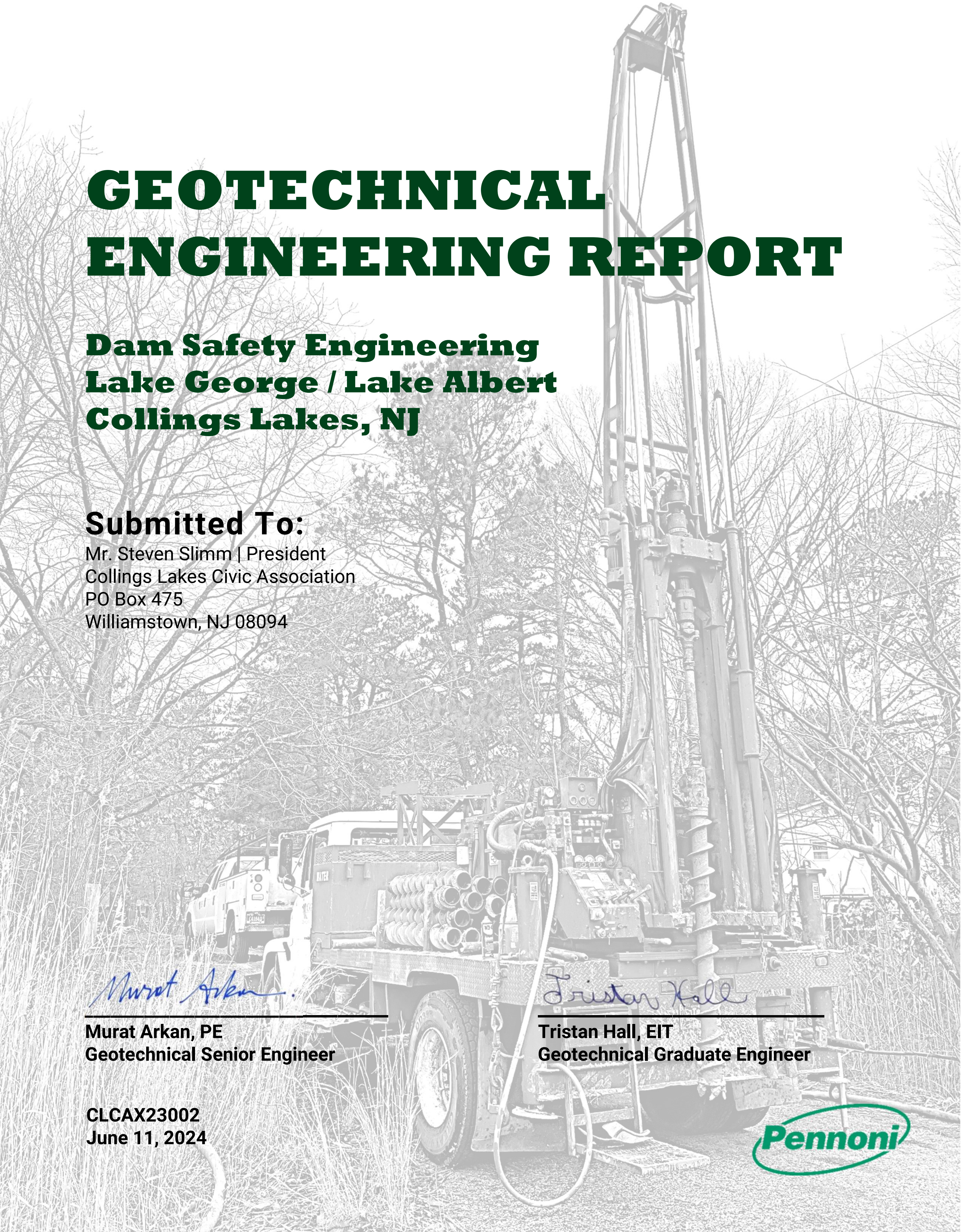


GEOTECHNICAL ENGINEERING REPORT

Dam Safety Engineering
Lake George / Lake Albert
Collings Lakes, NJ

Submitted To:

Mr. Steven Slimm | President
Collings Lakes Civic Association
PO Box 475
Williamstown, NJ 08094

Murat Arkan

Murat Arkan, PE
Geotechnical Senior Engineer

Tristan Hall

Tristan Hall, EIT
Geotechnical Graduate Engineer

CLCAX23002
June 11, 2024

Pennoni

June 11, 2024

CLCAX23002

Mr. Steven Slimm | President
Collings Lakes Civic Association
PO Box 475
Williamstown, NJ 08094

RE: Geotechnical Engineering Services
Lake George – NJ DAM File #31-103
Lake Albert – NJ Dam File #31-104
Collings Lakes, NJ

Dear Mr. Slimm:

We are pleased to submit our geotechnical engineering report and stability analysis for the proposed dam rehabilitation for the Lake George Dam (Dam File #31-103) and the Lake Albert Dam (Dam File #31104) located in Collings Lakes, New Jersey. This study was initiated in general accordance with the Scope of Services presented in our proposal dated September 18, 2023, and your subsequent authorization to proceed.

We trust that the information presented in this report is what you require at this time, and we thank you for the opportunity to assist you with this project. If you have any questions, or if you need any further assistance with this project, please contact this office at your earliest convenience.

Respectfully,

PENNONI ASSOCIATES INC.

Tristan Hall, EIT
Geotechnical Graduate Engineer

Murat Arkan, PE
Geotechnical Senior Engineer

CONTENTS

TABLES

1.0 Executive Summary

The results of our geotechnical study are summarized below. The executive summary should be used in conjunction with the body of the report as it details our recommendations.

PROPOSED CONSTRUCTION AND SITE CONDITIONS		*SECTION(S)*
Proposed Construction	Installing sheet piling on the upstream sides of the dams; rehabilitating Lake Albert spillway, installing a new trapezoidal spillway on the Lake Albert embankment.	2.1
Surface Materials	4 in. thick asphalt layer and embankment fill at the surface. Sites occupied by earthen embankments, timber drop box, timber pedestrian bridge, bituminous pavement, vegetation, flashboard controller weir and broad crested reinforced concrete weir.	2.1, 4.2
Subsoils	Stratum F – FILL: Sand, little Silt, trace Gravel (6 ft thick in one boring) Stratum 1 – Sand, trace Silt (4 ft thick) Stratum 2 – Sand, trace Silt, varying amounts of Clay/Organics (5 to 12 ft thick) Stratum 3 – Sand, trace Silt, varying amounts of Gravel (borings terminated in this layer) Stratum 4 – Silt, little Sand (5 to 6 ft thick, interbedded in Stratum 3)	4.2
Field Work	8 SPT borings advanced from 40 to 60 ft below the ground surface.	3.1

RECOMMENDATIONS		*SECTION(S)*
Seismic Site Class	D – a stiff soil profile in accordance with IBC 2018.	5.5
Lift Thickness and Compaction Requirements	Fine Grained Soil – 8 to 10 in. thick layers loose measure; Granular Soil – 10 to 12 in. thick layers loose measure. Foundations 98% (standard Proctor) or 95% (modified Proctor) depending on compaction equipment used. Pavements 95% (standard Proctor) or 93% (modified Proctor) depending on compaction equipment used.	5.8
Foundations	Thickened mat foundation bearing on new fill or Stratum 3 soils, with an allowable bearing capacity of 3,000 psf. Alternatively, grouted helical pile foundation system or timber piles.	5.3
Settlement	Total and differential settlements anticipated to be less than 1 in. and ½ in., respectively. Negligible with the use of pile foundations.	5.4
Frost Depth	36 in. below finished exterior grades.	5.3
Groundwater	Groundwater encountered at depths of 4 to 6 ft (Elev. 59.5 to Elev. 64.5) in the borings. Groundwater should be lowered a minimum of 5 ft below excavation depths using a well-point system or temporary shoring and high-capacity pumps.	4.3, 5.7

2.0 Introduction

2.1 Background

Based on our review of historical records titled, "A Brief History and Overview of Buena Vista Township," dated March 2001, the Collings Lakes area was originally a cranberry bog that was converted into man-made lakes in the 1950s, including the project sites, specifically Lake George and Lake Albert. Residential properties were constructed surrounding the man-made lakes in the 1950s. Dams were constructed between the water bodies, separating the man-made lakes.

The project sites are located within the Borough of Folsom and the Township of Buena Vista, New Jersey. Specifically, the project will encompass the Lake George Dam spillway (Site George) and the Lake Albert Dam spillway (Site Albert). Site George is bounded by one to two story residential houses to the north, by Lake Albert to the east, by residential houses and undeveloped densely wooded area to the south, and by Lake George to the west. Site Albert is bounded by Lake Albert to the north, and by undeveloped densely wooded area to the east, south, and west.

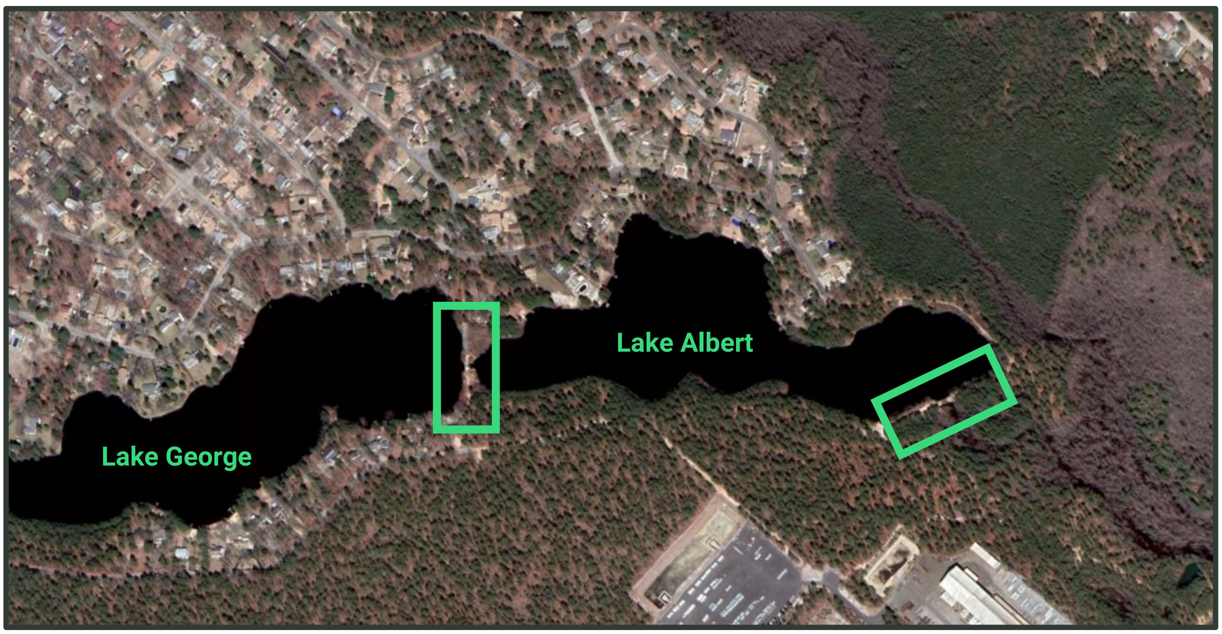


Figure 1: Site Location

Site George consists of an approximately 470 ft long dam with a height of 12 ft, an earthen embankment crest approximately 35 ft in width, and a timber drop box structure with two flashboard-controller weirs with a total length of 9.4 ft. The timber structure additionally supports a pedestrian bridge. The auxiliary spillway immediately north of the drop box consists of a 100 ft long, 25 ft wide bituminous crest. Overhead electrical and telecommunications (T/C reportedly unused) lines traverse along the east side of the dam. Additionally, steel guardrails (removed prior to the field investigation) traverse along the east and west sides of the earthen embankments.

Site Albert consists of an earthen embankment crest approximately 15 ft in width and a spillway with a broad crested reinforced concrete weir 70 ft in width, supported by a 3 in. thick timber pile cutoff wall extending downward 23 ft. 12 in. diameter timber pilings support the cut off wall.

Proposed construction is anticipated to consist of installing sheeting on the upstream sides of both dams, rehabilitating the Lake Albert spillway, and installing a new trapezoidal spillway on the Lake Albert embankment.

2.2 Objectives

The objectives of this geotechnical study were to determine subsurface conditions at the project sites, evaluate these conditions with respect to the proposed construction, and present our conclusions and recommendations regarding:

- seepage analysis of water flow through embankment and subsurface soils beneath the structure;
- discussion of factor of safety against piping;
- slope stability analysis at various cross sections using GSTABL software, identifying localized and global stability concerns;
- estimate the degree of seepage occurring from the lake to the downstream tailwater at normal operating conditions within the canal;
- determine global stability of the dam under normal, maximum pool, and rapid drawdown conditions.
- foundation design including a discussion of alternate solutions if applicable, allowable bearing capacity and anticipated total and differential settlements;
- Soil Site Class for "general procedure" seismic analysis in accordance with IBC requirements;
- use and treatment of in-situ material for controlled fill;
- removal or treatment of objectionable material;
- groundwater conditions and dewatering; and
- quality assurance, field-testing and observation during construction.

