have been allowed to grow along the shoreline. There is also some wave benching that should be addressed. A pressurized water main and gas line are buried in the dam along the upstream crest/shoulder. Pressurized conduits should be avoided in dams as a general rule.

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 significantly, both horizontally and vertically, within the embankment section. Standard Penetration Testing data and samples obtained suggest zones that appear to be poorly compacted and contain varying amounts of rock and sandy soils. The upper few feet of fill supporting the roadway appears to be somewhat better compacted than the deeper fills. The fill consistency in some borings decreases with increasing depth, while the fill consistency in other borings increases with depth. The lowest consistency fill materials were encountered in borings B-2A, B-3 and B-4 at depths ranging from approximately 8 feet to 13 feet below the ground surface, or approximately 5 feet or less above the transition from the embankment fill to the underlying Alluvium. These conditions suggest 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. The alluvial soils contained varying amounts of sand and where encountered were deepest in borings B-4 and B-5. These materials often included organics. No evidence of a man-made keyway, typically constructed about middle of the embankment to help control seepage, was encountered during drilling. The variability in the depth of the alluvial layer (thickness) suggests an irregular residual (natural ground) profile along the dam alignment. There appears to be a significant, and possibly erratic, change in the natural ground surface occurring in the area of borings B-2/2A, B-3, B-4, B-5 and B-6 where the top of residual materials (soil/PWR) immediately below the Alluvium varies from approximate elevation 817 feet (B-3) to 803 feet (B-5) to 811 feet (B-6). This variation in the top of the Residuum may represent the characteristics of the original stream valley prior to deposition of the alluvial materials. This variability may also indicate that more than one stream channel existed within the floodplain/dam footprint. Limited thickness of residual soils was encountered in the borings overlying the PWR materials. Partially weathered rock was encountered directly beneath the Alluvium in borings B-3 and B-5. The limited depth of the more weathered residual overburden materials may create subgrade preparation and time of construction dewatering challenges.

The manual hand auger borings performed along the downstream toe of the embankment, primarily within the anticipated floodplain limits, encountered variable depth, material type and consistency of alluvial materials. In general, borings HA-3, HA-4 and HA-5 depict the deepest alluvial deposits of approximately 8 to +11 feet below existing ground surface. The upper materials typically consisted of very low consistency silts/clays transitioning to somewhat sandier soils and clean sands with gravel at depth. The refusal materials encountered in borings HA-3, HA-4 and HA-5 are expected to be on large gravel which likely exists at or near the base of the alluvial deposits. Based on our experience, we typically expect these coarser sand/gravel deposits to be 1 to 3 feet thick for this size stream. While stable in their present condition, these coarse sand/gravel deposits often allow considerable seepage, becoming unstable when exposed to uncontrolled groundwater flow. The type conditions can exacerbate construction dewatering challenges if not properly addressed. With the lake at its current level, we measured stabilized groundwater levels at or above the current floodplain level in the hand auger borings further indicating the dam is experiencing uncontrolled seepage conditions.

The lake remained full during this geotechnical study. As such, no geotechnical evaluation of the upstream slope and lakebed were performed. We understand plans are to perform more limited improvements along the upstream slope to create a uniform embankment configuration and geometry that is more suited for maintenance and wave protection. With lack of specific geotechnical data, it should be assumed that subsurface conditions are similar to that encountered in the soil test and hand auger borings plus some added depth of more recent alluvial deposition that has occurred since dam construction.

Even with the deficiencies and subsurface conditions noted above, and with exception of the areas directly impacted by and surrounding the planned spillway modifications, 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 variable 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 new spillway structures. Furthermore, fairly significant modifications to the existing embankment geometry/configuration are recommended to create a flatter uniform slope for enhanced slope stability, ease of maintenance and to provide adequate cover over the recommended new internal seepage collection system. We recommend all alluvial materials underlying the recommended slope modifications (both upstream and downstream) be undercut and replaced with new structural fill materials

Positive dewatering and stream diversion operations will be required during construction so that the below grade activities can be accomplished in the dry. Draining of the lake as early in the construction sequence and 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 amount of water flowing along the downstream side of the embankment left of the stream channel and the standing water in the original plunge pool and stream channel. We expect the large trees and other vegetation on the embankment could be masking the seepage too. To control the critical seepage, a seepage collection system consisting of an all aggregate toe/foundation drain and a full height (to normal pool) Chimney Drain is recommended. A portion of the downstream slope will need to be removed to create a uniform slope surface on which to build the Chimney Drain. The underlying existing alluvial soils will need to be undercut and replaced beneath the removed portion of slope and the new flattened slope projection. The new flattened slope will encapsulate and protect the new seepage collection system and create a gentler slope configuration that can be maintained and monitored.

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 all inclusive lists of the required tasks. These are not presented in any order of importance or sequencing, and while listed separately will often be overlapping 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 and preservation of fish can be considered. The presence of any standing pool could have negative impact on the dewatering and construction efforts.
- Breaching the dam to construct the new LLDP/temporary diversion pipe and intake/outlet control structures.
- Construction of a temporary coffer dam (may be phased) to coincide with upstream embankment improvements and protect the breach/undercut areas.
- Installation of temporary dewatering systems, both upstream and downstream of the existing embankment, and possibly within the limits of the breach.
- Removal of all unsuitable vegetation on the embankment and impacted areas.
- Temporary/permanent routing of buried and overhead utilities. These activities and the temporary/permanent locations selected should consider the limits of undercutting, stream diversion and the associated time of construction dewatering operations footprint.

- Slope shaping, undercutting and replacement of unsuitable soils beneath the footprint of the new LSSB, new LLDP and both slope extensions.
- Removal of existing spillway pipes, subgrade preparation and backfilling of the excavations.
- Installation of seepage collection drains in coordination with earthwork. Foundation drain will need to be installed in vertical segments due to depth of undercutting.
- Fill placement to create flattened slopes in non-breach areas and achieve general fill to elevation 928 feet within the breach area.
- LSSB and bridge construction.
- 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, removal of existing spillway pipes, initial fill placement, toe and foundation drain construction and construction of the LLDP and LSSB and associated channel work. 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. 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 difficulty in draining the lake and of handling diversion will be directly impacted by the weather conditions prevalent during construction, the lack of a functioning low-level drainpipe, and the contractor's selected methods and sequencing of operations. It is anticipated that the lake will need to be essentially drained to help minimize the dewatering requirements, and also to provide for reasonable storage from rainfall events during the period of time that the excavation through the dam is created for constructing the LLDP, LSSB and the upstream slope modifications.

It is our understanding the existing low-level pipe is non-functional and will be removed. As such, the initial draining of this lake will likely require the use of siphons and/or pumps until the standing pool can be reduced and managed. At that point, we envision the existing dam embankment will be breached so that temporary diversion of normal and storm event rains can be accomplished via the pipe and/or channel. The anticipated breach of the existing embankment should as much as possible coincide with other planned excavations and undercutting for the new LLDP and LSSB. We anticipate diversion of the stream through the new LLDP could be accomplished early in the construction sequence and then used for stream diversion while other activities are performed. Undercutting, earthwork and drain construction in the proximity of the new LLDP will need to be carried a sufficient distance beyond the pipe alignment so that subsequent construction of the LSSB and seepage collection system can occur unimpeded. The contractor should be advised of the hydrologic characteristics of the watershed basin and the hydraulic capacity of the new LLDP so that they may be able to determine what storm event/water level to provide protection of their subsequent work, and the height and placement of temporary coffer dams and other associated erosion/sediment control features. 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. The diversion plan should also consider the relocation of utilities, which we understand will need to remain operational during construction.

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 downstream of the embankment, and no explorations performed upstream. Based on the hand auger borings, the ambient groundwater conditions downstream of the dam are essentially the same as the current ground surface to about -2 ft below the ground surface between elevations 816 to 825 feet with the lake at its normal pool of +/- 827 feet. As such, we anticipate extensive dewatering efforts will be required to lower the current groundwater levels by as much as 17 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 level. Draining of the lake well in advance of beginning upstream and below grade construction should help with the dewatering efforts.

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, construction of the lower foundation drain segments, the new LLDP and LSSB construction. Given the magnitude of dewater 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 is 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 needed 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 and businesses, 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.

RELOCATION OF EXISTING UTILITIES

Based on the information provided and our observations during the field exploration, we understand there to be an existing active water and gas line buried along and within the upstream crest/slope and an existing active communication cable buried along and within the downstream crest/slope. There is also an overhead power/communication easement with poles and lines immediately downstream of existing dam. We also suspect that other non-active buried utilities could exist in the dam. Boring B-2 encountered a suspected pipe/void at a depth of 7 feet below grade during drilling operations. All these existing utilities will be impacted by the planned dam construction. It is our understanding these active utilities will be temporarily/permanently relocated to avoid the planned construction. When possible, these utilities should be moved entirely out of the fill embankment and beyond the limits of planned embankment construction, including boundary areas where dewatering systems might be located. Otherwise, special construction techniques to take into account for dam and spillway construction around these functioning utilities should be anticipated and engineered.

Placement of pressurized conduits interior of the embankment should consider impacts of potential failure, seepage along transverse conduits, potential impacts/penetrations through seepage drains, concrete walls, etc. As a minimum, pressurized waterlines within the embankment should utilize restrained joint pipes, encasements, drainage sleeves, etc. and should include additional valve cutoffs at both ends of the dam that could be closed in the event of a waterline failure to isolate the dam area from further damage. Where possible, relocated utilities should be installed above the lake’s normal operating pool, and if possible, above the maximum flood pool, to prevent/minimize seepage along the utility, especially where utilities are installed upstream/downstream (transverse) direction. We will be pleased to assist with further detailing of the utilities as this design is finalized and plans prepared.

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 unmaintained. While no obvious indications of significant slope movement such as sloughs or “swayed” trees (bent/curved trunks) have been observed, we have observed cracked and mildly displaced asphalt pavement suggesting potential minor movement/slope creep along the downstream slope. There is also no evidence of current free-flowing seepage through the embankment above the downstream toe elevations, but the existing vegetation could be masking an embankment seepage problem. There is considerable water flowing along most of the downstream toe of slope and the hand auger borings encountered groundwater essentially at or above the ground surface with the lake at its current level.

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 center line of the existing Longview Road will remain essentially the same, but the required minimum road width, including a uniform shoulder, will be increased to approximately 34 feet, which will require some widening of the dam in both directions, but mostly to the downstream direction.

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 takes into account the planned crest widening and anticipated slope flattening. Several transverse profiles were developed at our request which present both existing and proposed 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 uniformly initiated from the dam centerline. As such, we recommend the downstream slope of the existing embankment be uniformly shaped/excavated to a 1.5(H):1(V) configuration with the upper or top limit of the excavation placed a maximum of 5 feet downstream of the proposed embankment/road centerline. 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 are provided for drain protection and working space.

Shaping/excavation along a portion of the upstream slope will also be required in areas where more than 1 to 2 feet of fill placement is required to create a uniform and well compacted 3(H):1(V) slope configuration. The existing upstream slope is much flatter than the existing downstream slope and no drain must be covered, so the recommended excavations into the upstream slope are considered less drastic. In some areas of the dam, especially towards the left end, no remedial shaping/excavation prior to fill placement will be required. To accomplish this task, we recommend the upstream slope between about +/- 350 feet right and +/- 100 feet left of the proposed LSSB be cut on a 3(H):1(V) downslope beginning +/- 7 feet upstream of the planned embankment/roadway centerline. Minor shaping, excavation and cosmetic grading will be required along the upstream slope in areas further left to the abutment.

In all areas where remedial slope shaping/excavation operations are recommended and 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 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, undercut depths ranging from essentially nothing beyond hand auger borings HA-1 and HA-6 to possibly in excess of 13 feet (HA-4) should be anticipated. 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 require excavations to be extended deeper than anticipated. 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. Given the access limitations and the potential dewatering challenges, we suggest 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. The depth of remedial undercutting along the upstream slope has not been determined by direct exploration, but rather by an approximation of the nearest available subsurface data. We do not anticipate that the undercutting depths along the upstream slope will extend to a lower elevation than the undercutting along the downstream slope; but the overall depth could be greater due to deposition of more recent sediments. We refer the reader to the Summary of Anticipated Undercutting Depths/Elevations included in the report Appendix.