3.0 Field and Laboratory Work

3.1 Field Work

From the period January 30 to February 2, 2024, eight Standard Penetration Test (SPT) borings were drilled by CGC Geoservices, LLC at the approximate locations presented on the Location Plan, TL-1 & TL-2. Representative soil samples were obtained in general accordance with ASTM D 1586 methods. The boring locations were selected and established in the field by Pennoni personnel. Appendix A includes the boring logs and the Location Plan, Drawing No. TL-1 & TL-2.

Our M. Arkan, PE directed the field work; our T. Hall, EIT, provided full-time observation and logging of the test borings.

3.2 Laboratory Work

The soil samples collected during our field study were delivered to our laboratory for testing. Table 1 below summarizes the geotechnical laboratory program.

Table 1: Geotechnical Laboratory Program

TEST	ASTM NO.	NUMBER OF TESTS
Moisture Content	D 2216	5
Sieve Analysis	D 422	5
Plasticity Index (Limits)	D 4318	2
Unit Weight	D 7263	1
Unconfined Compression	D 2166	1
Organic Content, Loss on Ignition	D 2974	1

Appendix B includes the laboratory testing results and a list of testing procedures.

4.0 Subsurface Characteristics

4.1 Geology

The project site is located within the Inner Coastal Plain Physiographic Province of New Jersey, which is characterized by relatively loose sedimentary materials, relatively flat terrain, underlain by sands and gravels of Cretaceous origin (about 100,000,000 years old) with meandering rivers which drain to Raritan or the Delaware River. The Inner Coastal Plain has been eroded down to older sediments richer and finer, such as clay and silts. The topography of this area can be characterized by rolling lowlands. Available geologic data shows that the site is underlain by the Cohansey Formation, consisting of fine to coarsely grained sand, locally gravelly. The formation is interbedded with discrete beds of clay or silty clay, thin- to thick-bedded, massive to finely laminated, dark gray. Dark gray beds commonly contain carbonized wood fragments.

4.2 Subsoils

Test Borings B-3 and B-4 revealed an approximately 4 in. thick asphalt layer at the surface. A Fill layer was observed below the surface layer in boring B-3, with a thickness of 6 ft. The underlying subsoils, including the fill, have been grouped into five principal strata based on their engineering properties and our interpretation of their origin. Brief strata descriptions are presented in Table 2.

Table 2: Strata Descriptions

STRATUM	THICKNESS (FT)	DESCRIPTION
F	6.0	FILL: Brown to black fine to medium to coarse SAND, little Silt, trace fine Gravel; loose
1	4.0	Brown to gray fine to medium SAND, trace Silt; very loose to medium dense
2	5.0 – 12.0	Brown to tan to gray to black fine to medium to coarse SAND, trace Silt, with varying amounts of Clay and Organics; very loose/very soft to loose/medium stiff
3	--	Multicolored fine to medium to coarse SAND, trace Silt, with varying amounts of fine to coarse Gravel and occasional Clay pocket; loose to very dense
4	5.0 – 6.0	Dark gray SILT, little fine to medium Sand; stiff; interbedded in Stratum 3
Notes: Stratum F only encountered in B-3. Stratum 1 not encountered in B-3. All borings terminated in Stratum 3. Stratum 4 observed as interbedded in Stratum 3 in B-1A and B-2A.		

4.3 Groundwater

Observations for groundwater were made in the borings during and prior to the addition of water for the performance of drilling operations. Evidence of groundwater was encountered in the borings at depths varying between 4 and 6 ft below the ground surface (Elev. 59.5 to Elev. 64.5). These observations are for the times and locations noted and may not be indicative of seasonal or daily fluctuations. Seasonal variations on the order of several feet should be anticipated. Variations in the groundwater level are anticipated to fluctuate based on the water level of Lake George and Lake Albert.

5.0 Analysis and Recommendations

5.1 Stability Analysis

Table 3 summarizes the water levels assumed for separate cases analyzed in the stability analysis. The table includes water elevations upstream and downstream of the existing dam and anticipated head differential between them. For our seepage analysis, normal pool conditions were used.

Table 3: Estimated Water Elevations

DESIGN SCENARIO	ESTIMATED UPSTREAM HEADWATER ELEVATION (FT)	
	LAKE GEORGE	LAKE ALBERT
Normal Pool	66.6	63.7
Maximum Storage Pool	71.0	66.0
Maximum Surcharge Pool	71.5	66.5
Rapid Drawdown from Max. Storage Pool	71.0 – Before Drawdown 58.0 – After Drawdown	66.0 – Before Drawdown 55.5 – After Drawdown
Rapid Drawdown from Max. Surcharge Pool	71.5 – Before Drawdown 58.0 – After Drawdown	66.5 – Before Drawdown 55.5 – After Drawdown

Pennoni utilized Gregory Geotechnical's GEOSTASE® software to perform our slope stability analyses. GEOSTASE® is a 2-D limit equilibrium slope stability analysis software program. Our analyses are based on the topography provided in the project site drawings, soil parameters, stratigraphy depths/thicknesses, and groundwater elevations based on the borings performed as part of this study, and water surface elevations based on normal pool, maximum storage pool, and maximum surcharge (overtopping) pool of the dam. The groundwater observations encountered in the geotechnical field exploration were assumed to be made during "normal pool" conditions. The cross-sections where global stability was performed were chosen within earthen embankment areas of the dam as indicated in the Location Plan, TL-1 & TL-2.

The purpose of a slope stability analysis is to provide an estimation of the slope stability based on the existing topography and loading conditions resulting in the estimation of a minimum factor of safety against slope failure. A factor of safety greater than one (> 1.0) generally indicates stability even if only marginally so and a factor of safety less than one (< 1.0) indicates an unstable slope condition. General design practice considers a minimum "long term" factor of safety of 1.5 and a "short term" (e.g., during construction, etc.) of 1.3. For the purpose of this discussion, slope failure is categorized in two types: veneer failure or global failure. Veneer failure occurs in a relatively "thin" zone near the slope's surface as opposed to global failure which encompasses a greater percentage of the slope.

Review of the US Army Corps of Engineers Engineering and Design Manual titled, "Slope Stability: 1110-2-1902," 2003 edition indicated the following analysis conditions and recommended minimum factors of safety (FS) for the design of slopes for embankment dams:

- Normal Pool Elevation – Downstream Slope (Min. FS of 1.5)
- Maximum Storage Pool Elevation – Downstream Slope (Min. FS of 1.5)
- Maximum Surcharge Pool (Overtopping) Elevation – Downstream Slope (Min. FS of 1.4)
- Rapid Drawdown from Maximum Storage Pool Elevation – Upstream Slope (Min. FS of 1.3)

- Rapid Drawdown from Maximum Surcharge Pool (Overtopping) Elevation – Upstream Slope (Min. FS of 1.1)

The results of the analyses and approximate locations where the cross-sections were selected and modeled as part of the slope stability analysis are attached herein. A summary of the results from the analyses performed on the referenced cross sections based on the upstream water surface elevation is provided in Table 4.

Table 4: Stability Results

CONDITION	CROSS-SECTION	MINIMUM FACTOR OF SAFETY ANALYZED (FS)	MINIMUM FACTOR OF SAFETY REQUIRED (FS) *
Normal Pool – Downstream Slope	Lake George	2.3	1.5
	Lake Albert	3.7	
Maximum Storage Pool – Downstream Slope	Lake George	2.1	1.5
	Lake Albert	3.7	
Maximum Surcharge Pool (Overtopping) – Downstream Slope	Lake George	2.1	1.4
	Lake Albert	3.7	
Rapid Drawdown from Max. Storage Pool – Upstream Slope	Lake George	1.4	1.3
	Lake Albert	1.4	
Rapid Drawdown from Max. Surcharge Pool (Overtopping) – Upstream Face	Lake George	1.4	1.1
	Lake Albert	1.2	
* Minimum factor of safety required is based on recommendation provided in US Army Corps of Engineers Engineering and Design manual titled, "Slope Stability: 1110-2-1902," 2003 Edition.			

The factor of safety analyzed for all conditions was observed to meet or exceed the recommended minimum factor of safety required. Review of the failure surfaces for the analyses producing the minimum factor of safety for each condition generally indicate the failure surfaces are more global than a veneer (i.e. surface) failure. Calculations for each condition are attached to this report.

5.2 Seepage Analysis

A seepage analysis to estimate the factor of safety against piping was conducted at two cross sections through Site George (one at the southern embankment, one at the bituminous pavement auxiliary spillway) and at two cross sections through Site Albert (one at the northern embankment and one at the spillway). The approximate locations are shown on TL-1 & TL-2. The factor of safety with respect to the exit gradient at the toe of the drainage path is generally defined as the ratio of the critical gradient to the estimated exit gradient. Based on published information from the United States Department of Interior Bureau of Reclamation, a minimum factor of safety against piping of 3.0 should be considered. The cross-sections analyzed estimated a minimum factor of safety greater than 3.0 for the existing embankments and spillways. The analysis results are presented in Table 5.

Table 5: Seepage Analysis Results

CROSS SECTION ANALYZED	ANALYZED FACTOR OF SAFETY	MINIMUM FACTOR OF SAFETY
Lake George Embankment	7.7	3.0
Lake George Auxiliary Spillway	5.5	3.0
Lake Albert Embankment	4.8	3.0
Lake Albert Spillway	5.1	3.0

5.3 Foundations

Currently, proposed construction is anticipated to consist of installing sheeting on the upstream sides of both dams, rehabilitating the Lake Albert spillway, and installing a new trapezoidal spillway on the Lake Albert embankment. Factors effecting foundation construction and behavior for this project consist of very loose to loose subsurface soil to depths of 9 to 16 ft below the ground surface (Elev. 50.5 to Elev. 59.5) and the shallow groundwater table (Elev. 59.5 to Elev. 64.5).

The following are recommendations for foundation systems based on our evaluation of these factors and our experience. Typically, shallow foundations consisting of isolated spread footings and continuous strip foundations provide a practical and economical solution for support of structures. However, when subsurface conditions cannot provide adequate bearing capacity or result in undesirable settlement estimates, other foundation options are considered. Foundations considered for the proposed structure include a thickened reinforced concrete mat at the base of proposed structures and grouted helical piers, for support of structures and hydrostatic uplift resistance. The subgrades of all foundations exposed to freezing temperatures during construction and/or the life of the structure should be established at least 3 ft below the adjacent exposed grades or otherwise protected against frost action.

5.3.1. THICKENED REINFORCED CONCRETE SLAB MAT FOUNDATION

Potential structures can be founded on a 3 ft thickened reinforced concrete mat founded on medium dense to dense Stratum 3 soils. We estimate a maximum net allowable bearing capacity of approximately 3,000 psf for foundations bearing within the Stratum 3 medium dense to dense Sand, embedded 10 to 16 feet below existing grade. We recommend that the foundation be designed to counterbalance the uplift pressure of the groundwater and that proposed structures be secured to the foundation. Based on published information from the United States Army Corps of Engineers, a factor of safety against uplift ranges on the order of 1.3 to 1.5 for design considerations.

Where bearing directly on Stratum 3 soils is not feasible, deep foundations (discussed below) or ground improvement such as performing a soil exchange will be necessary. With a soil exchange, we recommend performing the exchange to the medium dense to dense Stratum 3 soils. The width of the excavation at the bottom should be equal to the footing width plus 1 ft for every 1 ft of depth excavated below the footing bottom. All exposed subgrades should be thoroughly densified using vibratory compaction equipment. The excavated fill should be replaced with suitable load-bearing structural fill (modified NJDOT I-5 or recycled concrete of similar gradation) placed in layers and compacted as outlined in ***Section 5.8 Earthwork***. Prior to placement of new fills for the soil exchange, the exposed foundation subgrades should be densified and checked by a qualified representative of the geotechnical engineer. Soft/loose or otherwise unstable areas should be further undercut as directed in the field. Assuming this approach is followed, spread footings can be designed for a net allowable soil bearing capacity up to 3,000 psf. **It should be noted that if this option is chosen, the SOE design must account for the additional depth to be excavated.**

5.3.2. GROUTED HELICAL PILE FOUNDATION SYSTEM

Due to the soft/loose soils disclosed by our exploration that extend to depths on the order of 16 ft below the embankment ground surface (Elev. 50.5 to Elev. 59.5), a helical pile system bearing in the medium dense Stratum 3 sand is recommended for this project. This option is considered feasible to resist uplift pressures induced on potential structures by groundwater pressure. We recommend that the foundations be designed to counterbalance the uplift pressure of the groundwater and that the proposed structures be secured to the pile caps.

The subgrade of all pile caps exposed to freezing temperatures during construction and/or the life of the structure, should be established at least 36 in. below adjacent exposed grades or otherwise protected against frost action.

General

The grouted helical pile is a deep foundation system that employs the use of a grout column that encases the shaft of a standard helical pile anchor to utilize both skin friction resistance and end bearing resistance to develop the capacity of the pile. Grouted helical piles are installed to depths that will provide adequate individual allowable support based on the proposed loading. The drilled helical piles consist of a steel shaft (lead section) to which one or more helices are attached. Helical piles are rotated into the competent soils while an axial load is applied to the shaft. After the lead section is installed, displacement plates with subsequent extensions are installed. Flowable grout is pumped during the installation procedure to encase the shaft as the extensions are added until a termination depth is achieved. The design bearing capacity is confirmed by measuring the torque applied to the shaft by the installation equipment as well as static load testing. The helical piles are embedded in the pile caps, or attached to foundations with brackets or other structural elements, and the structure’s loads are transferred to competent soils at an appropriate depth through the helical piles. Helical piles typically develop little lateral resistance; lateral resistance can be improved by providing battered piles, as necessary up to 30 degrees from vertical. This drilling technique also offers the advantage of resulting in low noise and vibration, and fewer spoils as compared to other deep foundation options. Grouted helical piles provide an increase in the buckling resistance of the pile (as compared to non-grouted helical piles), as the pile slenderness ratio is inherently large due to the length of the pile and the relatively small cross-sectional area of the pile. The encasement of the steel shaft with grout also provides a benefit against corrosive resistance from the soil.

This type of foundation system is typically bid as a performance specification as opposed to a prescriptive specification. For a performance specification, the desired capacity and other foundation requirements are prescribed. The contractor and their engineer then design the foundation to meet the desired requirements. As helical piles typically involve proprietary systems and the contractors who install these often employ engineers, this type of specifications can lead to more economical foundation system. For these reasons, this foundation system is considered a practical solution for the proposed structure.

Design

Based on our analyses, an 8 in./10 in./12 in. triple-helix helical pile system with a minimum 6.0 in. diameter grout column installed to a depth of approximately 26 to 28 ft below the existing grades and embedded approximately 10 ft in the medium dense Stratum 3, will develop allowable capacities in compression and tension of 20 and 15 kips per unit, respectively, with a more than adequate factor of safety against failure (FS = 2.0 in compression and 3.0 in tension). We recommend that each helical pile unit consist of a 2.0 in. solid steel square shaft (SS200), with an 8 in. diameter bottom plate (helix) at the pile tip, a 10 in. diameter helix located at least 24 inches above the lower one, and a 12 in. diameter helix located at least 30 inches above the 10 in. diameter helix, with each helix welded to the shaft. In addition to the allowable compression and tension (uplift) capacities, an allowable lateral load capacity of 2 kips per pile may be used in design corresponding to a maximum horizontal displacement at the pile top of 0.5 in. Batter piles, if required, should be designed for a minimum batter, but in no case steeper than 1 horizontal to 5 vertical. The helical piles should be spaced a minimum of 3 times the largest helix diameter to prevent reduction in capacity due to group effects.

We recommend that the contract require the installer to perform a minimum of one static load test in accordance with ASTM D-1143 to verify the as-constructed pile capacity. The static load test is to be performed on a non-production pile to confirm the capacity. The installer should produce a submittal that is signed and sealed by a registered professional engineer in the State of New Jersey, which documents that their proposed helical pile system meets the project specifications.

We recommend that the installation of the helical piles be carried out in the full-time presence of a qualified representative of the Geotechnical Engineer.

5.3.3. TIMBER PILES

Based on our analyses, CCA treated timber piles can be used to support the proposed construction. The timber piles should penetrate through the loose Strata 1 and 2 and develop their vertical compression capacity by a combination of skin friction and end bearing in the medium dense sands of Stratum 3. The timber piles should conform to ASTM 25-99 and AWPA C3-03 Specifications and should have minimum tip and butt diameters of 8 and 12 in., respectively. Our static analyses indicate that CCA treated timber piles penetrating 25 to 30 ft below the existing ground surface will develop an allowable working load capacity of 13 tons per pile.

In addition to the allowable vertical compression capacity, an allowable tension (uplift) capacity of 4 tons per pile and an allowable lateral load capacity of 1 ton per pile may be used in design. Batter piles, if required, should be designed for a minimum batter, but in no case steeper than 1 horizontal to 4 vertical.

All piles should be driven within the following maximum tolerances:

- Location: 4 in. from the location indicated after initial driving, and 6 inches after pile driving is completed,
- Plumb: Maintain 1 in. in 10 ft from the vertical, or a maximum of 4 inches measured when the pile is above ground in leads,
- Batter Angle: Maximum 1 in. in 10 ft from required angle, measured when pile is above ground in leads.

After the piles penetrate through the loose Strata 1 and 2 soils, they should be driven to develop the recommended allowable capacity of 13 tons per pile. The installation of timber piles to develop the recommended pile capacities should be done in accordance with a suitable dynamic formula, such as the Wave Equation, Modified Engineering News Formula, etc. Approved equipment, including a hammer having a rated energy of at least 15,000 ft lbs, should be employed to drive the piles. Prior to the installation of the production piles, at least six “drive test piles” or “indicator piles” should be installed over the project site, at locations to be selected by the Geotechnical Engineer to determine the pile driving characteristics and the required pile lengths more accurately.

We recommend dynamic load testing (ASTM D4945) for this project in lieu of a static load test. We recommend that piles be instrumented and monitored by a Pile Driving Analyzer (PDA) during the installation of “indicator piles”. The indicator piles should be monitored continuously by PDA, during initial drive and restrike. The indicator test piles can be at product pile locations.

All pile installation, including the drive test piles, should be carried out in the full-time presence of a qualified representative of the Geotechnical Engineer who should evaluate and correlate the driving data and depth of penetration of each pile with the results of the drive test piles, our static analyses, and the boring log data. The Geotechnical Engineer's representative should ensure that the tip of each pile is located satisfactorily in the Stratum 3 soils, and that the required driving resistance of each pile is attained.

5.4 Settlement

Settlement of a soil mass is a function of the characteristics of the supporting soils (type of soil, void ratio, pre-consolidation, etc.), the thickness of the layer(s), and the stresses imposed on the soils by an applied load (fill, shallow foundations, floor slab, etc.). The stresses affecting subsoils generally decrease with increasing depth and are variable based on the magnitude and area of applied loading.

Provided that the recommendations discussed herein are followed, it is expected that total and differential settlements will be less than 1 in. and ½ in., respectively. With the use of the pile foundations as recommended, it is expected that total and differential settlements will be negligible. Detrimental post-construction settlements are not expected if the recommendations presented herein are followed.

5.5 Seismic Site Classification

The borings disclosed subsurface conditions generally described according to the 2018 International Building Code (IBC), New Jersey Edition, section 1613.3.2 referencing ASCE 7, Chapter 20 as having a soil-profile corresponding to Site Class D – Stiff soil profile. Site class determination is based on the properties of the upper 100 ft of the ground surface. The borings performed herein were advanced to a maximum depth of 60 ft. Values beyond 60 ft were estimated based on our local experience in this area.

5.6 Lateral Earth Pressure Parameters

The soil parameters presented in Table 6 can be used to estimate lateral earth pressures on below grade structures and temporary shoring. If the top of the structure is restrained from movement, thereby preventing the mobilization of active soil pressures, the structure should be designed using the at-rest pressure coefficient, k_o.

The earth pressure coefficients are based on the assumption of vertical walls, horizontal backfill, no surcharges, no wall friction, and a safety factor of 1.0. Hydrostatic pressures associated with seepage must also be considered in the design. Depending on the type of retention system selected, active or at-rest coefficients should be used in the earth retention system design.

Table 6: Lateral Earth Pressure Parameters

PARAMETER	FILL / STRATUM 1	STRATUM 2	STRATUM 3	ENGINEERED FILL (NJDOT I-5, or similar)
Unit Weight, pcf	120	115	130	135
Buoyant Unit Weight, pcf	58	53	68	73
Angle of Internal Friction, degrees	28	26	34	38
Cohesion, psf	0	0	0	0
Friction Factor (concrete)	0.34	0.31	0.42	0.47
k_a	0.36	0.39	0.28	0.24
k_o	0.53	0.56	0.44	0.38
k_p	2.77	2.56	3.54	4.20

Excavations during subgrade preparation and foundation construction should be in accordance with Occupational Safety and Health Administration (OSHA) regulations. If site restrictions prevent excavations from being adequately sloped, the use of temporary shoring should be expected.

If the contractor is responsible for the design of temporary or permanent retaining structures, then the contract documents should clearly require that a competent registered engineer perform the design and that satisfactory earth support is solely the contractor’s responsibility. Furthermore, the contract documents should require the contractor to notify the engineer immediately if differing or unforeseen subsurface conditions are encountered during construction.

5.7 Groundwater and Surface Water Management

The groundwater observations made in the borings suggest that free-standing groundwater is anticipated in excavations for the proposed construction. We recommend that the groundwater be lowered to a minimum of 5 feet below excavation depths during construction using a well point system or temporary shoring and high-capacity pumps to reduce the potential for the groundwater table to degrade the foundation subgrades. The contractor shall obtain the required State and Local permits associated with groundwater withdrawal/well points and discharge the groundwater in accordance with the applicable permit conditions. The temporary lowering of the groundwater table should be performed during and after foundation construction and fill placement.

The foundation excavations should not be used as a detention basin or sump. During construction, surface runoff should be prevented from entering the excavations by creating soil berms or diversion swales along the perimeter if the excavation is expected to be open for a long period of time. Where ponding does occur, the water should be pumped immediately, and grades should then be established to prevent further ponding.

The shallow subsurface material is considered susceptible to damage from moisture and construction traffic. Therefore, precipitation and other water should not be permitted from accumulating on the exposed subgrade and construction traffic should be minimized over exposed subgrade.

5.8 Earthwork

Prior to construction, all topsoil, vegetation, concrete, and asphalt pavement must be removed from within the proposed area of construction. Any existing utilities located within the proposed construction area should be abandoned and relocated outside the proposed footprint. Any existing utility line abandoned in-place should be grouted, or the line should be removed and the trench appropriately backfilled.

Exposed subgrades should be thoroughly proof-rolled in the presence of a representative from Pennoni. Where space is limited subgrade soils should be manually probed in an attempt to disclose unstable surface areas. Any unstable surface areas (soft, yielding, etc.) found should be stabilized by excavating and replacing those soils with suitable soil that is adequately compacted. This can be accomplished by properly adjusting the moisture content of the subgrade soils and compacting them, or by other methods (placing a geotextile and stone layer, etc. or soil exchange).

Our experience indicates that the clean/inert and granular portions of the near surface soils can be reused for earthwork construction, provided it is free of deleterious material (i.e. organics, ash and cinders, etc.), debris larger than 3 in. in its greatest dimension, and there are no environmental concerns associated with the soils. Laboratory testing indicates that the near surface soils consist of up to 20% of fine-grained (silts/clays) material, with a moisture content of up to 59%. These types of soils are sensitive to moisture and may require wetting or drying prior to compaction. Additionally, drying "wet" soil is difficult during wet periods and during lower temperatures. Therefore, depending on the season that the earthwork operations are taking place, adjusting the moisture content of these on-site soils before use in any compacted fills and/or subgrade preparation may be required. Provisions for importing structural fill should be included in the contract documents. Proper compaction equipment and placing soil in thinner layers should be considered when preparing earthwork schedules.

Imported fill should be selected from suitable borrow sources and be approved by the Geotechnical Engineer well in advance of fill construction. Granular fill should consist of well-graded material meeting the classification characteristics of GW, GP, GM, GC, SW, SP, SM, or a combination of those listed with not more than 20 percent passing the No. 200 sieve and have a plasticity index not greater than 8 percent, conforming to an overall gradation similar to NJDOT I-5. Crushed aggregate such as AASHTO #57 stone should be considered for fill placement below groundwater levels. Other gradations can be considered based on laboratory testing and at the discretion of the Geotechnical Engineer.

Fine grained and granular fills should be placed in layers not exceeding 8 to 10 in. and 10 to 12 in. loose thickness, respectively. This criterion might be adjusted by the geotechnical engineer in the field depending on the conditions present at the time of construction, on the compaction equipment used, and on the fill materials selected. Table 7 below presents the compaction requirements.

Table 7: Compaction Requirements

FILLS SUPPORTING	STANDARD PROCTOR (ASTM D698)	MODIFIED PROCTOR (ASTM D1557)
Foundations	98%	95%
Pavements	95%	93%

Fills should be compacted to ASTM D 698 percentages of the laboratory determined maximum dry density when small, hand-operated compaction equipment is used, and to ASTM D 1557 percentages of the laboratory determined maximum dry density, when self-propelled, heavy-duty construction equipment is used. Fills should extend a minimum of 5 ft beyond the exterior edge of a loaded area and have side slopes not steeper than 2 horizontal to 1 vertical.