Beneath and extending laterally to the prescribed limits of the new LLDP and LSSB, we recommend all the existing fills and alluvial soils be undercut to expose the underlying firm undisturbed residual subgrade. Excavations for the new LLDP may need to occur early in the construction sequence so the pipe can be used for stream diversion. We recommend the limits of the undercut excavation for the new LLDP and LSSB extend a minimum of 10 feet beyond the greater horizontal extent of each structure (including walls) at the approved residual subgrade level. Undercutting should also include the existing fills and Alluvium located between the new LLDP and LSSB. From the approved residual subgrade level, the resulting excavation should be sloped up on a 1.5(H):1(V) or flatter slope to the existing ground surface. Based on the results of the soil test borings (B-2A, B-3, B-4 and B-5) drilled along the existing dam crest and from hand auger borings (HA-4 and HA-5), undercut depths are expected to extend to at least 16 to 29 feet below the dam crest, or to about elevation 803 to 816 feet. The soil test boring data suggest a considerable change in the top of the residual materials between borings B-2A and B-6. As such, we recommend the spillway structures be kept as close to borings B-2A/B-3 as possible in an effort to support the new structures on the better foundation conditions.

The two existing concrete spillway pipes located near the left end of the dam will 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 both pipes plus 5 feet and include a minimum 1.5(H): 1(V) entrance/exit slope. The plunge pool at these pipes appears to be at least 2+ feet deeper than the surround ground surface. Remedial subgrade preparation will include the plunge pool area and drainage ditch. High consistency residual materials were observed in near proximity to these pipes. 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 20 feet wide down to a minimum elevation 824 feet. A deeper and wider excavation may be required based on the actual conditions encountered. Additional subgrade preparation measures, including rock surface cleaning using compressed air or water jet may be required prior to filling, if an irregular PWR/rock materials surface is exposed.

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 large stumps/root system from the downstream slope. If the prescribed excavations fail to remove the large stumps and root systems, and in areas where minimal to no excavations are planned, additional/intentional stump and root removal may be required prior to beginning subsequent 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. The success of the undercutting operations will significantly depend on the effectiveness of the dewatering/diversion operations. No undercutting or subgrade preparation should be performed until the contractor can demonstrate the zone/area to be undercut has been adequately dewatered. 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 grade, and downstream of the chimney/foundation blanket drain section. In the dam breach, created to remove the existing spillway pipe and to undercut for support of the new spillways, "select" soils should be placed from the approved residual subgrades to at least 2 feet above the normal pool level (elevation 829 feet), for the full width of the breach and toe-to-toe beneath the dam footprint. 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. To be efficiently accomplished moisture conditioning/drying, 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 so that moisture conditioning can be accomplished. The upstream left shoreline may be suited for these type operations, if accessible. We note that our assessment of the existing fill materials is based purely on our observations of limited SPT samples recovered during the drilling of borings B-1 through B-7. We expect actual conditions within the embankment to vary. 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 above elevation 830 feet within the road footprint 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 for improved support of the pavement section. Due to the limited space, 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). Temporary fill surfaces oriented upstream to downstream should not be allowed within the limits of the prescribed breach upstream of the chimney drain. 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, 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 outlet 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 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 34 feet to accommodate a 19-feet wide paved section with 7.5 feet wide shoulders. If possible, the crest should be sloped with a minimum 1-2% cross-slope grade towards the lake (down towards the upstream) to minimize surface flows across the longer downstream slope section. We recommend the wave protection rock be installed into an excavated notch after the fill embankment section has been raised to at least 1 to 2 feet higher than the armored section (approximate elevation 830-831 feet). Excavated soil materials from wave protection construction can be used to complete the embankment.

SPILLWAY MODIFICATIONS

We understand plans are to replace the existing spillway pipes with a new concrete single-cycle Labyrinth Weir Spillway with Stilling Basin (LSSB) and a new Low-Level Drainpipe (LLDP). The proposed LSSB will be placed in the general area of the existing non-functioning spillway pipe and the new LLDP will be located further to the right. The existing spillway pipes will be removed as part of the planned embankment modifications and spillway upgrades.

The proposed LSSB chute will be 20 feet wide (inside dimension) and include approximately 90 feet of weir length. The two-stage weir will include a normal pool notch set at elevation 826.9 feet to control flows up to about the ¼ PMP with the majority of the weir set at elevation 829.5 feet. Walden, Ashworth and Associates, Inc. have determined the ½ PMP flood pool level to be elevation 830.44 feet. A 12-inch x 12-inch sluice gate will be included in a weir wall just above the chute floor. The chute floor elevation will be 821.5 feet. The spillway chute will step down to the Stilling Basin elevation of 815.0 feet using one intermediate step set at elevation 818.0 feet. The Stilling Basin will include chute blocks and an end sill wall. Entrance and exit wingwalls are proposed. A Type I riprap rock lined outlet channel downstream of the Stilling Basin and stepped chute section is planned. The new LLDP will penetrate the right downstream wingwall and discharge into the riprap lined channel. The proposed LSSB will also include cutoff walls beneath the slab and seepage walls extending beyond the side walls at the dam centerline. An underdrain beneath the Stilling Basin and stepped chute section is planned. Minimum concrete slab thickness will be 18 inches. A bridge will span the spillway and will be supported on the spillway walls.

The new LLDP will be a 30-inch diameter ductile iron pipe with concrete cradle. The pipe inlet will be at elevation 817.5 feet and the outlet invert elevation will be 816.0 feet. The pipe will include a 45-degree bend to align with the LSSB wingwall. A concrete structure will support the inlet gate control and sluice gate.

To ensure that the new spillways are uniformly supported, we recommend the width of the undercut zone (breach through existing embankment) be equal the width of the spillway at its widest dimensions (at the wingwalls) plus 10 feet further on the left side projected down to the approved residual subgrade level (estimated to be elevation +/- 803 feet). The right edge of the breach excavation should be widened further to the right to include the new LLDP. We recommend the new LLDP alignment be no closer than 10 feet of the LSSB walls to provide minimal working clearance between structures and to lessen any additional loading to the LLDP from the LSSB and bridge. The right breach limits should extend a minimum of 10 feet beyond the new LLDP and cradle at the approved residual subgrade (estimated to be +/- 817 feet) to provide minimal clearance so that adequate earthwork operations can be conducted. The breach excavation slopes should then be projected upward from the approved residual materials through the remaining embankment/alluvial materials to the ground surface at a configuration no steeper than 1.5(H):1(V). An evaluation of settlement was not performed as part of this geotechnical study. As such, to minimize the potential impacts of the total and differential settlement (primarily from the consolidation of the new fill material) on the LSSB, we recommend the general embankment within the full limits of the prescribed breach, extending left to right and toe-to-toe for the new 3(H):1(V) sloped configuration footprint, be filled to a minimum elevation of 928 feet, and then the areas impacted by the new LSSB be re-excavated to subgrade before construction of the concrete structure can begin. The placement of these fills to elevation 928 feet will help preload this area and lessen the impacts of settlement on the spillway and bridge structures. Following successful undercutting and fill placement, the new spillway structures should be fully and uniformly supported by a new structural fill.

In conjunction with the LSSB, several turndowns, two steps and the associated underdrain system will be designed into this structure. As a minimum, turndowns are typically required at the upstream and downstream limits of the spillway base slab that supports the weir walls, and at the downstream end of the Stilling Basin. In addition, underdrain systems (aggregate and perforated pipe) are normally included downstream of the weir walls, which generally corresponds to the downstream end section of the main slab and beneath the stepped sections of slab and the Stilling Basin. For this application, we recommend that a total aggregate system be utilized for the underdrain system. Generally, no filter fabric should be included. The fine filter layers should consist of a minimum 9-inch layer of natural sand meeting ASTM C-33 standard gradation placed on the approved soil subgrade followed by a minimum 9-inch thick layer of washed No. 89 stone. Overlying the #89 stone, a coarse aggregate filter layer comprised of washed No. 57 stone should be placed to support the concrete structures and encapsulate the drainpipes. We recommend a minimum 12-inch thick layer of #57 stone for all areas with underdrain, except that the minimum thickness of #57 stone should be increased to at least 18 inches beneath the Stilling Basin section. Embedded perforated PVC pipe should be a minimum of 6 inches in diameter. Perforated pipe should transition to solid pipe about 2 to 3 feet before exiting the coarse filter materials. In lieu of the three-layer aggregate system recommended, PGC will allow the use of a 12-inch thick 50/50 blend by volume of C33/#89 stone fine filter and the 12 inches of #57 stone. The spillway underdrain system should be constructed independent/separate from the embankment seepage collection system. Typically, a 5 feet H/V separation between drain systems is sufficient.

Plans are to install a new 30-inch diameter ductile iron LLDP with inlet/outlet controls through the embankment. The LLDP will outlet through the right downstream LSSB wingwall. It is not possible to adequately compact soil materials beneath the bottom of a round pipe. Therefore, we recommend a concrete cradle and mudmat be constructed beneath the full length of pipe to help alleviate the problem of soil compaction and potential seepage along this critical area. Typically, this cradle would extend a minimum of 6 inches beneath the pipe and up to the spring line of the pipe. The concrete cradle should be a minimum 6 inches wide at the spring line of the pipe and should be formed and include a minimum 1(H):4(V) batter (minimum of 6 inches wide) from the spring line to the base. The contractor should not be allowed to cast the cradle against the sidewalls of any excavated soil trench configuration. Rather, if a trench is excavated within the backfill materials, the base of the excavation should be at least 2 feet wider than the concrete mudmat, and the excavated side slopes no steeper than 1.5(H):1(V). We recommend that the cradle be jointed to coincide with the pipe joints and include at least ¾-inch thick expansion board between cradle sections at the joint to allow for maximum articulation of the pipe and bedding during settlement as the earthen embankment is constructed.

We anticipate the excavation for the LLDP to be exposed for a considerable amount of time as the pipe is installed and the concrete cradle constructed. Therefore, we recommend that the prepared pipe subgrade include the installation of a thin (minimum 4 inches) concrete “mudmat” seal to protect the approved subgrade. The mudmat thickness does not replace the concrete cradle. As such, the soil subgrade prior to placement of the concrete mudmat should be at least 10 inches below the outside invert of the LLDP. The mudmat should be formed on the approved subgrade.

All fill materials placed immediately adjacent to the LLDP and spillway walls should be “select” clayey fill materials that are placed in thin lifts and uniformly well-compacted utilizing hand operated tamps and self-propelled vibratory pad-foot trench rollers. The recommended internal drainage system should be designed to surround the conduit where the two coincide, as previously described, so that any seepage that may occur along the LLDP is safely controlled and collected. The portion of the internal drain that exists below the proposed LLDP invert will require installation prior to construction of the mudmat, pipe and cradle.

As an added precaution, we recommend that each of the LLDP joints be wrapped in a double layer of the approved polypropylene geotextile filter fabric. The fabric strip should be at least three feet in width and centered about the pipe joint. The intent of the fabric is to help prevent the migration of soil fines into the conduit should a crack or open joint occur. This is a very simple and inexpensive procedure and can help significantly in preventing such problems. The filter fabric should be installed prior to cradle construction and firmly affixed to prevent movement and/or damage during forming and concrete placement.

All exposed subgrade materials should be in a compact condition at the time of underdrain and/or concrete 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 geotechnical engineers, and should be provided at no additional cost to the project. All subgrades should be evaluated by the geotechnical engineer and approved for subsequent construction activities.

INTERNAL EMBANKMENT DRAINS

The following recommendations concerning embankment seepage control are based on 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 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 of wet ground conditions at the site, and the groundwater levels measured in the hand auger borings performed just downstream of the existing dam, we suspect the existing embankment dam is currently experiencing a considerable amount of under-seepage through the more permeable alluvial soils that remain beneath the embankment fill materials. Furthermore, we are suspect the embankment dam is experiencing seepage that is possibly being masked by the trees and vegetation growing on the steep downstream slope. As such, we recommend the entire embankment be modified to include a full height chimney/foundation seepage collection system, as would be typically required on Category I dams in Georgia. These conditions were discussed with the Georgia Safe Dams Program prior to our field studies and were considered mandatory modifications to the project in addition to 3(H):1(V) embankment slopes previously recommended, 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 relatively low height embankment dam.

The recommended seepage collection system for this dam includes a combined toe/foundation/chimney drain placed generally parallel along the downstream toe of the prepared embankment slope at the recommended pad level or on approved residual materials. The proposed drain should extend laterally to the fullest limit possible as defined by the normal pool elevation of 827 feet as projected to the downstream residual abutment grade. 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 the lower portions of this recommended internal seepage collection system. Once the downstream slope has been cut to the 1.5(H):1(V) configuration as recommended, the exposed subgrades should be evaluated by the geotechnical engineer prior to subsequent work by the contractor. Some additional undercutting may be required in the mid-section of the dam where cracking/settlement in the pavement was observed should sloughed materials be encountered, before earth fills and/or the drains can be constructed. For areas where undercutting is required, the 1.5(H):1(V) slope beginning 12 feet upstream from the new downstream edge of crest (or 5 feet downstream of the dam centerline) should be extended to the approved residual soils and should extend downstream the distance of the new 3(H):1(V) slope projection to the approved residual subgrade. We refer the reader to the Summary of Undercutting Depths in the report Appendix for additional information.

Due to the anticipated depths of undercutting at the planned breach area, and as defined by borings HA-3, HA-4 and HA-5, it will likely be necessary to construct the toe/foundation drain segment through the deeper undercut areas (undercut areas below about elevation 813 feet) in multiple lifts to prevent having to create an overly deep excavation for this purpose after backfilling has been completed to the final pad grade. A maximum trench depth of 4 feet is recommended for these interim lifts. In areas where the undercut depths outside the limits of the spillway breach excavation are below about elevation 811 feet, the foundation drain segment detail will need to be field modified to shift the vertical section from about +/- 5 feet to up to +/-15 feet downstream, increasing the horizontal blanket section width, to minimize excessively deep vertical excavations into the remaining existing embankment fills and underlying alluvial soils. This situation will likely occur to the left of the prescribed spillway breach and extend towards and to about boring HA-3. Once the toe/foundation drain is completed, the lower portions of the chimney drain can be constructed on the prepared downstream slope and pad surfaces, and additional fill materials placed to cover the drain system to create the extended downstream toe configuration previously recommended. The spillway underdrain system and the embankment seepage collection system should be separated beneath the spillway structure footprint. However, outside the limits of the Labyrinth Spillway, the Chimney Drain should extend to normal pool elevation and be contiguous to the outside spillway walls above the pad grade. The toe/foundation piece of the drain should stop a minimum of 5 feet from the outside edge of the spillway structure foundation. The spillway underdrain will serve as the foundation drain beneath the structure.

The foundation/toe drain alignment will generally follow the toe of the excavated 1.5(H):1(V) temporary slope at the created pad grade of approximately elevation 819 feet. For ease of construction and to lessen the risk of slope failure/trench collapse along the interface between the existing dam and new fill, we recommend the toe/foundation/chimney design provide for an approximately 5-feet minimum horizontal offset in the trench alignment from the 1.5(H):1(V) temporary toe. As discussed in the previous paragraph, the 5-feet minimum offset from the 1.5(H):1(V) cut slope may require adjustment by widening the offset distance to account for areas of undercutting below about elevation 811 feet. Where the Chimney Drain is placed above the toe drain, a horizontal section of Chimney Drain (blanket section) will be required to connect the chimney to the offset toe drain. Where drain construction can commence without need for interim lifts, the drain will still generally follow the toe of the 1.5(H):1(V) slope with a 5-feet minimum offset. In this situation, the toe/foundation drain should extend a minimum of 2 feet up to a maximum depth of 6 feet into residual soils. For all of the drainage system components described, we anticipate that our staff will continue to work closely with WAA to assist 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.

Careful control of the alignment of the drain system to assure that the individual lifts line up appropriately is necessary. This condition will occur in areas where the drain alignment is shifted in areas where the undercut depths are below elevation 813 feet. Excavation and construction of the toe/foundation segments of drain should be performed in short manageable lengths to allow installation of full pipe segments and aggregate 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 the shift. Positive dewatering during toe/foundation drain construction should be anticipated during excavations below elevation 819-820 feet. A straight edge blade on the backhoe 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-exposed to minimize the need for laborers to enter the excavation for final cleanup. 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. We recommend that the foundation drain portion of this system below about elevation 815 feet consist entirely of natural sand meeting ASTM C-33 standard gradation. Manufactured sand is not allowed.

The filter fabric 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 (+/- elevation 813.5 feet) into the upper portion of foundation drain (C33 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 an actual trench depth of about 5.5 to 6 feet, with the top of the lowest lift of foundation drain prior to constructing the toe drain maintained at least 24 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 should also have an open configuration for the filter fabric to allow direct contact between the recommended Chimney Drain (blanket section) with the top of the toe/foundation drain. To provide the proper filter transition, it will be necessary to provide a layer of No. 89 stone at least 9 inches at the top of the toe drain section before beginning the Chimney Drain blanket section. Another similar layer of No. 89 stone 9 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 the right side and two on the left side of the LSSB should be sufficient for this system. All the non- perforated outlet pipes should provide for at least a 1% minimum slope for proper drainage. We recommend maximum 22.5-degree bends 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 walls at an elevation above the normal design flow depths and at least 12 inches above all horizontal surfaces. 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 minimum 10 feet length of ductile iron pipe that would extend through the headwall/spillway wall and remain exposed. We recommend that headwall construction be such that a minimum 12-inch drop below the pipe invert can be maintained for monitoring purposes. The outlet pipe should extend at least 3 inches beyond any headwall/vertical wall surface, primarily to accommodate the installation of the small animal guards.

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 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 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 imminent 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. We request the Minimum Average Roll Values (MARV) sheets referencing each fabric roll number be provided to the engineer at the time of delivery to the project.

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 or created fill slope adjacent to the new spillway (within the breach limits) 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 horizontal blanket drain segment up to at least elevation 827 feet across the full length of dam. The Chimney Dam should also be placed in intimate contact with the outside of the LSSB walls.

With the toe drain alignment offset from the 1.5(H):1(V) toe of slope, as discussed previously, a short to moderate essentially horizontal section of the Chimney Drain (blanket section) would initially be placed at the pad elevation or approved residual subgrade to connect the top of the toe/foundation drain with the toe of the existing slope. This section of horizontal drain should have a minimum thickness depth of 2 feet. Where the Chimney Drain is constructed, the top of the fabric wrapped toe/foundation drain should be left with an "open top" configuration by placing the extra filter fabric on the subgrade to either side of the trench on the approved subgrade. 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 horizontal Chimney Drain (blanket section) should be backfilled with "select" fill to prevent any seepage from bypassing the 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.

We also recommend the installation of a filtered rock lined toe ditch in the abutment/floodplain areas outside the limits of the plunge pool/outlet channel to help control surface runoff, but also to provide a backup mechanism to safely collect any minor seepage that might bypass the internal seepage collection system. In the abutment areas where no to minimal undercutting is performed, the toe ditch should parallel the downstream toe of the embankment dam at its contact with the finish grades. The ditch alignment may deviate somewhat from this alignment in areas where the deeper undercutting is performed. We recommend the finished ditch be trapezoidal shaped and have a minimum 2 feet wide flat bottom, 2(H):1(V) or flatter side slopes and be at least 2 feet deep, unless runoff requires a large ditch for capacity. The larger rock should be designed 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. 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.

Some additional drain details will need to be worked out, especially where the recommended internal drainage system crosses or is in contact with any existing and new conduits or structures associated with this dam. PGC will be please to assist WA&A with the development of the drain layout and details as the design is finalized.

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 the earth retaining structures, such as the LSSB and associated wingwalls. 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 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 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 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 spillways will be supported entirely by new fill, we recommend that the spillway 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.

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 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 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. 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

H Craig Robinson

Ali B. Rana, E.I.T.
Staff Engineer

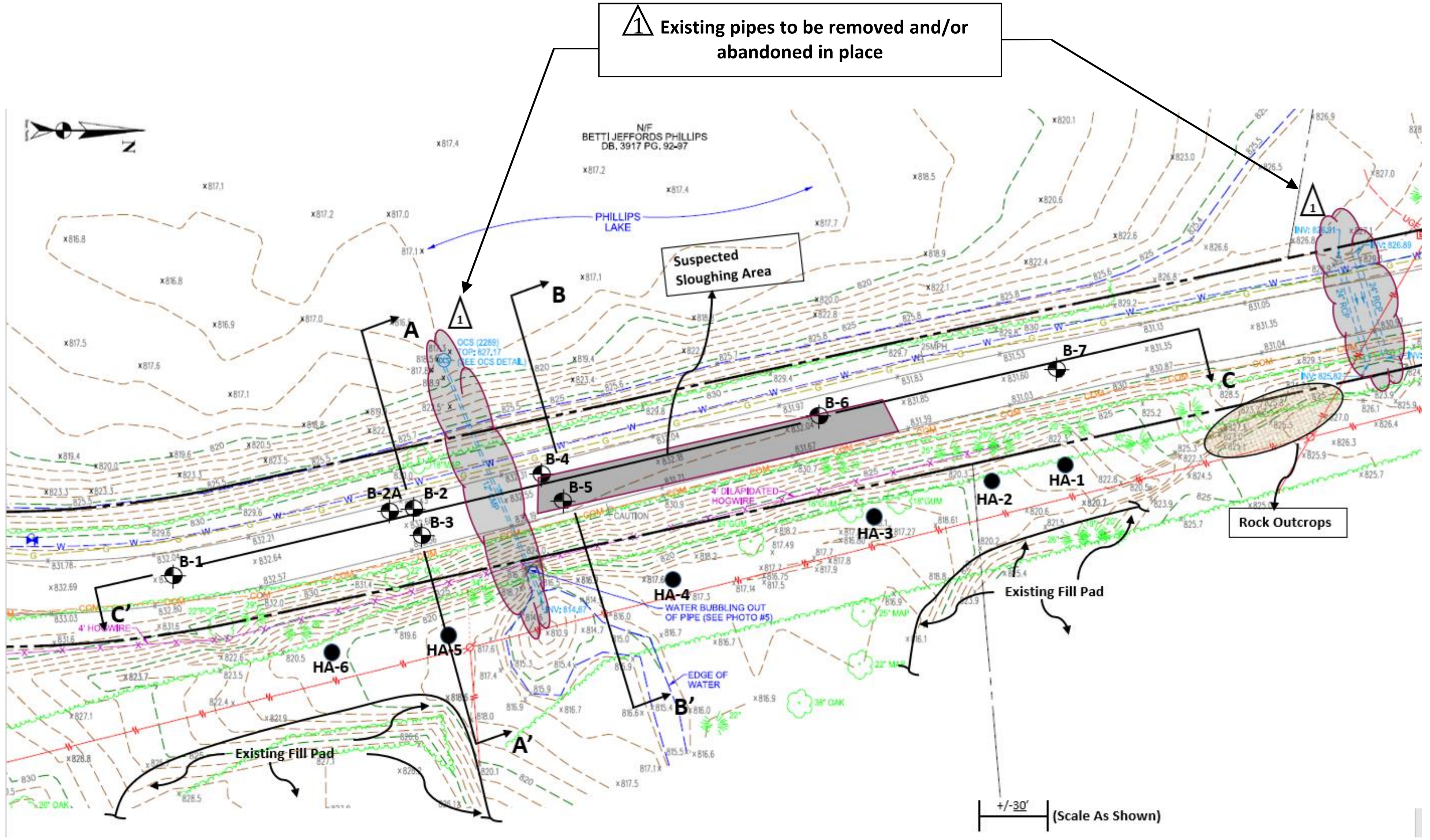
H. Craig Robinson, P.E.
Senior Registered Engineer
Registered Georgia 19121

No. 019121
HORACE CRAIG ROBINSON

AR/HCR/ew

Attachments:
- Figure 1: Site and Boring Location Plan
- Soil Test Boring Procedures
- Correlation with Standard Penetration Test Results
- Soil Classification Chart
- Soil Test Boring Records (8)
- Summary of Hand Auger Borings
- Figure 2 through Figure 5: Subsurface Profiles
- Summary of Anticipated Undercutting Depths
- Selected Project Photos of Current Conditions

ATTACHMENTS



LEGEND

- ⊕ SOIL TEST BORING
- ● HAND AUGER BORING

FIGURE 1: SITE AND TEST LOCATION PLAN

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
— AN ATLAS COMPANY —
3000 NORTHFIELD PLACE, SUITE 1100
ROSWELL, GA 30076

DRAWN BY: AR
APPROVED BY: CR
PROJECT NO.: 118337
DATE: 6/3/2019
SCALE: NTS

NOTES:

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

SOIL CLASSIFICATION CHART

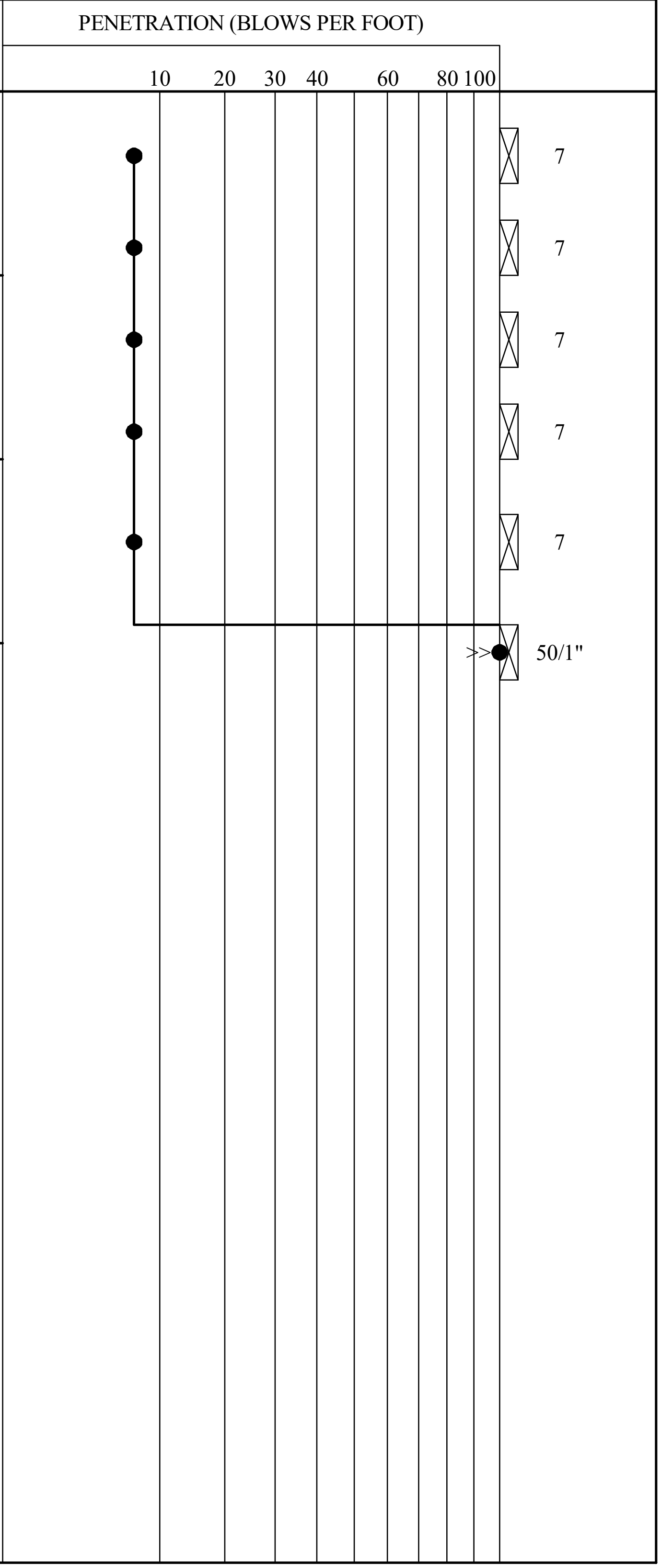
MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	●	**GW**	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			○	**GP**	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	○	**GM**	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			▨	**GC**	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (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 (APPRECIABLE AMOUNT OF FINES)	⁘	**SM**	SILTY SANDS, SAND - SILT MIXTURES
			▨	**SC**	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS 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, LLC
— AN ATLAS COMPANY —

Margaret Phillips Lake Dam

DEPTH (FT)	DESCRIPTION	ELEV. 833	PENETRATION (BLOWS PER FOOT)
	ASPHALT: 6 inches		
	FILL: Firm orange tan brown sandy clayey SILT (ML-MH)		7
			7
5		828	
			7
	Firm gray tan brown fine sandy clayey SILT (ML-MH)		7
10		823	
	RESIDUUM: Firm to stiff gray orange brown sandy SILT (ML), micaceous		7
15	PARTIALLY WEATHERED ROCK: Sampled as very dense white orange gray silty medium to fine SAND (SM)	818	50/1"
	Refusal at 17 feet		
20			
25			
30			
35			
40			



SOIL BORING RECORD 118337 MARGARET PHILLIPS DAM.GPJ PIEDMONT GEO.GDT 12/3/19

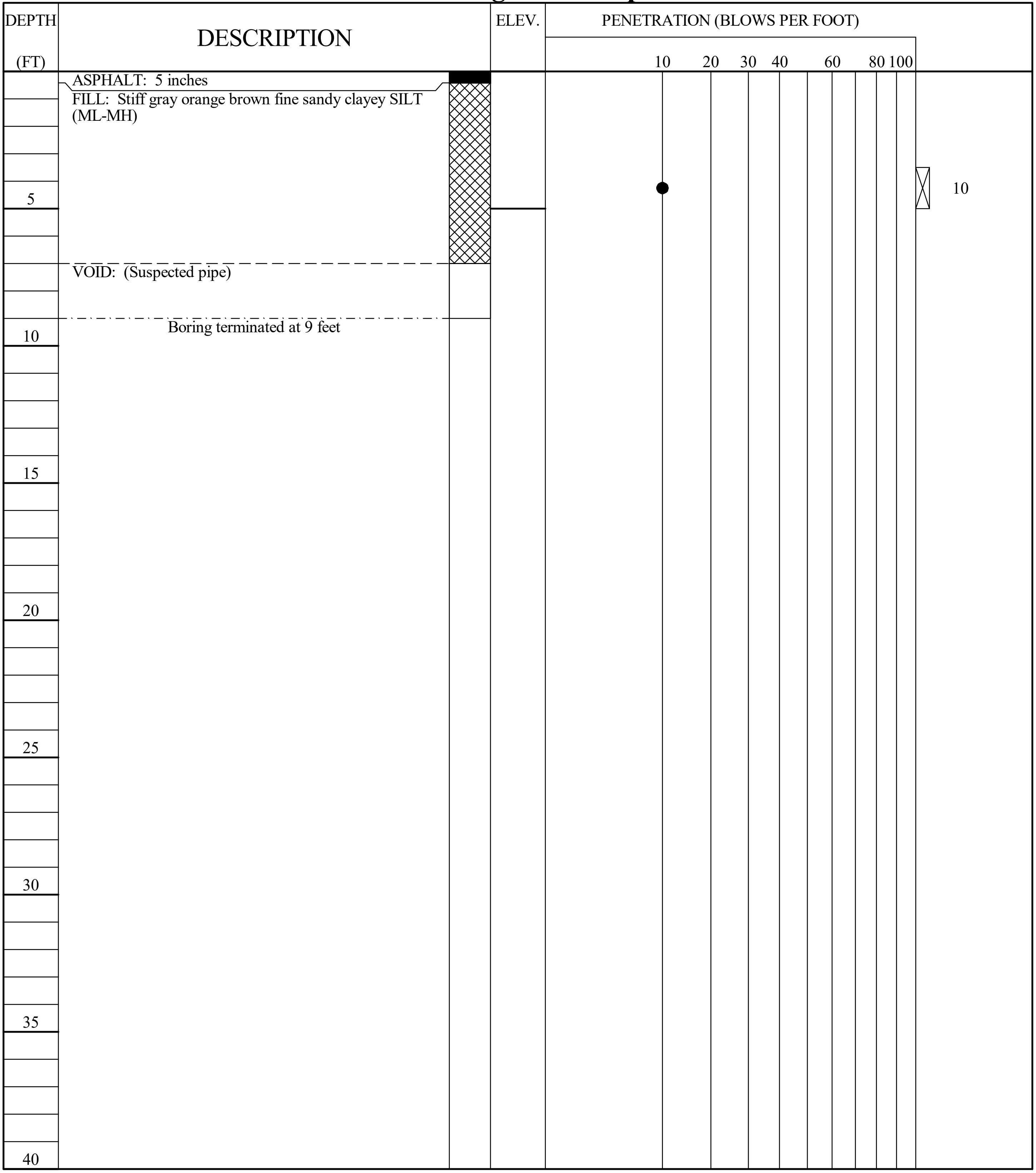
REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

SOIL BORING RECORD

BORING NUMBER	B-1
DATE DRILLED	5/9/2019
PROJECT NUMBER	118337
PAGE	1 of 1

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

Margaret Phillips Lake Dam



ASPHALT: 5 inches

FILL: Stiff gray orange brown fine sandy clayey SILT (ML-MH)

VOID: (Suspected pipe)

Boring terminated at 9 feet

REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with group upon completion of drilling.

∇ Groundwater level at time of boring

▼ Groundwater level - 24 hrs

C Caved depth - 24 hrs

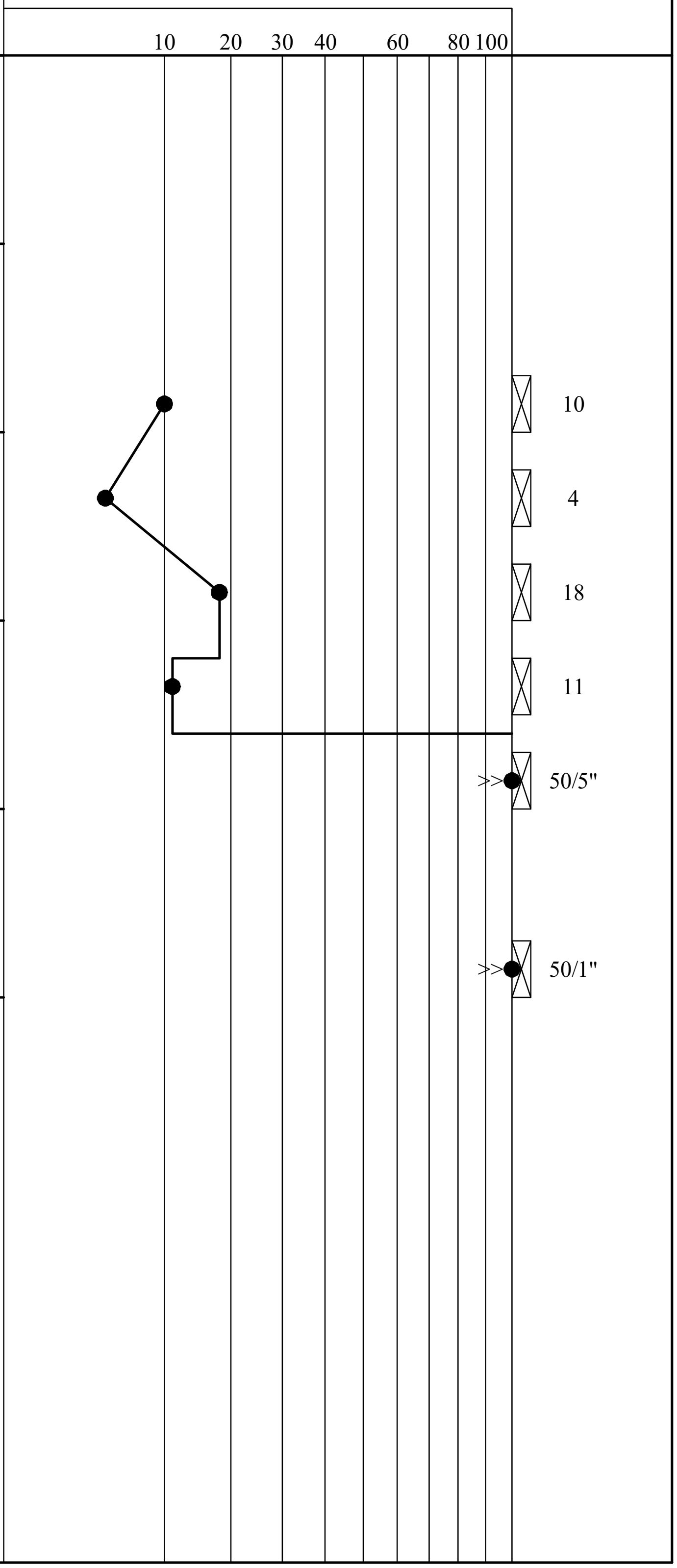
■ Undisturbed sample

SOIL BORING RECORD

BORING NUMBER	B-2
DATE DRILLED	5/7/2019
PROJECT NUMBER	118337
PAGE	1 of 1

Margaret Phillips Lake Dam

DEPTH (FT)	DESCRIPTION	ELEV. 832	PENETRATION (BLOWS PER FOOT)
	ASPHALT: 5 inches Offset Boring - No samples taken to 8.5 feet		
5		827	
	FILL: Stiff gray orange brown fine sandy clayey SILT (ML-MH)		
10		822	10
	Very loose gray tan brown silty medium to fine SAND (SM)		4
	Medium dense gray tan brown silty medium to fine SAND (SM)		18
15		817	
	ALLUVIUM : Medium dense tan gray silty medium to fine SAND (SM), trace organics		11
	RESIDUUM: Medium dense gray white silty medium to fine SAND (SM)		50/5"
20	PARTIALLY WEATHERED ROCK: Sampled as very dense white gray silty medium to fine SAND (SM), micaceous	812	
	PARTIALLY WEATHERED ROCK: No sample recovered		50/1"
25	Refusal at 25 feet Note: Boring B-2A offset 10 feet left of Boring B-2.	807	
30			
35			
40			



REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

SOIL BORING RECORD

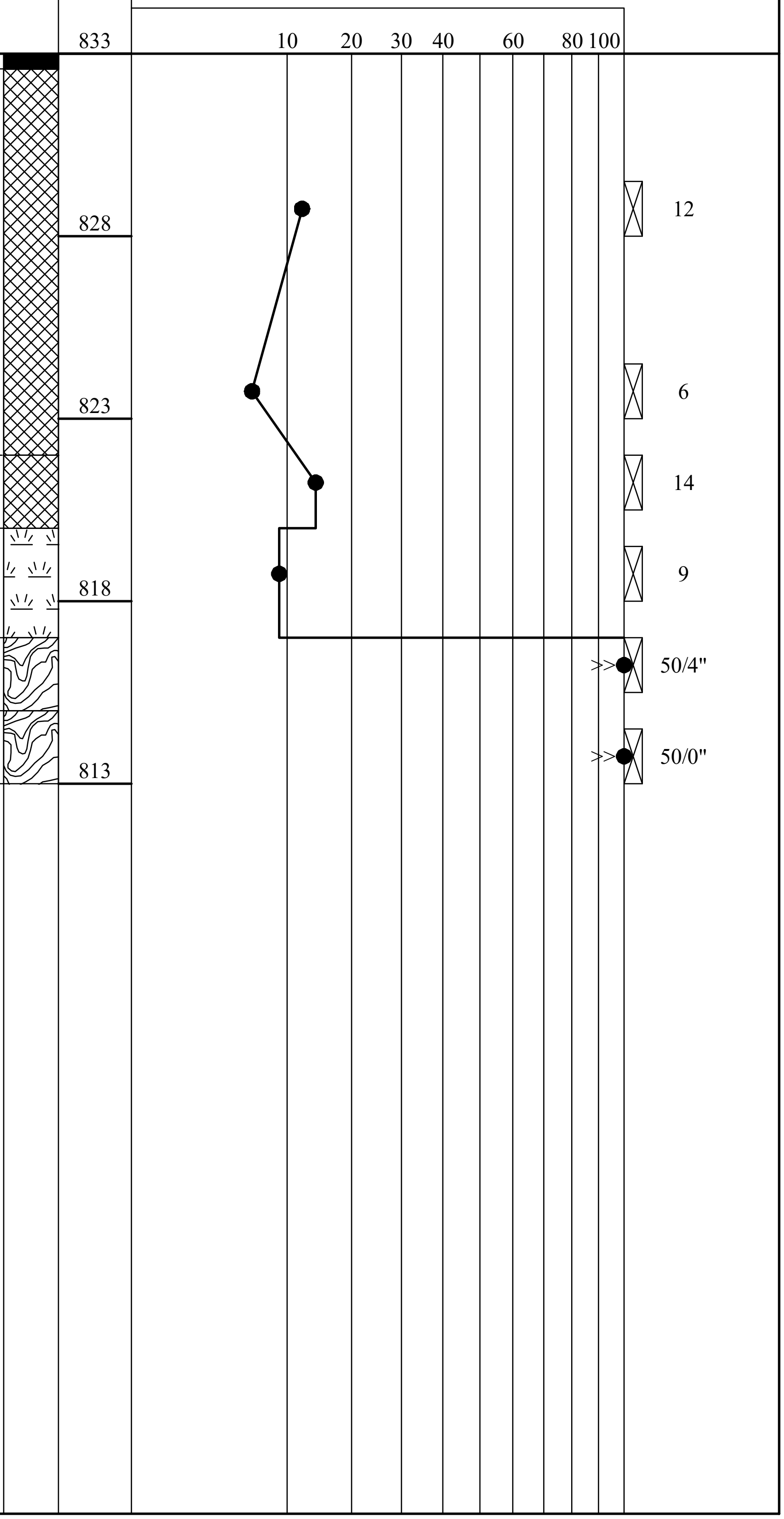
BORING NUMBER	B-2A
DATE DRILLED	5/7/2019
PROJECT NUMBER	118337
PAGE	1 of 1

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

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
— AN ATLAS COMPANY —

Margaret Phillips Lake Dam

DEPTH (FT)	DESCRIPTION	ELEV. 833	PENETRATION (BLOWS PER FOOT)
	ASPHALT: 5 inches		
	FILL: Stiff to firm gray orange brown fine sandy clayey SILT (ML-MH)		
5		828	12
10		823	6
	Stiff gray tan brown sandy clayey SILT (ML), rock fragments		14
	ALLUVIUM: Loose blue gray clayey silty medium to fine SAND (SM-SC)		9
15		818	
	PARTIALLY WEATHERED ROCK: Sampled as very dense black white gray silty medium to fine SAND (SM)		50/4"
	PARTIALLY WEATHERED ROCK: No sample recovered		50/0"
20		813	
	Refusal at 20 feet		
25			
30			
35			
40			



SOIL BORING RECORD 118337 MARGARET PHILLIPS DAM.GPJ PIEDMONT GEO.GDT 12/3/19

REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

SOIL BORING RECORD

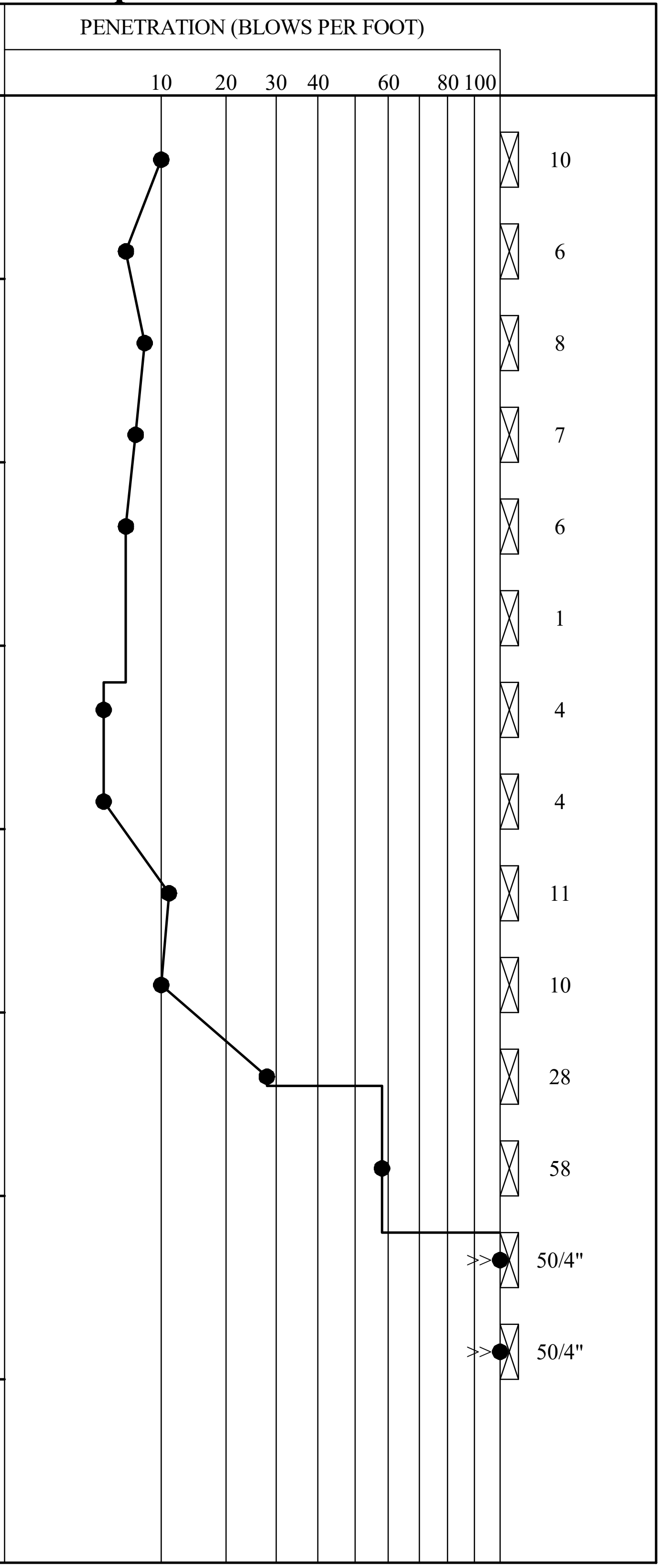
BORING NUMBER	B-3
DATE DRILLED	5/7/2019
PROJECT NUMBER	118337
PAGE	1 of 1

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

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
— AN ATLAS COMPANY —

Margaret Phillips Lake Dam

DEPTH (FT)	DESCRIPTION	ELEV. 832
	ASPHALT: 9 inches	
	FILL: Stiff gray orange brown fine sandy clayey SILT (ML-MH)	
	Firm gray orange brown fine sandy clayey SILT (ML-MH)	
5		827
10		822
	Loose gray tan brown silty medium to fine SAND (SM)	
	Very loose gray tan brown silty medium to fine SAND (SM)	
15		817
	ALLUVIUM: Very loose gray black brown silty SAND (SM), organics. Drilling circulation temporarily lost.	
	Very loose blue gray silty medium to fine SAND (SM)	
20		812
	Medium dense to loose gray white brown silty medium to fine SAND (SM), trace organics	
25		807
	Medium dense gray white brown silty medium to fine SAND (SM), quartz fragments	
	RESIDUUM: Very dense white gray silty coarse SAND (SM), quartz pieces	
30		802
	PARTIALLY WEATHERED ROCK: Sampled as very dense black white gray medium to fine SAND (SM), micaceous	
35	Boring terminated at 35 feet	797
40		



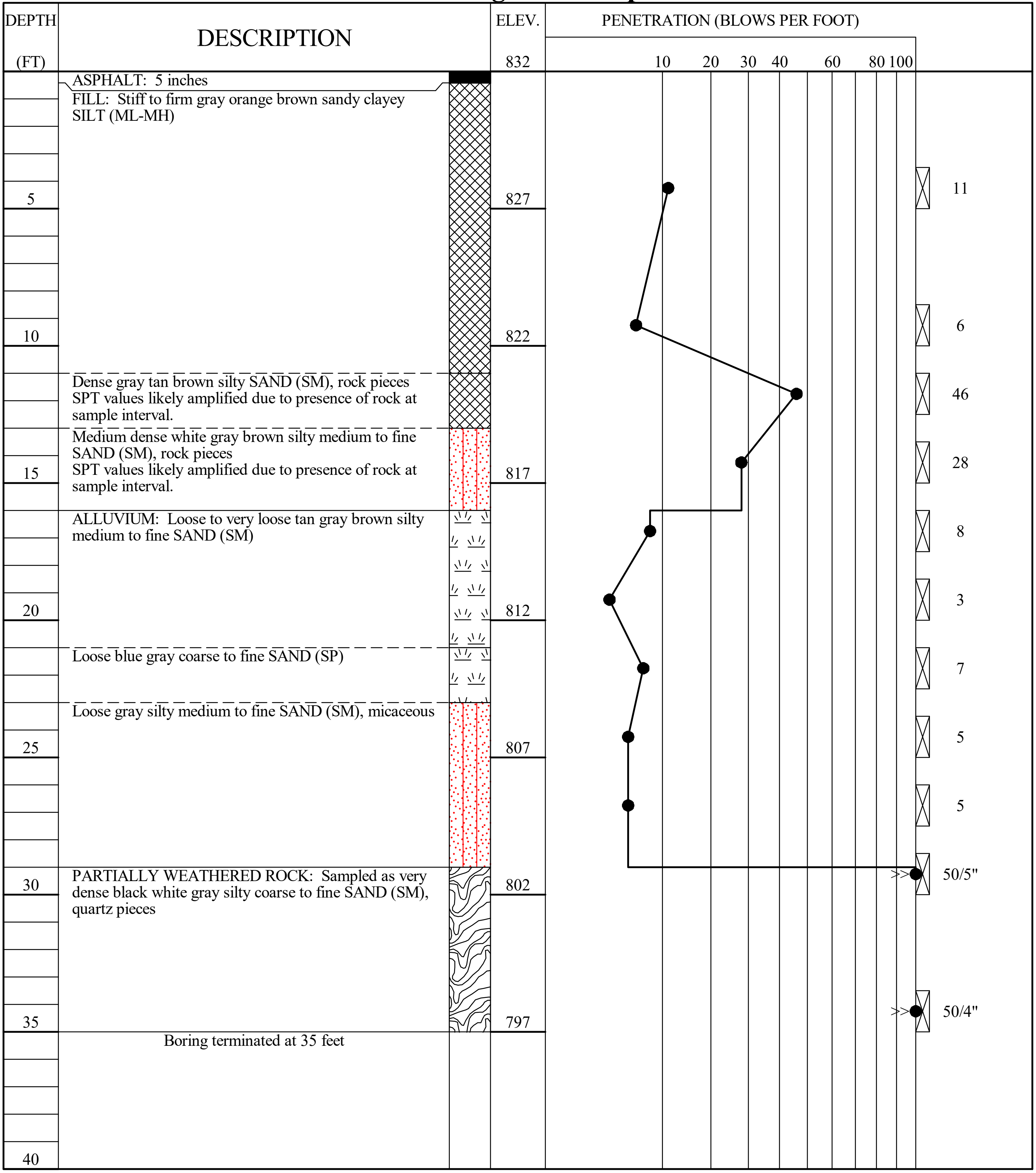
REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

SOIL BORING RECORD

BORING NUMBER	B-4
DATE DRILLED	5/6/2019
PROJECT NUMBER	118337
PAGE	1 of 1

∇ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
▮ Undisturbed sample

Margaret Phillips Lake Dam



ASPHALT: 5 inches

FILL: Stiff to firm gray orange brown sandy clayey SILT (ML-MH)

Dense gray tan brown silty SAND (SM), rock pieces
SPT values likely amplified due to presence of rock at sample interval.