Specifications should indicate that the percentage of maximum dry density attained in the field is not the only criteria to be used for assessing fill compaction. Observation of the behavior of the fill under the loads of construction equipment should also be used. If the test results indicate that the percentage of compaction is being achieved, but the soil mass is moving under the equipment, placement of additional fill should not be continued until the movement is stabilized. Otherwise, settlement of the fill may occur.

5.9 Existing Utilities

As mentioned above, any existing utilities located within the proposed construction areas should be abandoned and relocated outside the proposed structure footprint. Any existing utility line abandoned in-place should be grouted or the line should be removed from the trench and appropriately backfilled.

6.0 Recommendations for Further Geotechnical Services

Our experience on numerous construction projects is that the interests of the project team are best served by retaining the Geotechnical Engineer to provide construction observations during earthwork and foundation construction operations. To determine if soils, other materials, and ground water conditions encountered during construction are similar to those encountered in the borings, and that they have comparable engineering properties or influences on the design of the structure, we recommend that Pennoni should provide field observation services during excavation; preparation of foundation subgrades; and installation/construction of foundations. Pennoni's Geotechnical Technology should review specifications for earthwork and foundation design/construction when they are prepared.

7.0 Limitations

This work has been done in accordance with our authorized scope of work and in accordance with generally accepted professional practice in the fields of geotechnical and foundation engineering. This warranty is in lieu of all other warranties either expressed or implied. Our conclusions and recommendations are based on the data revealed by this exploration. We are not responsible for any conclusions or opinions drawn from the data included herein, other than those specifically stated, nor are the recommendations presented in this report intended for direct use as construction specifications. This report is intended for use with regard to the specific project described herein; any changes in loads, structures, or locations should be brought to our attention so that we may determine how they may affect our conclusions. An attempt has been made to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during construction. If this should occur, or if additional or contradictory data are revealed in the future, we should be notified so that modifications to this report can be made, if necessary. If we do not review relevant construction documents and witness the relevant construction operations, then we cannot be responsible for any problems that may result from misinterpretation or misunderstanding of this report or failure to comply with our recommendations. our virtual executive summary should be used for informational purposes only and should not be used for construction related purposes.

Appendix A:
Field Data

LEGEND

B -

TEST BORING LOCATION AND IDENTIFYING NUMBER

CROSS-SECTION ANALYZED

NORTH

Pennoni

PENNONI ASSOCIATES INC.
515 Grove Street, Suite 1B
Haddon Heights, NJ 08035
T 856.547.0505 **F** 856.547.9174

DAM SAFETY ENGINEERING

Lake George // Lake Albert
Collings Lakes, NJ

LOCATION PLAN

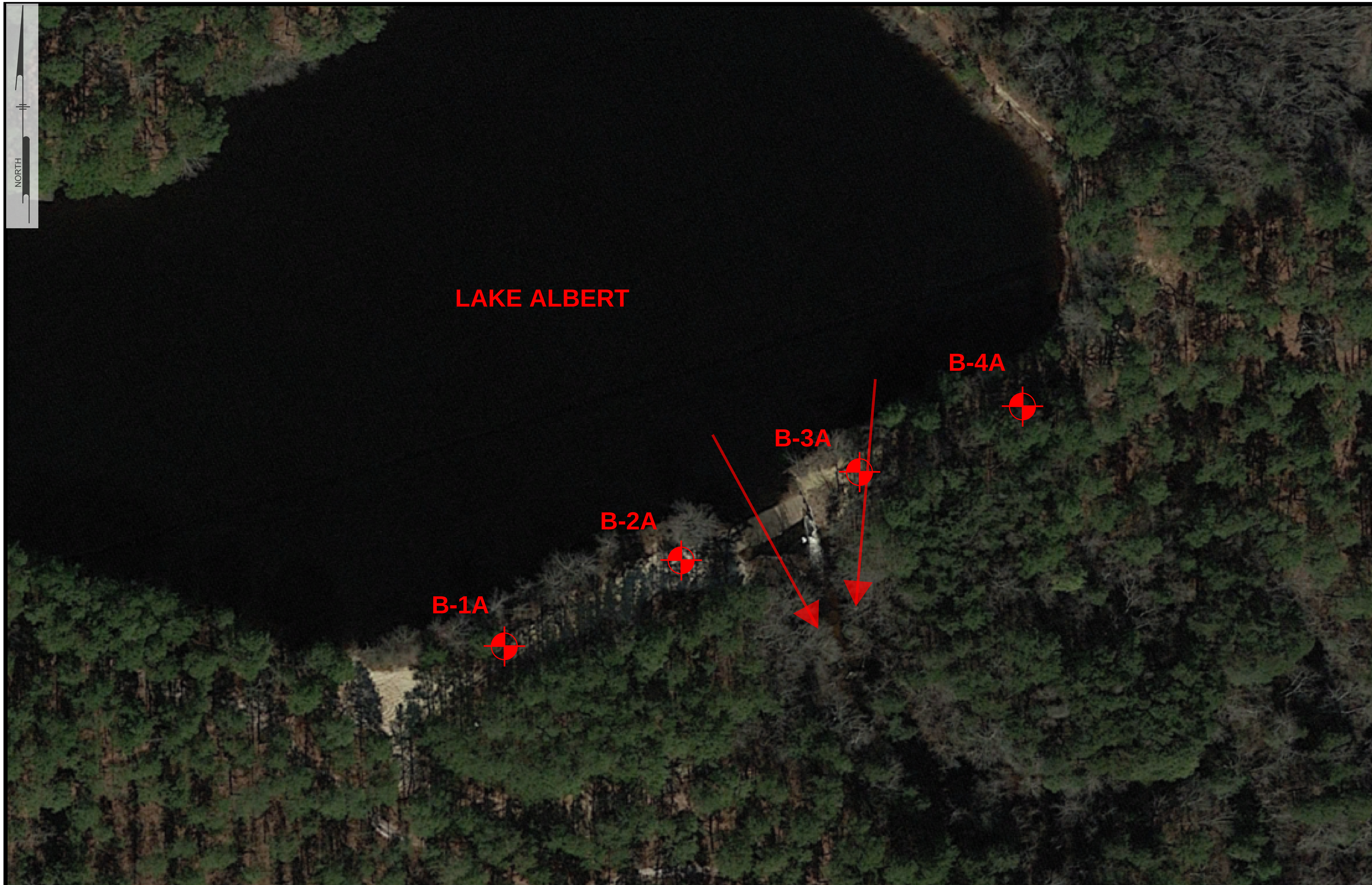
Collings Lakes Civic Association
PO Box 475
Williamstown, NJ 08094

ALL DOCUMENTS PREPARED BY PENNONI ASSOCIATES ARE INSTRUMENTS OF SERVICE IN RESPECT OF THE PROJECT. THEY ARE NOT INTENDED OR REPRESENTED TO BE SUITABLE FOR REUSE BY OWNER OR OTHERS ON THE EXTENSIONS OF THE PROJECT OR ON ANY OTHER PROJECT. ANY REUSE WITHOUT WRITTEN VERIFICATION OR ADAPTATION BY PENNONI ASSOCIATES FOR THE SPECIFIC PURPOSE INTENDED WILL BE AT OWNERS SOLE RISK AND WITHOUT LIABILITY OR LEGAL EXPOSURE TO PENNONI ASSOCIATES; AND OWNER SHALL INDEMNIFY AND HOLD HARMLESS PENNONI ASSOCIATES FROM ALL CLAIMS, DAMAGES, LOSSES AND EXPENSES ARISING OUT OF OR RESULTING THEREFROM.

PROJECT	CLCAX23002
DATE	10 JUNE 2024
DRAWING SCALE	NTS
DRAWN BY	TH
APPROVED BY	MA

TL - 1

SHEET 1 OF # 2



LEGEND

B -
TEST BORING LOCATION
AND IDENTIFYING NUMBER

CROSS-SECTION ANALYZED

PENNONI ASSOCIATES INC.
515 Grove Street, Suite 1B
Haddon Heights, NJ 08035
T 856.547.0505 **F** 856.547.9174

DAM SAFETY ENGINEERING

Lake George // Lake Albert
Collings Lakes, NJ

LOCATION PLAN

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PROJECT	CLCAX23002
DATE	10 JUNE 2024
DRAWING SCALE	NTS
DRAWN BY	TH
APPROVED BY	MA

TL - 2

SHEET 2 OF # 2

TEST BORING LOG

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 1/30/24 **COMPLETED** 1/30/24
GROUND ELEVATION 69.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 6.0' / Elev 63.5'
DRILLER / HELPER Eugene Blemings / Eric Blemings
AT END OF DRILLING 6.0' / Elev 63.5'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
AFTER DRILLING 6.0' / Elev 63.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	DESCRIPTION (Depth / Elev.)	REMARKS
0	S-1	18	1-4-5-5		1	Orange to brown F/M SAND, trace Silt	
	S-2	18	3-3-3-6			Gray F/M SAND, trace Silt	
						4.0 — 65.5	
5	S-3	16	2-2-2-2		2	Gray F/M SAND, trace Silt	Wet at bottom
	S-4	20	2-1-1-2			Dark brown F/M/C SAND, trace fine Gravel	Trace organic odor
10	S-5	24	2-5-5-5			Brown F/M/C SAND, trace Organics	
						10.0 — 59.5	
	S-6	24	6-12-16-15		3	Brown to tan F/M SAND, trace Silt	
	S-7	16	5-6-7-8				
15	S-8	12	6-10-12-10				
20	S-9	18	9-11-19-20			Tan to gray F/M/C SAND, some fine Gravel	
25	S-10	18	5-6-6-8			Tan to beige F/M/C SAND, trace Silt	
30	S-11	20	3-3-3-8			White to light gray F/M SAND, trace Silt	
35	S-12	24	3-7-24-35			Brown to orange F/M/C SAND, trace Silt	
						39.0 — 30.5	

NOTES:

(Continued Next Page)

TEST BORING LOG

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	DESCRIPTION (Depth / Elev.)	REMARKS
40	S-13	24	6-24-27-35			Brown F/M SAND, trace Silt	
45	S-14	3	12-30-48-50/5			Brown to tan F/M/C SAND, trace Silt	
50	S-15	8	14-22-20-17		3		
55	S-16	24	41-27-39-46			Reddish brown F/M/C SAND, trace Silt	
60	S-17	24	24-38-42-40			60.0 / 9.5	

Borehole terminated at 60.0 feet.

NOTES:

TEST BORING LOG

TEST BORING B-2

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 1/30/24 **COMPLETED** 1/30/24
GROUND ELEVATION 70.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 6.0' / Elev 64.5'
DRILLER / HELPER Eugene Blemings / Eric Blemings
AT END OF DRILLING 6.0' / Elev 64.5'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
AFTER DRILLING 6.0' / Elev 64.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0	S-1	10	2-3-4-4	1		Brown F/M SAND, trace Silt		
	S-2	15	4-3-4-3	1		Brown to gray F/M SAND, trace Silt		
					4.0		66.5	
5	S-3	18	4-2-1-1	2		Brown to gray F/M SAND, trace Silt		
	S-4	24	WOH/12-1/12	2				Wet
	S-5	24	WOR/24	2				
10	S-6	24	WOR/24	2				
	S-7	24	WOR/24	2				
					15.0		55.5	
15	S-8	20	1-2-5-9	3		Brown to dark brown to gray F/M SAND, trace Silt		Trace organic odor
	S-9	24	5-10-14-16	3				
	S-10	24	8-5-8-7	3		Brownish gray F/M SAND, trace F/C Gravel		
20				3				
25	S-11	3	7-8-10-11	3		Brownish gray F/M/C SAND, little fine Gravel		
30	S-12	20	8-7-12-16	3		Brown F/M/C SAND, trace fine Gravel		
35	S-13	18	2-8-27-48	3		Brown to orange F/M/C SAND, some white to gray Clay (last 6")		
	S-14	24	8-12-16-24	3		Brown F/M SAND, trace Silt		
40					40.0		30.5	

Borehole terminated at 40.0 feet.

NOTES:

Pennoni

TEST BORING LOG

TEST BORING B-3

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 1/31/24 **COMPLETED** 1/31/24
GROUND ELEVATION 68.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 6.0' / Elev 62.5'
DRILLER / HELPER Eugene Blemings / Eric Blemings
AT END OF DRILLING 4.0' / Elev 64.5'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
AFTER DRILLING 4.0' / Elev 64.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0				A	0.3	4" ASPHALT	68.2	
	S-1	14	2-2-1	F		FILL: Brown F/M/C SAND, little Silt, trace fine Gravel		
	S-2	18	2-3-3-3			FILL: Black to brown F/M/C SAND, little Silt		
5	S-3	0	2-4-5-4		6.0		62.5	
	S-4	16	3-1-1-1	2		Brown to tan F/M SAND, trace Silt		Wet
10	S-5	20	WOH/24					
	S-6	14	WOH/24			Dark gray to brown F/M/C SAND, little Silt (NP), trace fine Gravel		Trace organic odor
	S-7	20	2-4-7-6		14.0	Dark gray to dark brown F/M SAND, trace Silt, trace Organics	54.5	Organic odor
15	S-8	18	14-16-16-22	3		Gray F/M/C SAND, trace Silt		
20	S-9	10	29-50/4			Dark brown M/F/C SAND, trace fine Gravel		
25	S-10	20	6-11-16-19			Brown F/M SAND, trace Silt		
30	S-11	24	10-32-36-29			Brown to reddish brown F/M/C SAND, trace Silt		
35	S-12	24	8-15-19-21			Brown F/M/C SAND, trace Silt		
40	S-13	24	12-18-17-22		40.0		28.5	

Borehole terminated at 40.0 feet.