Medium dense white gray brown silty medium to fine SAND (SM), rock pieces
SPT values likely amplified due to presence of rock at sample interval.

ALLUVIUM: Loose to very loose tan gray brown silty medium to fine SAND (SM)

Loose blue gray coarse to fine SAND (SP)

Loose gray silty medium to fine SAND (SM), micaceous

PARTIALLY WEATHERED ROCK: Sampled as very dense black white gray silty coarse to fine SAND (SM), quartz pieces

Boring terminated at 35 feet

REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

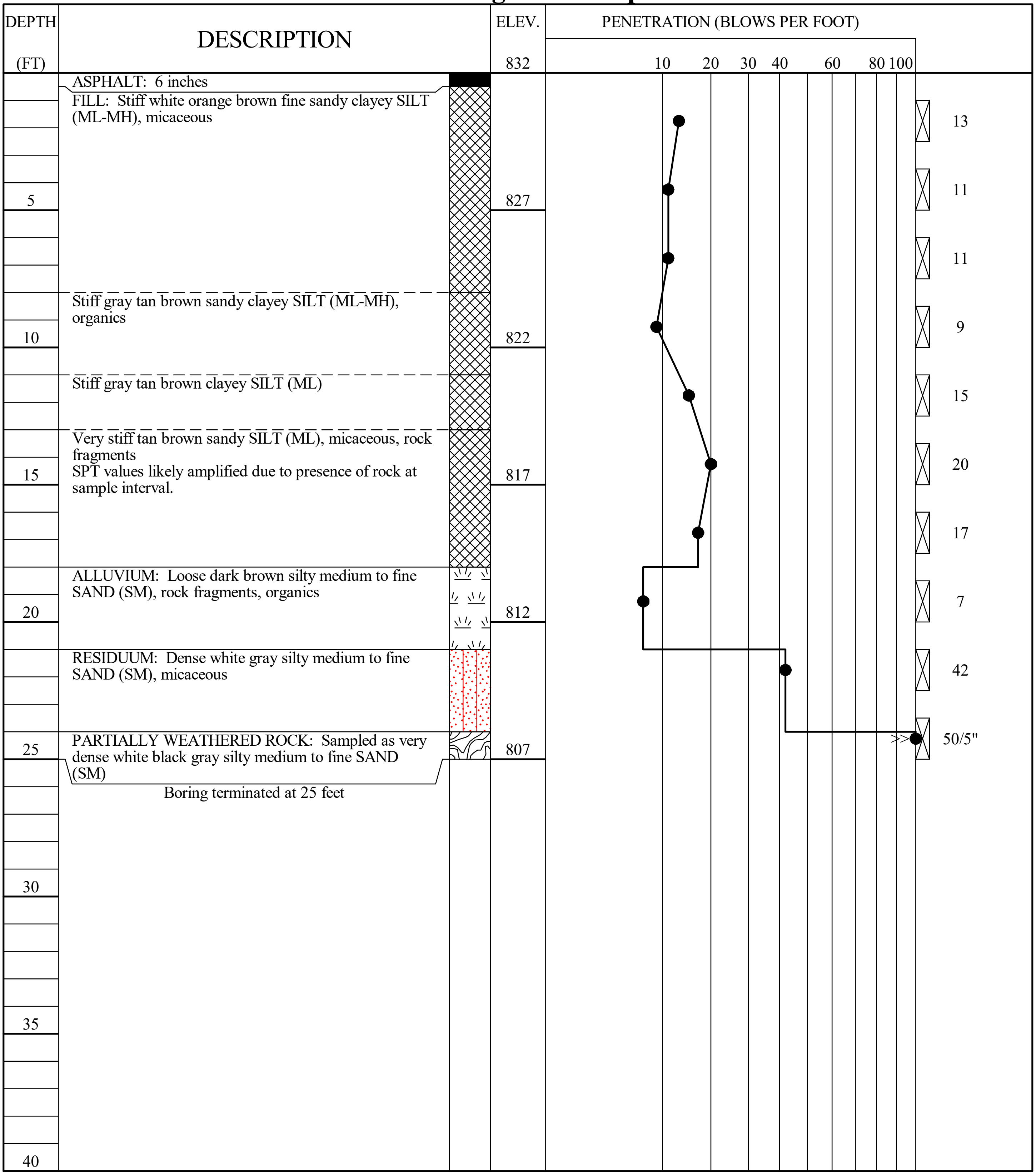
▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
◧ Undisturbed sample

SOIL BORING RECORD

BORING NUMBER	B-5
DATE DRILLED	5/8/2019
PROJECT NUMBER	118337
PAGE	1 of 1

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
— AN ATLAS COMPANY —

Margaret Phillips Lake Dam



ASPHALT: 6 inches

FILL: Stiff white orange brown fine sandy clayey SILT (ML-MH), micaceous

Stiff gray tan brown sandy clayey SILT (ML-MH), organics

Stiff gray tan brown clayey SILT (ML)

Very stiff tan brown sandy SILT (ML), micaceous, rock fragments
SPT values likely amplified due to presence of rock at sample interval.

ALLUVIUM: Loose dark brown silty medium to fine SAND (SM), rock fragments, organics

RESIDUUM: Dense white gray silty medium to fine SAND (SM), micaceous

PARTIALLY WEATHERED ROCK: Sampled as very dense white black gray silty medium to fine SAND (SM)

Boring terminated at 25 feet

REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

SOIL BORING RECORD

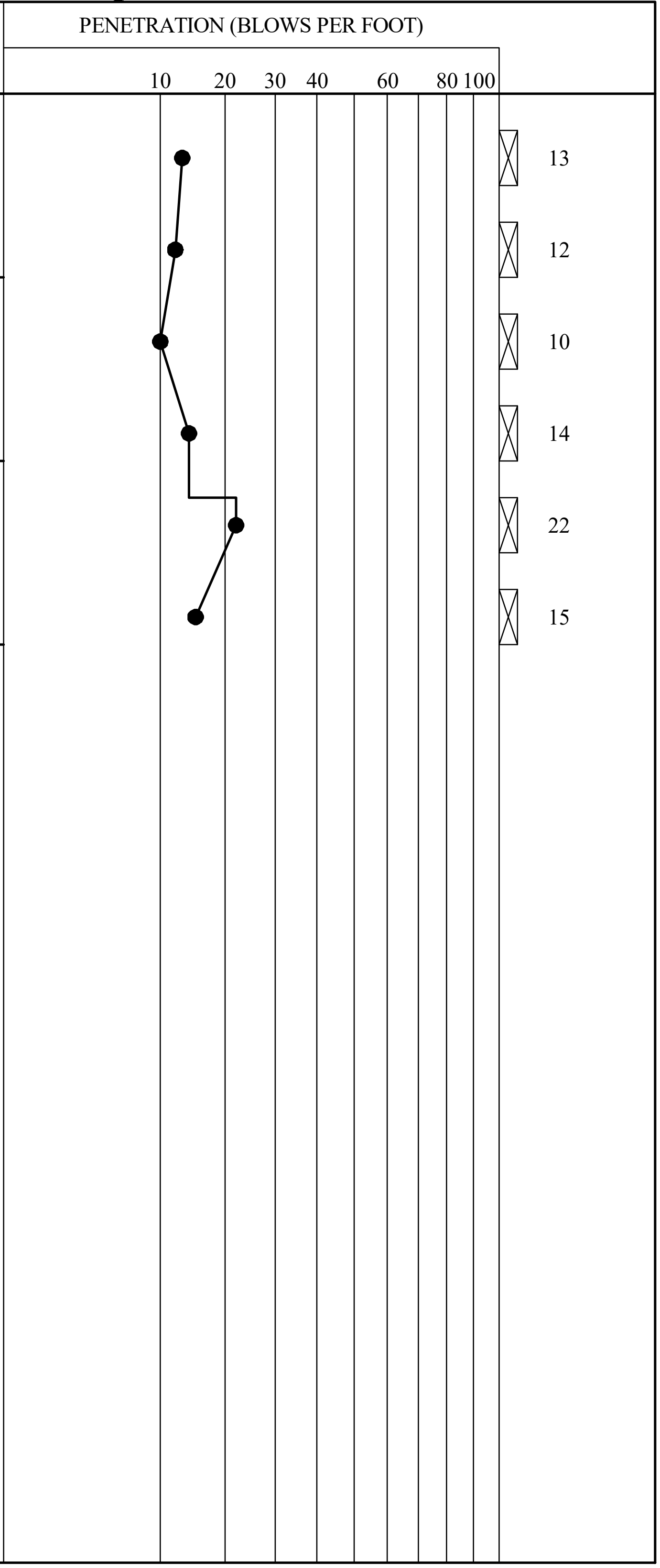
BORING NUMBER	B-6
DATE DRILLED	5/9/2019
PROJECT NUMBER	118337
PAGE	1 of 1

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

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
— AN ATLAS COMPANY —

Margaret Phillips Lake Dam

DEPTH (FT)	DESCRIPTION	ELEV. 832
	ASPHALT: 5 inches	
	FILL: Stiff white orange brown fine sandy clayey SILT (ML-MH)	
5		827
	Stiff gray orange brown sandy silty CLAY (CL-CH)	
10		822
	RESIDUUM: Medium dense white gray brown silty medium to fine SAND (SM), micaceous	
	Medium dense orange tan brown silty medium SAND (SM), micaceous	
15		817
	PARTIALLY WEATHERED ROCK: Hard drilling. No samples taken.	
	Refusal at 18 feet	
20		
25		
30		
35		
40		



SOIL BORING RECORD 118337 MARGARET PHILLIPS DAM.GPJ PIEDMONT GEO.GDT 12/3/19

REMARKS: Boring advanced using mud-rotary drilling methods. Groundwater measurements not attempted. Borehole filled with grout upon completion of drilling.