NOTES:

TEST BORING LOG

TEST BORING B-4

Pennoni

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 1/31/24 **COMPLETED** 1/31/24
GROUND ELEVATION 68.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 4.0' / Elev 64.5'
DRILLER / HELPER Eugene Blemings / Eric Blemings
AT END OF DRILLING 4.0' / Elev 64.5'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
AFTER DRILLING 4.0' / Elev 64.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0				A	0.3	4" ASPHALT	68.2	
	S-1	16	5-5-7	1		Orange to tan brown F/M SAND, trace Silt		
	S-2	20	5-4-3-3			Light gray F/M SAND, trace Silt		Moist at bottom
					4.0		64.5	
5	S-3	10	3-2-1-1	2		Light gray F/M SAND, trace Silt		Wet
	S-4	20	1-1-1-1			Brownish gray F/M SAND, trace Silt		
10	S-5	22	2-4-7-9					
					11.0		57.5	
	S-6	24	3-3-5-8	3		Brown F/M SAND, trace Silt		
	S-7	24	11-12-15-17					
15	S-8	12	4-8-10-12					
	S-9	18	5-11-15-16					
20	S-10	24	6-11-14-22			Brown F/M/C SAND, trace fine Gravel		
25	S-11	16	17-45-50/4			Dark brown to multicolored M/C/F SAND, little F/C Gravel		
30	S-12	22	26-24-29-35			Grayish brown F/M/C SAND, little F/C Gravel		
35	S-13	24	17-19-25-23			Brown F/M/C SAND, trace Silt		
40	S-14	24	14-20-21-24		40.0		28.5	

Borehole terminated at 40.0 feet.

NOTES:

TEST BORING LOG

TEST BORING B-1A

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 1/30/24 **COMPLETED** 1/30/24
GROUND ELEVATION 65.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
▽ **DURING DRILLING** 6.0' / Elev 59.5'
DRILLER / HELPER Eugene Blemings / Eric Blemings
▼ **AT END OF DRILLING** 6.0' / Elev 59.5'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
▼ **AFTER DRILLING** 6.0' / Elev 59.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0	S-1	18	1-1-3-4	1		Tan to brown F/M SAND, trace fine Gravel		
	S-2	15	3-6-5-5		4.0		61.5	
5	S-3	12	4-2-2-1	2		Gray F/M SAND, trace Silt		
	S-4	14	1-1-2-3			Grayish brown F/M SAND, trace Silt		Wet
	S-5	20	3-4-5-8		9.0		56.5	
10	S-6	18	8-8-8-9	3		Brown to gray F/M/C SAND, trace Silt		
	S-7	20	5-6-8-10					
15	S-8	12	5-4-4-5					
20	S-9	18	6-5-5-7					
25	S-10	16	5-9-10-11			Multicolored C/M/F SAND, little fine Gravel		
					29.0		36.5	
30	S-11	20	4-5-4-5	4		Dark gray SILT (elastic), little F/M Sand		
					34.0		31.5	
35	S-12	18	5-6-6-8	3		Brown F/M/C SAND and dark gray CLAY		
40	S-13	20	8-18-25-20		40.0	Brown F/M/C SAND, trace Silt	25.5	

Borehole terminated at 40.0 feet.

NOTES:

TEST BORING LOG

TEST BORING B-2A

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 1/30/24 **COMPLETED** 1/30/24
GROUND ELEVATION 65.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 6.0' / Elev 59.5'
DRILLER / HELPER Eugene Blemings / Eric Blemings
AT END OF DRILLING 6.0' / Elev 59.5'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
AFTER DRILLING 6.0' / Elev 59.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0	S-1	20	1-2-4-6	1		Brown to orange F/M SAND, trace Silt		
	S-2	18	5-7-6-6	1	4.0	Light to dark gray F/M SAND, trace Silt	61.5	
5	S-3	8	1-2-2-1	2		Brown F/M SAND, trace Silt		
	S-4	16	1-2-2-2	2		Brown F/M/C SAND, trace Silt		Wet
10	S-5	24	2-2-6-7	2	10.0		55.5	
	S-6	24	5-5-7-8	3		Brown to tan to gray F/M SAND, trace Silt		
	S-7	20	6-8-10-11	3				
15	S-8	16	4-4-3-4	3				
	S-9	15	3-3-7-8	3				
20	S-10	18	7-7-8-9	3				
25	S-11	20	6-4-5-8	3	29.0	Multicolored M/C/F SAND, some C/F Gravel	36.5	
30	S-12	18	5-4-6-5	4	35.0	Dark gray SILT (elastic), little F/M Sand	30.5	
35	S-13	20	4-7-12-18	3		Orange to reddish brown F/M/C SAND, trace Silt		
40	S-14	18	4-12-21-27	3	40.0		25.5	

Borehole terminated at 40.0 feet.

NOTES:

TEST BORING LOG

TEST BORING B-3A

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 2/2/24 **COMPLETED** 2/2/24
GROUND ELEVATION 65.5' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
DRILLING METHOD Hollow Stem Auger
DRILLER / HELPER Jay Blemings / Adrian Martinez
LOGGED BY T. Hall **CHECKED BY** M. Arkan

WATER ENCOUNTERED:
- ∇ **DURING DRILLING** 4.0' / Elev 61.5'
- ▼ **AT END OF DRILLING** 4.0' / Elev 61.5'
- ▼ **AFTER DRILLING** 4.0' / Elev 61.5'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	Depth	DESCRIPTION	Elev.	REMARKS
0	S-1	16	1-1-2-1		1		Brown to tan to gray F/M SAND, trace Silt		
	S-2	15	2-3-3-3				Brown F/M SAND, trace Silt		Damp
						4.0		61.5	
5	S-3	18	2-2-1-1		2		Orange to tannish brown F/M SAND, some white Clay		wet
	S-4	18	1-1-1-2						
10	S-5	15	1/12-1/12						
	S-6	12	WOH/24						
	S-7	12	1-1-1-2						
15	S-8	16	1-2-3-5			15.0	Black ORGANIC CLAYEY SILT, some fine Sand	50.5	
					3		Gray F/M/C SAND, little Silt, trace fine Gravel		
20	S-9	14	8-6-7-11				Gray F/M/C SAND, trace Silt		
25	S-10	18	11-17-19-21				Grayish brown to dark brown M/C/F SAND, some F/C Gravel		Trace organic odor
30	S-11	10	17-30-45-47				Reddish brown F/M/C SAND, trace Silt		
35	S-12	20	12-16-27-24				Tannish brown F/M SAND, trace Silt		
40	S-13	18	15-24-33-46			40.0		25.5	

Borehole terminated at 40.0 feet.

NOTES:

TEST BORING LOG

Pennoni

CLIENT Collings Lakes Civic Association
PROJECT NAME Dam Safety Engineering Services
PROJECT NUMBER CLCAX23002
PROJECT LOCATION Lake George/Albert, Williamstown, NJ
DATE STARTED 2/2/24 **COMPLETED** 2/2/24
GROUND ELEVATION 66.0' NAVD 1988
DRILLING CONTRACTOR CGC Geoservices, LLC
WATER ENCOUNTERED:
DRILLING METHOD Hollow Stem Auger
DURING DRILLING 4.0' / Elev 62.0'
DRILLER / HELPER Jay Blemings / Adrian Martinez
AT END OF DRILLING 4.0' / Elev 62.0'
LOGGED BY T. Hall **CHECKED BY** M. Arkan
AFTER DRILLING 4.0' / Elev 62.0'

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	DESCRIPTION (Depth / Elev.)	REMARKS
0	S-1	20	1-1-4-4		1	Brown to tan to gray F/M SAND, trace Silt	
	S-2	20	5-4-5-5			Orangish brown F/M SAND, trace Silt	
						4.0 / 62.0	
5	S-3	24	3-3-3-3		2	Orangish brown F/M SAND, trace Clay	Wet
	S-4	22	2-2-6-6			Brown F/M SAND, trace Silt	
10	S-5	24	4-4-3-2				
	S-6	12	1-1/18				Trace organic odor
	S-7	4	WOH/24				
15	S-8	2	1/24			Gray F/M SAND, trace Organics	
						16.0 / 50.0	
	S-9	18	4-3-4-4		3	Dark brown to multicolored M/C/F SAND, little F/C Gravel	
20	S-10	20	4-9-13-13				
25	S-11	24	10-15-15-15			Gray to multicolored C/M/F SAND, some F/C Gravel	
30	S-12	18	12-19-17-24			Brown F/M/C SAND, trace Silt	
35	S-13	18	9-14-18-20				
						39.0 / 27.0	

NOTES:

(Continued Next Page)

TEST BORING LOG

CLIENT Collings Lakes Civic Association

PROJECT NAME Dam Safety Engineering Services

PROJECT NUMBER CLCAX23002

PROJECT LOCATION Lake George/Albert, Williamstown, NJ

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in.)	BLOW COUNTS	GRAPHIC LOG	STRATA	DESCRIPTION (Depth / Elev.)	REMARKS
40	S-14	24	22-34-41-40			Tan to brown F/M SAND, trace Silt	
45	S-15	18	10-24-32-37				
50	S-16	20	12-18-26-34		3	Brown F/M/C SAND, trace Silt	
55	S-17	24	32-40-46-48				
60	S-18	24	29-37-41-39			60.0 / 6.0	

Borehole terminated at 60.0 feet.

NOTES:

TEST BORING/TEST PIT/AUGER PROBE LOG KEY SHEET

COLUMN	DESCRIPTION
Depth	Depth in feet below ground surface
Description	Description of sample including color, texture, and classification of subsurface material as applicable. Estimated depths to bottom of strata as interpolated from the boring are also shown.
Stratum	Strata numbers as assigned by the geotechnical engineer
Sample No.	Split barrel sample and sample number (S-x) Undisturbed Tube sample and sample number (U-x) Rock core run and core number (R-x) NR indicates no recovery
Blow Counts	For soils sample (ASTM D 1586): indicates number of blows obtained for each 6 inches penetration of the standard split-barrel sampler. For rock coring (ASTM D 2113): indicates percent recovery (REC) per run and rock quality designation (RQD). RQD is the sum of rock pieces that are 4 inches or longer in length in one core run divided by the total core run.
Recovery	For soil samples indicates the length of recovery in the sample spoon
Remarks	Special conditions or test data as noted during drilling

Ground Water: Free water level as shown ( )*; * Free water level as noted may not be indicative of daily, seasonal, or long term fluctuations.