SOIL BORING RECORD

BORING NUMBER	B-7
DATE DRILLED	5/10/2019
PROJECT NUMBER	118337
PAGE	1 of 1

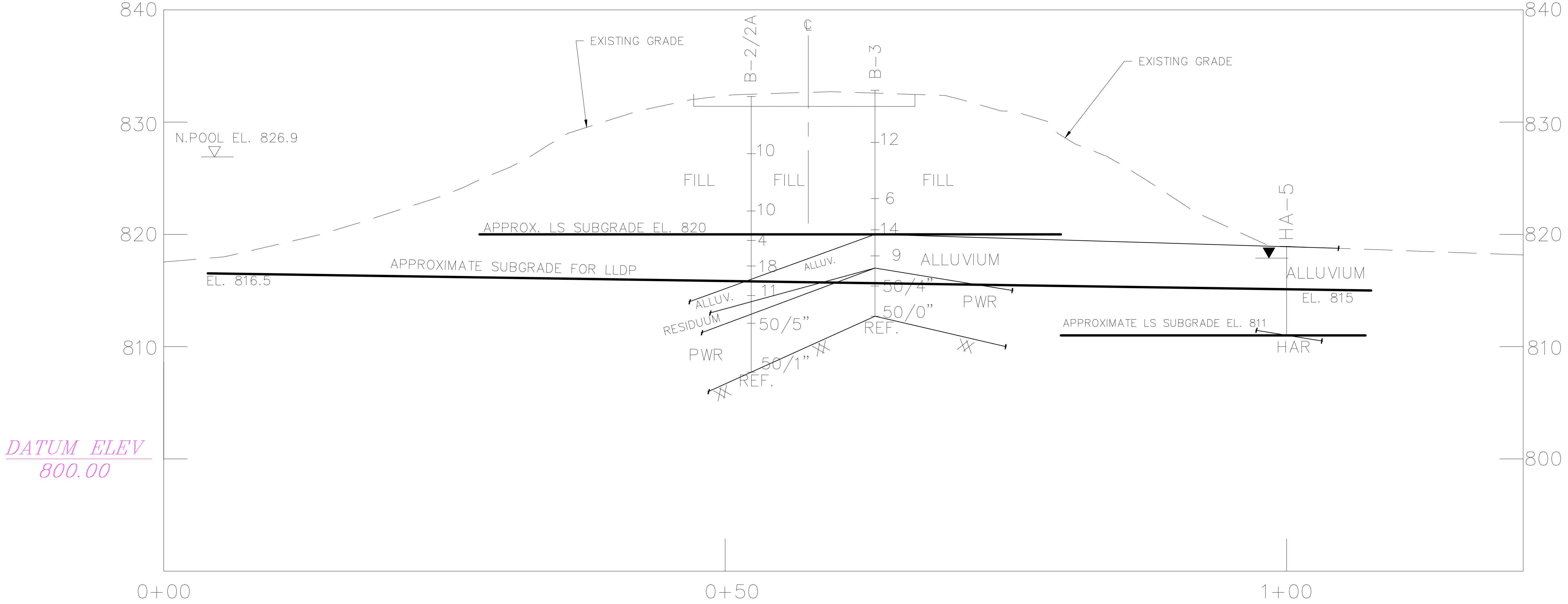
▽ Groundwater level at time of boring	C Caved depth - 24 hrs
▼ Groundwater level - 24 hrs	◨ Undisturbed sample

SUMMARY OF HAND AUGER BORINGS
Margaret Phillips Lake Dam
Longview Road, Fayette County, Georgia
PGC Project No. 118337

Hand Auger Boring No.	Depth (Inches)	Soil Description
HA-1/1A	0-6	TOPSOIL: Dark brown silty clay with organics
	6-18	FILL: Very soft gray tan brown silty CLAY (CL-CH) with organics
	18-42	ALLUVIUM: Soft gray clayey medium to fine SAND (SC) with small gravel
	42	Hand Auger Refusal
		Groundwater measured at -6” following completion of boring. Stabilized groundwater was measured at the ground surface. Ground probes very easily to a depth of 36”.
HA-2	0-6	TOPSOIL: Dark brown silty clay with organics
	6-24	FILL: Very soft gray tan silty CLAY (CL-CH) with organics
	24-36	ALLUVIUM: Firm dark brown clayey medium to fine SAND (SC) with organics
	36-48	Loose slightly clayey silty medium to fine SAND (SP-SM) with small gravel
	48-54	RESIDUUM: Loose to medium dense gray tan micaceous silty fine SAND (SM)
	54	Boring Terminated
		Groundwater measured at -12” following completion of boring. Stabilized groundwater measured at the ground surface. Ground probes very easily to a depth of 36”.
HA-3	0 – 6	TOPSOIL: Dark brown silty clay with organics
	6-42	ALLUVIUM: Very soft dark brown silty CLAY (CL-CH) with organics
	42-84	Dark brown silty fine SAND (SM-SP) with organics
	84-108	Dark brown to black silty medium to fine SAND (SP-SM); flowing sand conditions/hole collapsed and was cased using PVC pipe.
	108-120	Bluish gray coarse to fine SAND (SP) with small gravel; flowing sand conditions/hole collapsed and was cased using PVC pipe.
	120	Hand Auger Refusal on large gravel.
		Groundwater measured at -6” following completion of boring. Stabilized groundwater measured at +10” above the ground inside PVC pipe. Ground probes very easily to a depth of 60”.

SUMMARY OF HAND AUGER BORINGS
Margaret Phillips Lake Dam
Longview Road, Fayette County, Georgia
PGC Project No. 118337

Hand Auger Boring No.	Depth (Inches)	Soil Description
HA-4	0-12	FILL: Soft red tan silty CLAY (CL-CH)
	12-60	ALLUVIUM: Loose tan brown silty medium to fine SAND (SP)
	60-108	Loose gray brown slightly clayey silty medium to fine SAND (SM-SC); flowing sand conditions/ hole collapsed and was cased using PVC pipe.
	108-130	Loose tan coarse to fine SAND (SP) with small gravel; flowing sand conditions/ hole collapsed and was cased using PVC pipe.
	130-132	Very firm gray plastic silty CLAY (CH)
	132	Hand Auger Refusal
		Groundwater measured at the ground surface following the completion of boring. Stabilized groundwater measured at +24" above the ground inside PVC pipe. Ground probes very easily to a depth of 60".
HA-5	0-6	TOPSOIL: Dark brown silty clay with organics
	6-36	ALLUVIUM: Tan gray brown silty medium to fine SAND (SM)
	36-72	Loose tan brown silty medium to fine SAND (SP); flowing sand conditions/hole collapsed and was cased using PVC pipe.
	72-88	Gray blue medium to fine sandy CLAY (SC-CH)
	88-96	Medium dense gray brown coarse to fine SAND (SP)
	96-98	RESIDUUM: Medium dense gray brown micaceous silty medium to fine SAND (SM)
	98	Hand Auger Refusal
		Groundwater measured at -24" following completion of the boring. Stabilized groundwater measured at -12". Ground probes very easily to a depth of 36".
HA-6/6A	0-4	TOPSOIL: Dark brown silty clay with organics
	4-20	FILL: Tan brown silty medium to fine SAND (SM-SP)
	20	Hand Auger Refusal
		No groundwater encountered at the completion of the boring. Stabilized groundwater measured at -16". Ground probes very easily to a depth of 12".



SECTION A-A

NOTE:

WHILE INDIVIDUAL SOIL BORING RECORDS AND HAND AUGER BORINGS ARE CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THE RESPECTIVE TEST LOCATIONS ON THE DATES PERFORMED, IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS INTERMEDIATE OF THE TEST LOCATIONS AND/OR AT OTHER TIMES. ESTIMATES OF THE SUBSURFACE CONDITIONS ARE BASED ON ACCEPTED SOIL ENGINEERING PRINCIPALS AND PRACTICES AND REASONABLE ENGINEERING JUDGEMENT.

KEY TO SYMBOLS

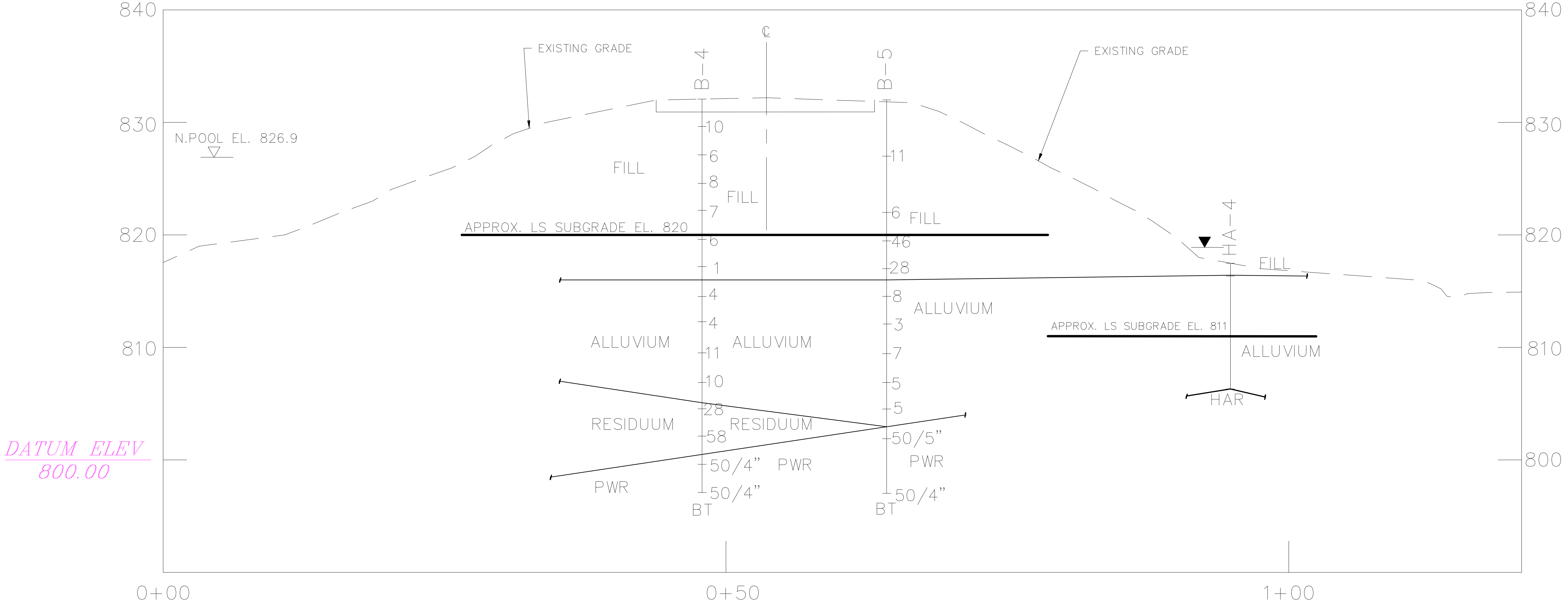
B- # - SOIL TEST BORING
HA - # - HAND AUGER BORING
PWR - PARTIALLY WEATHERED ROCK
BT - BORING TERMINATED
REF - REFUSAL TO DRILL RIG
HAR - HAND AUGER REFUSAL
▼ - STABILIZED GROUNDWATER

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD FAYETTEVILLE, GEORGIA
PGC PROJECT NO. 118337

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
3000 NORTHFIELD PLACE, SUITE 1100
ROSWELL, GEORGIA 30076
(770) 752-9205

FIGURE 2: SECTION A-A

SCALE: 1" = 10' (H/V)



SECTION B-B

NOTE:

WHILE INDIVIDUAL SOIL BORING RECORDS AND HAND AUGER BORINGS ARE CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THE RESPECTIVE TEST LOCATIONS ON THE DATES PERFORMED, IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS INTERMEDIATE OF THE TEST LOCATIONS AND/OR AT OTHER TIMES. ESTIMATES OF THE SUBSURFACE CONDITIONS ARE BASED ON ACCEPTED SOIL ENGINEERING PRINCIPALS AND PRACTICES AND REASONABLE ENGINEERING JUDGEMENT.

KEY TO SYMBOLS

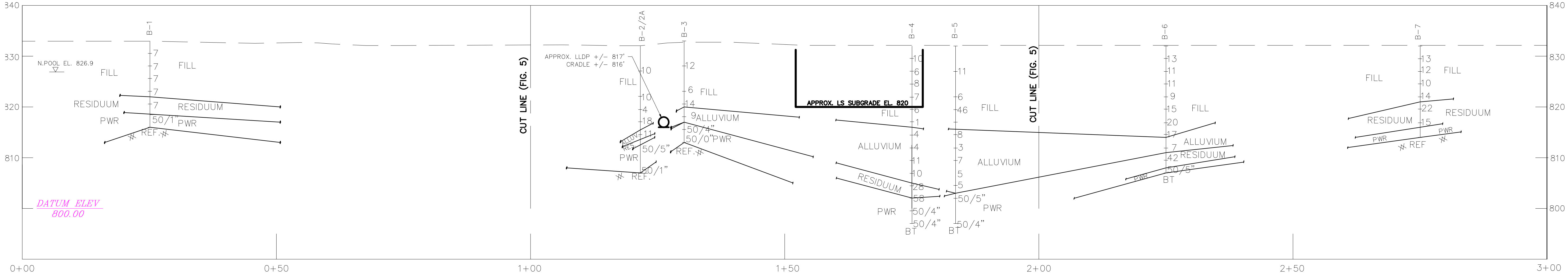
B- # - SOIL TEST BORING
HA - # - HAND AUGER BORING
PWR - PARTIALLY WEATHERED ROCK
BT - BORING TERMINATED
REF - REFUSAL TO DRILL RIG
HAR - HAND AUGER REFUSAL
▼ - STABILIZED GROUNDWATER

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD FAYETTEVILLE, GEORGIA
PGC PROJECT NO. 118337

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
3000 NORTHFIELD PLACE, SUITE 1100
ROSWELL, GEORGIA 30076
(770) 752-9205

FIGURE 3: SECTION B-B

SCALE: 1" = 10' (H/V)



SECTION C-C

NOTE:

1) WHILE INDIVIDUAL SOIL BORING RECORDS AND HAND AUGER BORINGS ARE CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THE RESPECTIVE TEST LOCATIONS ON THE DATES PERFORMED, IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS INTERMEDIATE OF THE TEST LOCATIONS AND/OR AT OTHER TIMES. ESTIMATES OF THE SUBSURFACE CONDITIONS ARE BASED ON ACCEPTED SOIL ENGINEERING PRINCIPALS AND PRACTICES AND REASONABLE ENGINEERING JUDGEMENT.

2) BORINGS B-2/2A AND B-3 AND B-4 AND B-5 ARE LOCATED APPROXIMATELY UPSTREAM/DOWNSTREAM OF EACH OTHER ALONG THE SAME RELATIVE LONGITUDINAL POSITION ON THE DAM. BOTH PAIRS OF BORINGS ARE SHOWN ON SECTION C-C SO THE READER CAN BETTER UNDERSTAND THE ANTICIPATED ELEVATION DIFFERENCES BETWEEN BORINGS ACROSS THIS TRANSVERSE SECTION OF THE DAM.

KEY TO SYMBOLS

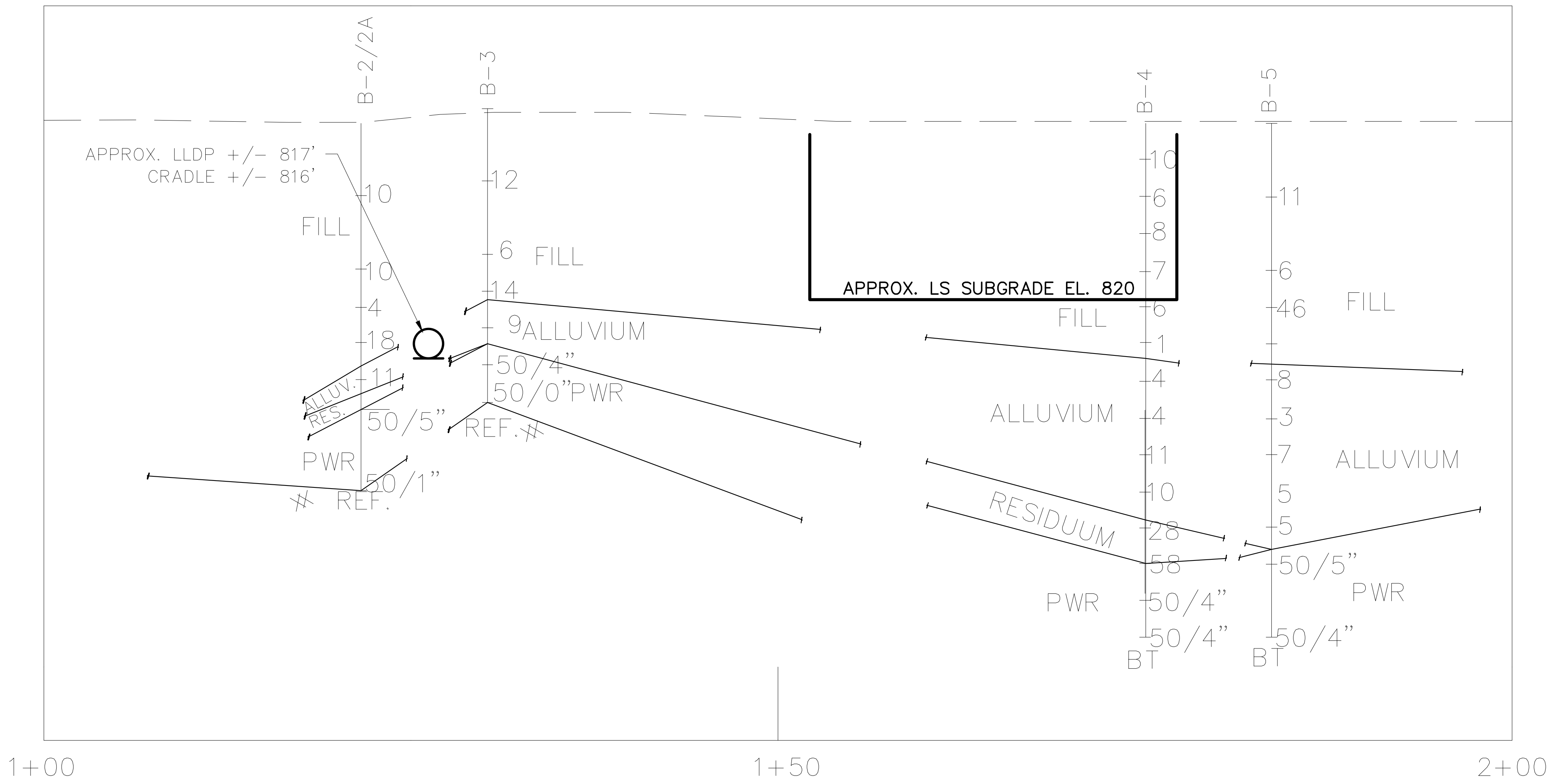
B- # - SOIL TEST BORING
HA - # - HAND AUGER BORING
PWR - PARTIALLY WEATHERED ROCK
BT - BORING TERMINATED
REF - REFUSAL TO DRILL RIG
HAR - HAND AUGER REFUSAL
▼ - STABILIZED GROUNDWATER

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD FAYETTEVILLE, GEORGIA
PGC PROJECT NO. 118337

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
3000 NORTHFIELD PLACE, SUITE 1100
ROSWELL, GEORGIA 30076
(770) 752-9205

FIGURE 4: SECTION C-C

SCALE: 1" = 20' (H/V)



SECTION C-C (STATION 1+00-2+00)

NOTE:

1) WHILE INDIVIDUAL SOIL BORING RECORDS AND HAND AUGER BORINGS ARE CONSIDERED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THE RESPECTIVE TEST LOCATIONS ON THE DATES PERFORMED, IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS INTERMEDIATE OF THE TEST LOCATIONS AND/OR AT OTHER TIMES. ESTIMATES OF THE SUBSURFACE CONDITIONS ARE BASED ON ACCEPTED SOIL ENGINEERING PRINCIPALS AND PRACTICES AND REASONABLE ENGINEERING JUDGEMENT.

2) BORINGS B-2/2A AND B-3 AND B-4 AND B-5 ARE LOCATED APPROXIMATELY UPSTREAM/DOWNSTREAM OF EACH OTHER ALONG THE SAME RELATIVE LONGITUDINAL POSITION ON THE DAM. BOTH PAIRS OF BORINGS ARE SHOWN ON SECTION C-C SO THE READER CAN BETTER UNDERSTAND THE ANTICIPATED ELEVATION DIFFERENCES BETWEEN BORINGS ACROSS THIS TRANSVERSE SECTION OF THE DAM.

KEY TO SYMBOLS

B- # - SOIL TEST BORING
HA - # - HAND AUGER BORING
PWR - PARTIALLY WEATHERED ROCK
BT - BORING TERMINATED
REF - REFUSAL TO DRILL RIG
HAR - HAND AUGER REFUSAL
▼ - STABILIZED GROUNDWATER

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD FAYETTEVILLE, GEORGIA
PGC PROJECT NO. 118337

PIEDMONT
GEOTECHNICAL CONSULTANTS, LLC
3000 NORTHFIELD PLACE, SUITE 1100
ROSWELL, GEORGIA 30076
(770) 752-9205

FIGURE 5: SECTION C-C (STATION 1+00 - 2+00)

SCALE: 1" = 10' (H/V)

SUMMARY OF ANTICIPATED UNDERCUTTING
MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA
PGC PROJECT NUMBER 118137

BORING NUMBER	GROUND SURFACE (FT-MSL)	THICKNESS OF ASPHALT PAVEMENT (INCHES)	DEPTH OF EXISTING TOPSOIL/FILL (FEET)	DEPTH OF ALLUVIUM (FEET)	DEPTH OF ANTICIPATED UNDERCUTTING (FEET)	MINIMUM ELEVATION OF ANTICIPATED UNDERCUTTING (FT-MSL)
B-1	833	6	0-11	–	11	822
B-2/B-2A	832	5/5	0-16	16-18	18-20*	812-814
B-3	832	5	0-13	13-16	16	817
B-4	832	9	0-16	16-27	27	805
B-5	832	5	0-16	16-29	29	803
B-6	832	6	0-18	18-21	18-21	811-814
B-7	832	5	0-11	–	11	821
HA-1/1A	821	N/A	0-1.5	1.5-3.5	3.5	817
HA-2	820	N/A	0-2	2-4	4	816
HA-3	818	N/A	0-0.5	0.5-10+	7-12+*	806-811
HA-4	818	N/A	0-1	1-11+	7-12+*	806-811
HA-5	818	N/A	0-0.5	0.5-8	3-9+*	809-815
HA-6/6A	820	N/A	0-2	--	2	818

(+) DENOTES DEPTHS GREATER THAN EXPLORED DURING THIS STUDY.

(*) DENOTES POTENTIAL RANGE OF UNDERCUTTING WITH TOTAL DEPTH TO BE DETERMINED AT THE TIME OF CONSTRUCTION.

MARGARET PHILLIPS LAKE DAM

LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA
PGC PROJECT NUMBER 118337

VIEW LOOKING SOUTH ALONG LONGVIEW ROAD FROM NEAR SOUTH END OF DAM.

VIEW LOOKING NORTH ALONG LONGVIEW ROAD FROM NEAR SOUTH END OF DAM.

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA
PGC PROJECT NUMBER 118337

VIEW LOOKING SOUTH ALONG THE UPSTREAM SLOPE FROM NEAR THE NORTH END OF THE DAM.

VIEW LOOKING NORTH ALONG UPSTREAM SLOPE FROM NEAR THE NORTH END OF THE DAM.

MARGARET PHILLIPS LAKE DAM

LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA
PGC PROJECT NUMBER 118337

VIEW LOOKING SOUTH ALONG THE DOWNSTREAM SLOPE FROM ABOUT MIDWAY OF THE DAM.

VIEW LOOKING NORTH ALONG THE DOWNSTREAM SLOPE FROM NEAR THE SOUTH END OF THE DAM.

VIEW OF TWO CONCRETE PIPES LOCATED NEAR THE NORTH (LEFT) END OF THE DAM. PIPES ARE CURRENTLY SERVING AS SERVICE SPILLWAY (ORIGINALLY EMERGENCY SPILLWAY PIPES).

VIEW FACING SOUTH ALONG THE DOWNSTREAM TOE OF DAM WITHIN POWERLINE EASEMENT. WATER FLOWS WITHIN THE EASEMENT FOR ABOUT 400 FEET BEFORE ENTERING STREAM.

MARGARET PHILLIPS LAKE DAM

LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA
PGC PROJECT NUMBER 118337

WATER FLOW MEANDERS ONCE THE FLOODPLAIN IS ENCOUNTERED. THE DAM IS TO THE RIGHT.

VIEW FACING NORTH ALONG THE POWERLINE EASEMENT FROM NEAR THE STREAM CHANNEL.

VIEW FACING SOUTH ALONG THE POWERLINE EASEMENT FROM NEAR THE STREAM CHANNEL.

2012 PHOTO OF LAKE FROM NEAR THE EMERGENCY SPILLWAY PIPES AT THE LEFT END OF THE DAM. THE LAKE IS APPROXIMATELY 2 FEET BELOW ITS NORMAL POOL.

MARGARET PHILLIPS LAKE DAM
LONGVIEW ROAD
FAYETTE COUNTY, GEORGIA
PGC PROJECT NUMBER 118337

2012 PHOTO: VIEW OF ORIGINAL CORRUGATED METAL OVERFLOW PIPE. LAKE LEVEL IS ABOUT 2 FEET BELOW NORMAL POOL DUE TO LEAKAGE THROUGH THE PIPE.

2012 PHOTO: VIEW OF DOWNSTREAM AREA WHERE THE PSP DISCHARGES. THE DOWNSTREAM END OF THE PIPE IS SUBMERGED.