DESCRIPTIVE TERMS

RELATIVE PROPORTIONS

Descriptive Term	Symbol	Estimated Percentages
Trace	tr	1 to 10
Little	l	10 to 20
Some	sm	20 to 35
And	and	35 to 50

GRADATION OF COARSE GRAINED COMPONENTS

Soil Component	Size Range	Particle Size Maximum	Minimum
Boulders		-	12"
Cobbles		12"	3"
Gravel	Coarse	3"	¾"
	Fine	¾"	#4 Sieve
Sand	Coarse	#4 Sieve	#10 Sieve
	Medium	#10 Sieve	#40 Sieve
	Fine	#40 Sieve	#200 Sieve
Silt		#200 Sieve	.005 mm
Clay		.005 mm	-

COMPOSITION OF COARSE-GRAINED COMPONENTS

Gradation Designation	Symbol	Defining Proportions
Coarse to Fine	CF	All fractions greater than 10% of the component
Coarse to Medium	CM	Less than 10% Fine
Medium to Fine	MF	Less than 10% Coarse
Coarse	C	Less than 10% Fine and Medium
Medium	M	Less than 10% Coarse and Fine
Fine	F	Less than 10% Coarse and Medium

Appendix B:
Laboratory Data

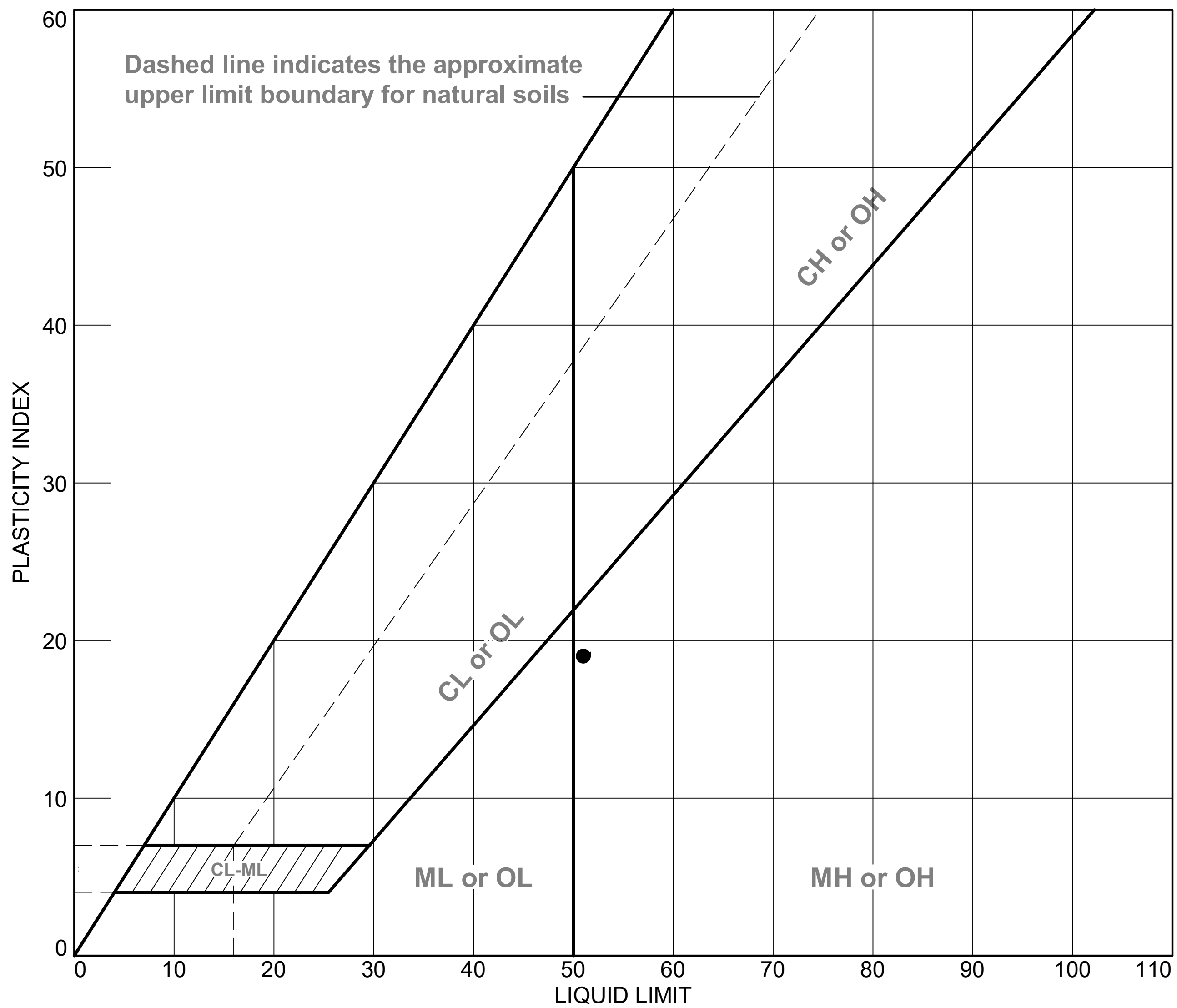
SUMMARY OF LABORATORY DATA

BORING NUMBER	SAMPLE NUMBER	DEPTH (ft)	SOIL GROUP SYMBOL	GRAIN SIZE DISTRIBUTION				PLASTICITY				MOISTURE CONTENT w %	VOLUMETRIC				Unconfined Compression			Organic Content (%)			SHEAR STRENGTH		
				GRAVEL %	SAND %	SILT %	CLAY %	LIQUID LIMIT w_l	PLASTIC LIMIT wp	PLASTICITY INDEX I_P	LIQUIDITY INDEX I_L		SPECIFIC GRAVITY (G)	DRY UNIT WEIGHT (pcf)	VOID RATIO (e)	DEGREE OF SATURATION %	Max Load (lbs)	Compresive Strength (psi)					UNCONFINED COMPRESSIVE STRENGTH (tsf)	COHESION (tsf)	AXIAL STRAIN (%)
B-2	S-13	34-36'		1	79	20						17.3													
B-3	S-6	10-12'	SM	6	81	13		NP	NP	NP	NP	25.8													
B-1A	S-11	29-31'	MH	0	17	83		51	32	19	0.5	41.1	2.58	77.3	1.15	92	29	15.5							
B-3A	S-3	4-6'		0	80	20						21.8													
	S-8	14-16'		3	83	14						59.0								11.7					

PENNONI ASSOCIATES INC.	DRAWN BY: JTR CHECKED BY: TH	DATE: 3/8/2024 DATE: 5/20/2024	PROJECT: **Collings Lake Civic Association** LOCATION: **Lake George/Lake Albert, Williamstown, NJ**
JOB No.: **CLCAX23002-02**	TABLE No.: **L-1**		

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1A	S-11	29-31'	41.1	32	51	19	MH

PENNONI ASSOCIATES INC.

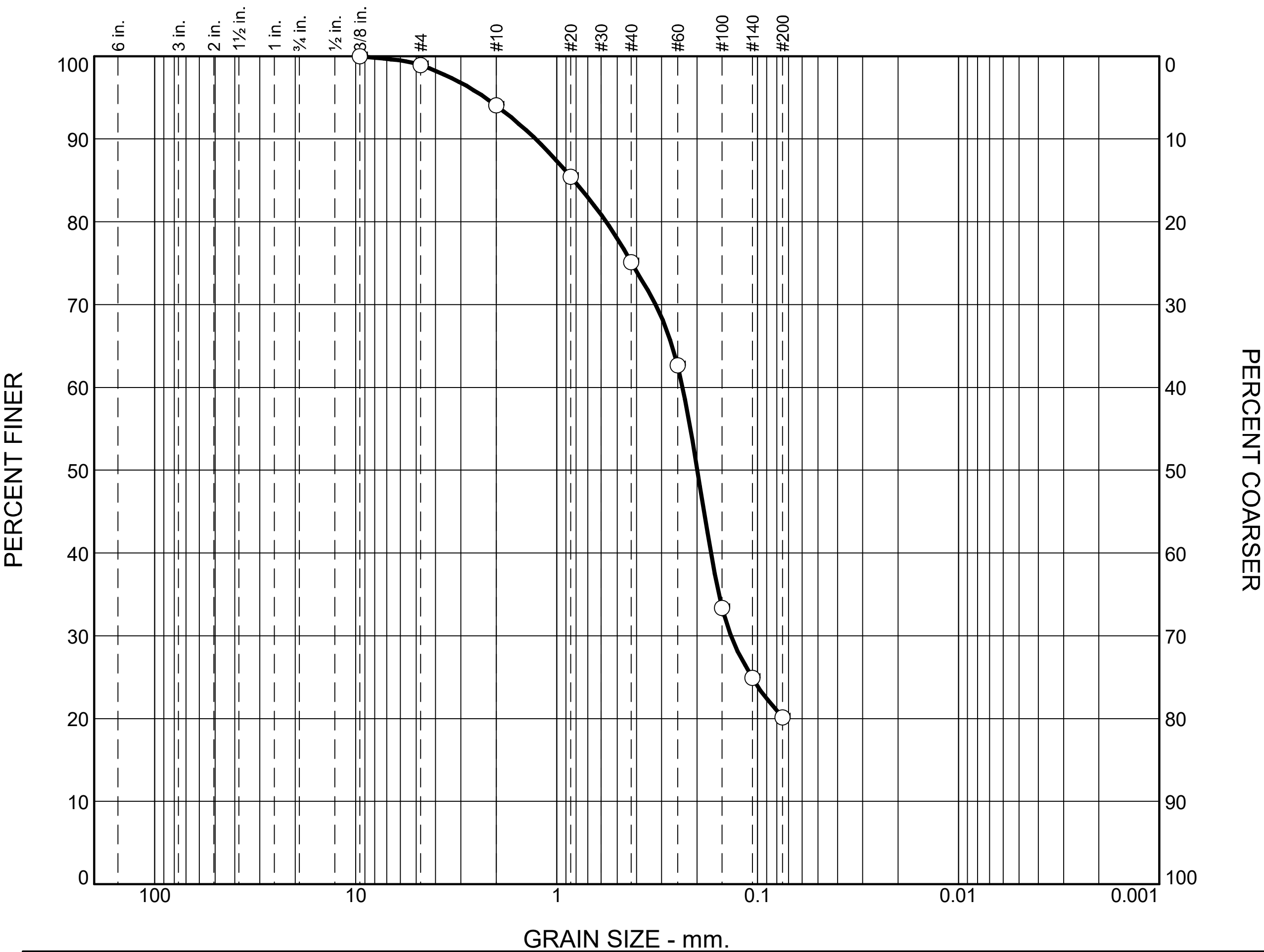
Client: Collings Lake Civic Association

Project: Dam Safety Engineering

Project No.: CLCAX-23002.02 **Figure** A-1

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical s

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	1	5	19	55	20	

SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-2	S-13	34-36'	F/M/C Sand, Some Silt/Clay, Trace Fine Gravel	

PENNONI ASSOCIATES INC.

Client: Collings Lake Civic Association

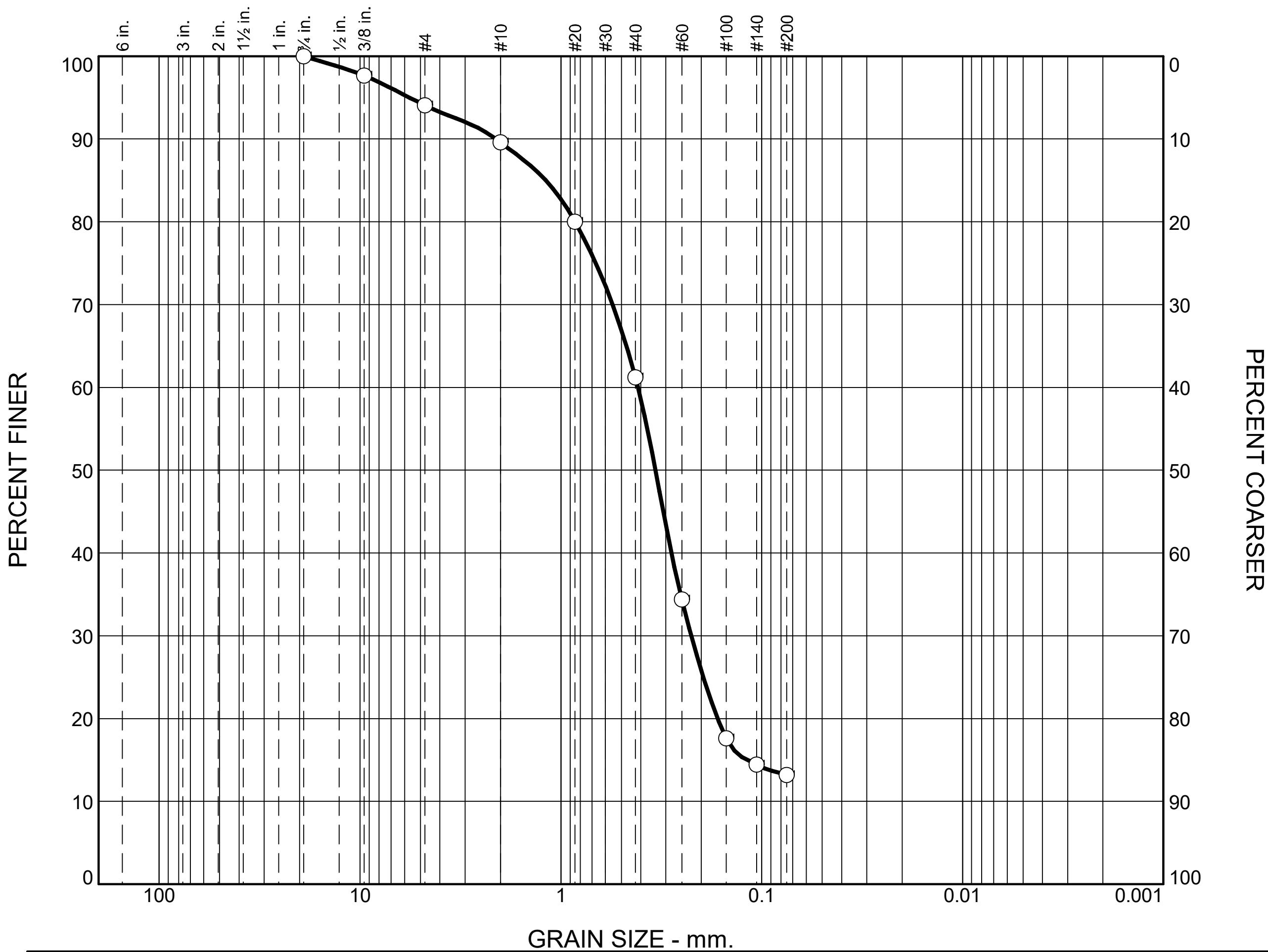
Project: Dam Safety Engineering

Project No.: CLCAX-23002.02 **Figure** S-1

Tested By: MB **Checked By:** JTR

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical s

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	6	4	29	48	13	

SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-3	S-6	10-12'	F/M/C Sand, Little Silt/Clay, Trace Fine Gravel	SM

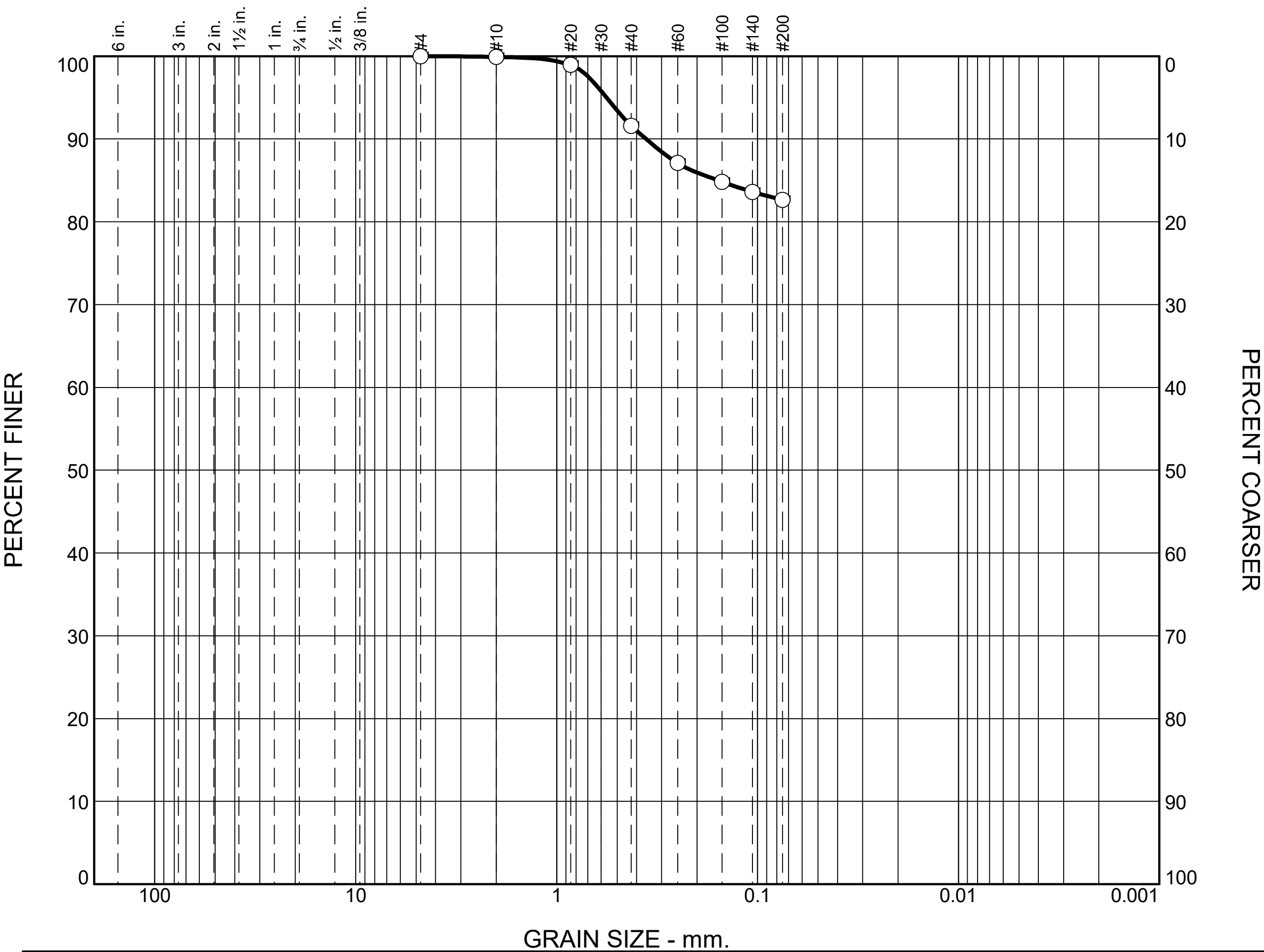
PENNONI ASSOCIATES INC.

Client: Collings Lake Civic Association
Project: Dam Safety Engineering
Project No.: CLCAX-23002.02 **Figure** S-2

Tested By: MB **Checked By:** JTR

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical s

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	0	0	8	9	83	

SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1A	S-11	29-31'	Silt/Clay, Little F/M Sand	MH

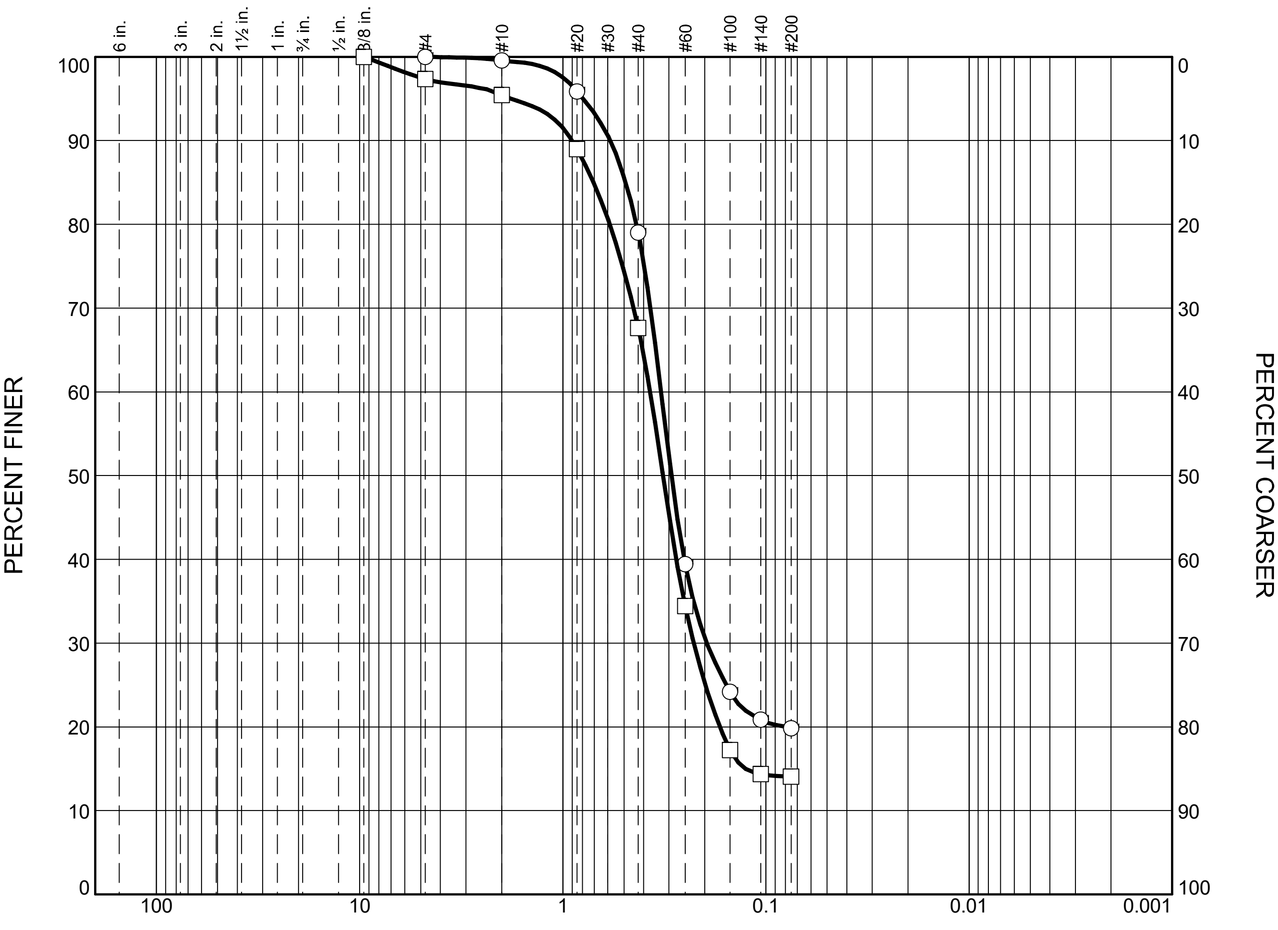
PENNONI ASSOCIATES INC.

Client: Collings Lake Civic Association
Project: Dam Safety Engineering

Project No.: CLCAX-23002.02 **Figure** S-3

Tested By: MB **Checked By:** JTR

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	0	0	21	59	20	
□	0	0	3	2	27	54	14	

SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-3A	S-3	4-6'	F/M Sand, Some Silt/Clay	
□	B-3A	S-8	14-16'	F/M/C Sand, Little Silt/Clay, Little Fine Gravel	

PENNONI ASSOCIATES INC.

Client: Collings Lake Civic Association

Project: Dam Safety Engineering

Project No.: CLCAX-23002.02 **Figure** S-4

Tested By: MB **Checked By:** JTR

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical s

LABORATORY TESTING PROCEDURES

All testing is either done in accordance with the indicated ASTM Designation-latest edition, or with other standard or generally accepted engineering practice as described:

1. Consolidation Test of Soils
   Preparation of samples and testing procedures generally follow the methods described in Lambe, op. Cit. In addition, the time of loading may be selected on the basis of:

   a. Controlled rate of percent of consolidation
   b. Controlled pore pressure gradient
   c. Controlled strain

   The method of test is selected to suit the soil type in question and the test is conducted in accordance with generally accepted engineering practice.

2. Atterberg Limits – Plasticity Indices
   a. Liquid limit of soils, ASTM D 4318
   b. Plastic limit and plasticity index of soils, ASTM D 4318
   c. Shrinkage factors of soils, ASTM D 427

   (Moisture content is also determined with the Atterberg Limit test, and liquidity index is also computed)

3. Moisture Content of Soil
   ASTM D 2216

4. Particle Size Analysis of Soils
   ASTM D 421, Dry preparation of soil samples; ASTM D 422, Sieve and/or hydrometer analysis.

5. Triaxial Compression Test of Soils
   Sample preparation, apparatus, and testing generally follow the procedures outlined in Soil Testing for Engineers, T.W. Lambe, John Wiley & Sons, Inc., New York, 1951 and in The Measurement of Soil Properties in the Triaxial Test, Alan W. Bishop & D.J. Henkel, 2nd Edition, St. Martin's Press, New York, 1962

6. Unconfined Compression Strength of Cohesive Soil
   ASTM D 2166

7. Specific Gravity of Soils
   ASTM D 854

8. Unit Weight Determination of Soils
   See ASTM D 2166 for preparation of specimen except that sample size may differ. For moisture content see ASTM D 2216.

9. Visual Identification of Soil Samples
   All soil samples are visually identified and/or classified. The classification system used is shown in Table L-1.

10. Identification of Rock
    Rock core samples are identified by the character and appearance of newly fractured surfaces of unweathered pieces, by core conditions and characteristics, and by the determination of simple physical and chemical properties.

11. Compaction Test of Soils
    a. Moisture-density relations of soils using 5.5 lb. hammer and 12 in. drop, ASTM D 698
    b. Moisture-density relations of soils using 10 lb. hammer and 18 in. drop, ASTM D 1557

12. Maximum and Minimum Densities of Granular Soils
    Testing procedures follow D.M. Burmeister, "Suggested Method of Test for Maximum and Minimum Densities of Granular Soils" cited in Proceedings for Testing Soils, Fourth Edition, ASTM, Philadelphia. 1964, pp 175-177.

13. Bearing Ratio of Laboratory Compacted Soils
    ASTM D 1883 (Sometimes called California Bearing Ratio or CBR)

14. Organic Content
    A modified dichromate oxidation method using ferrous ammonium sulfate is employed in determining the percent of organic matter in soil.

Appendix C:
Standard Symbols

STANDARD SYMBOLS

B	Width of footing
c	cohesion
c_v	coefficient of consolidation
C_c	compression index
C	coefficient of secondary compression
C_3	swelling index
C_u	uniformity coefficient (D_{60}/D_{10})
CBR	California Bearing Ratio
D_f	depth of foundation
D_p	diameter of grain corresponding to percentage p on grain size curve
D_{10}	effective grain size
E	modulus of linear deformation
E_s	Young's Modulus
e	void ratio
F_s	factor of safety
G	specific gravity
h	hydraulic head
H	stratum thickness
i	hydraulic gradient
I_L	liquidity index
I_P	plasticity index
k	coefficient of permeability
k_h	coefficient of horizontal subgrade reaction
k_v	coefficient of vertical subgrade reaction
l	length of footing
n	porosity
P	deviator stress
P_c	estimated probable preconsolidation pressure
P_o	existing overburden pressure
q_a	allowable soil bearing pressure
Q	triaxial compression test unconsolidated and undrained
Q_c	triaxial compression test consolidated and undrained
S	triaxial compression test consolidated and drained
S_r	degree of saturation
υ	pore-water pressure
U	degree of consolidation
U_c	unconfined compression test
w_f	moisture content at end of test
w_l	liquid limit
w_n	natural moisture content
w_p	plastic limit
γ	unit weight
γ_d	dry unit weight
γ_b	submerged unit weight
ε	unit linear strain
ε_f	unit linear strain at failure
σ	normal stress
σ_1	major principal stress
σ_3	minor principal stress
τ	shear stress
ϕ	angle of internal friction
k_a	coefficient of active pressure
k_p	coefficient of passive pressure
δ	friction angle
$\tan \delta$	friction factor

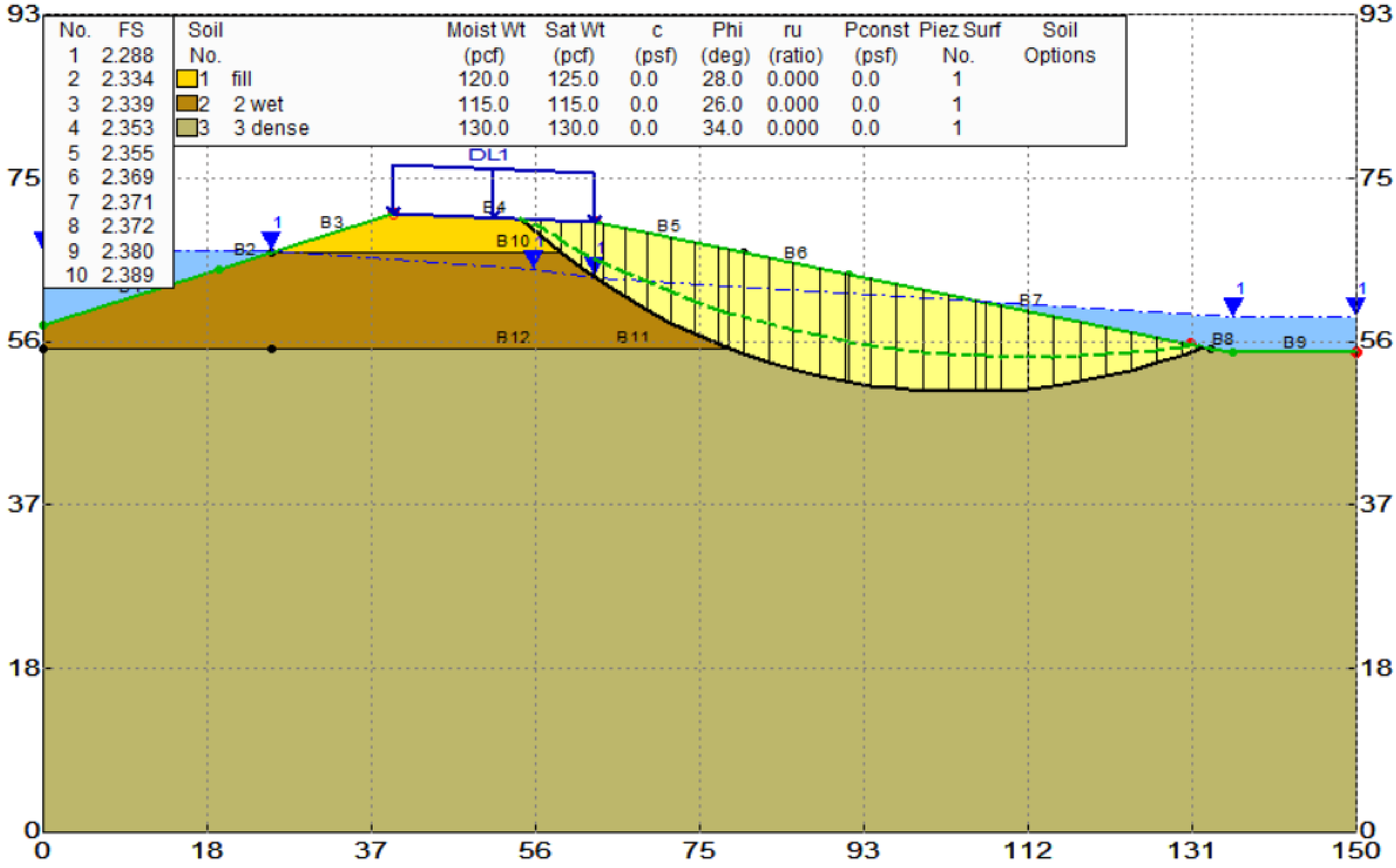
Appendix D:
Analyzed Cross-Sections

Pennoni

Lake George Dam Rehabilitation
Earthen Embankment Normal Pool

Pennoni / TH

\LG b2.gsd



GEOSTASE FS = 2.288

Spencer Method

GEOSTASE Slope Stability Analysis

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PLATE C.1

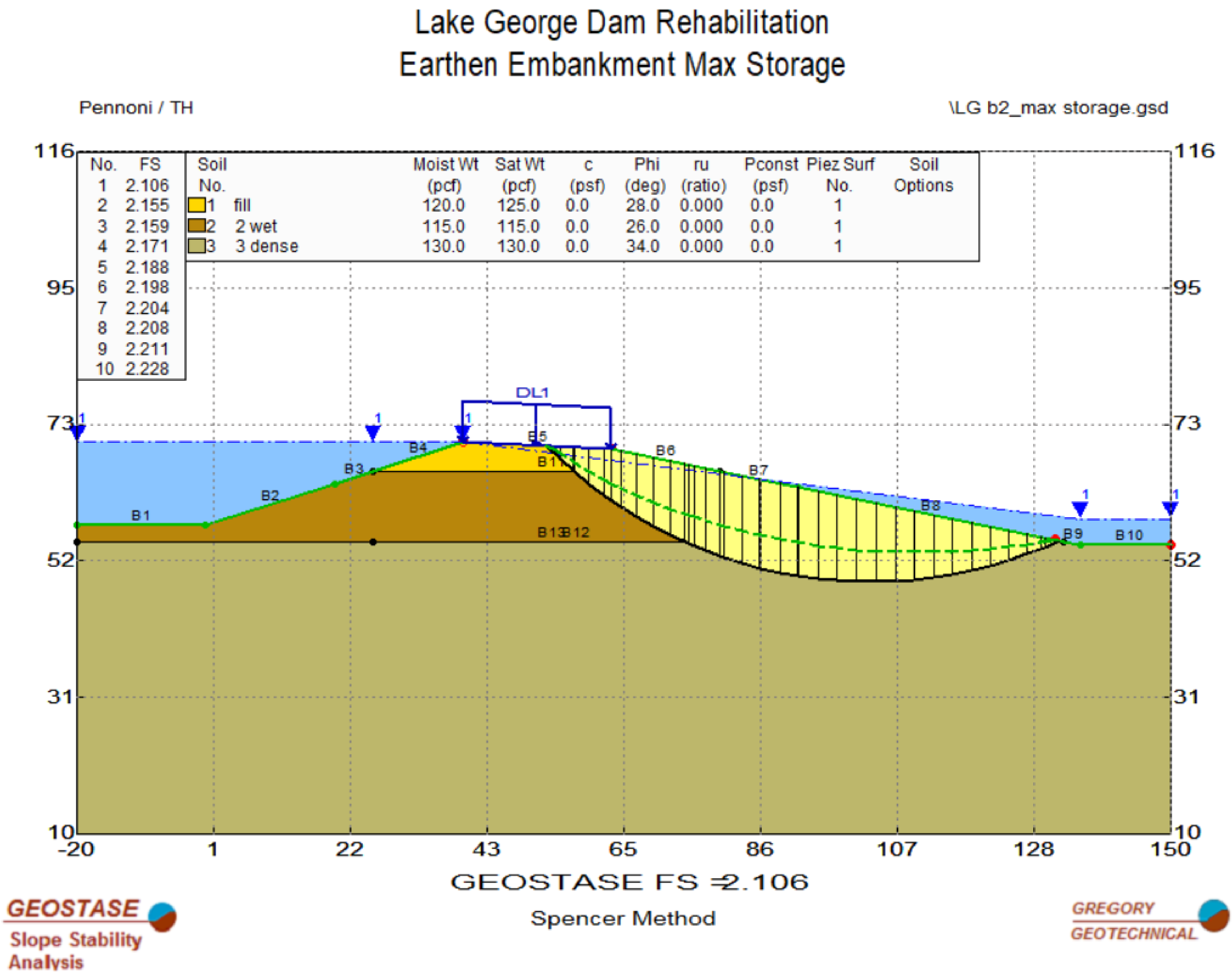

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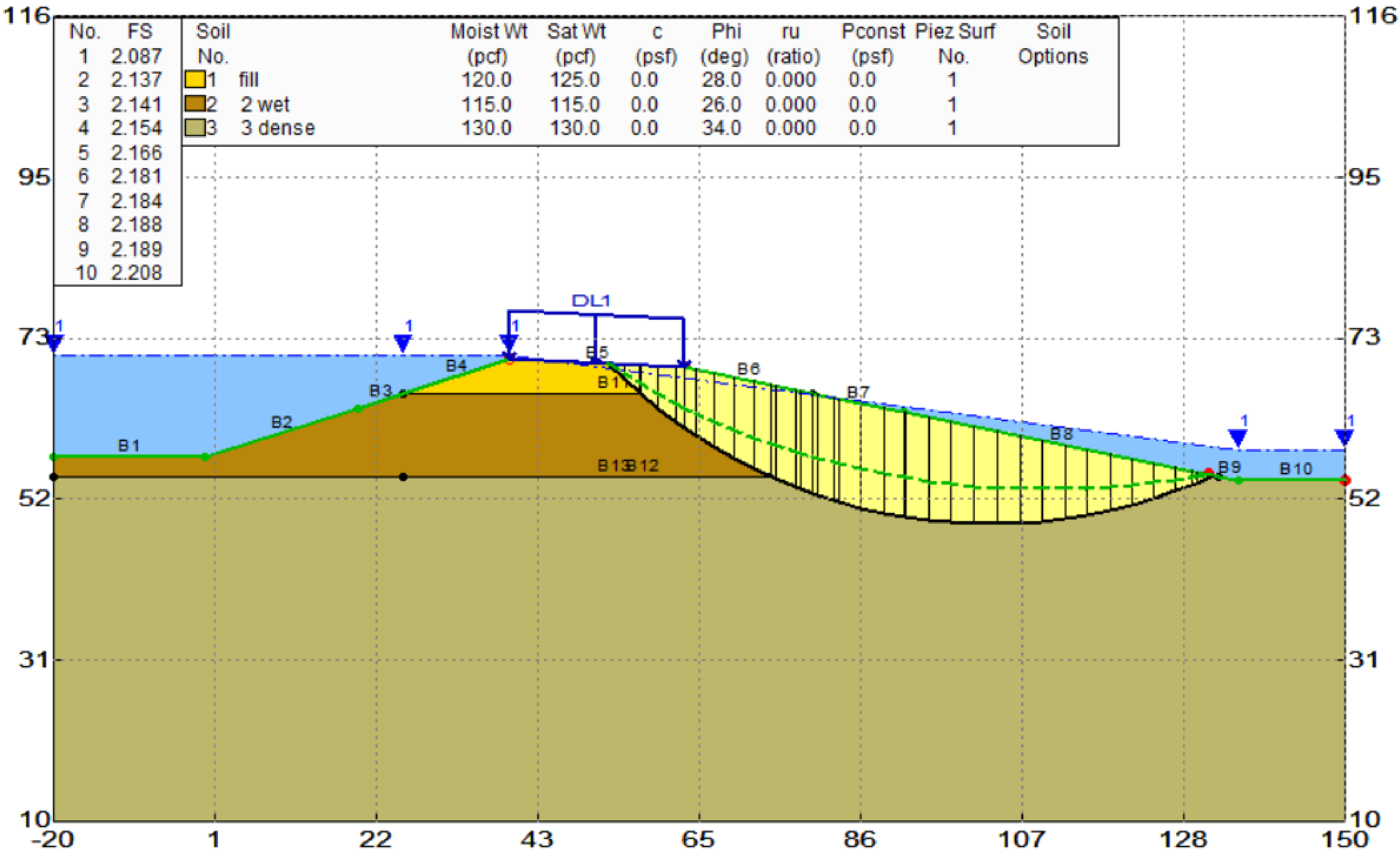
PLATE C.1

Lake George Dam Rehabilitation
Earthen Embankment Max Surcharge

Pennoni / TH

\LG b2_overtop.gsd



GEOSTASE FS = 2.087

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

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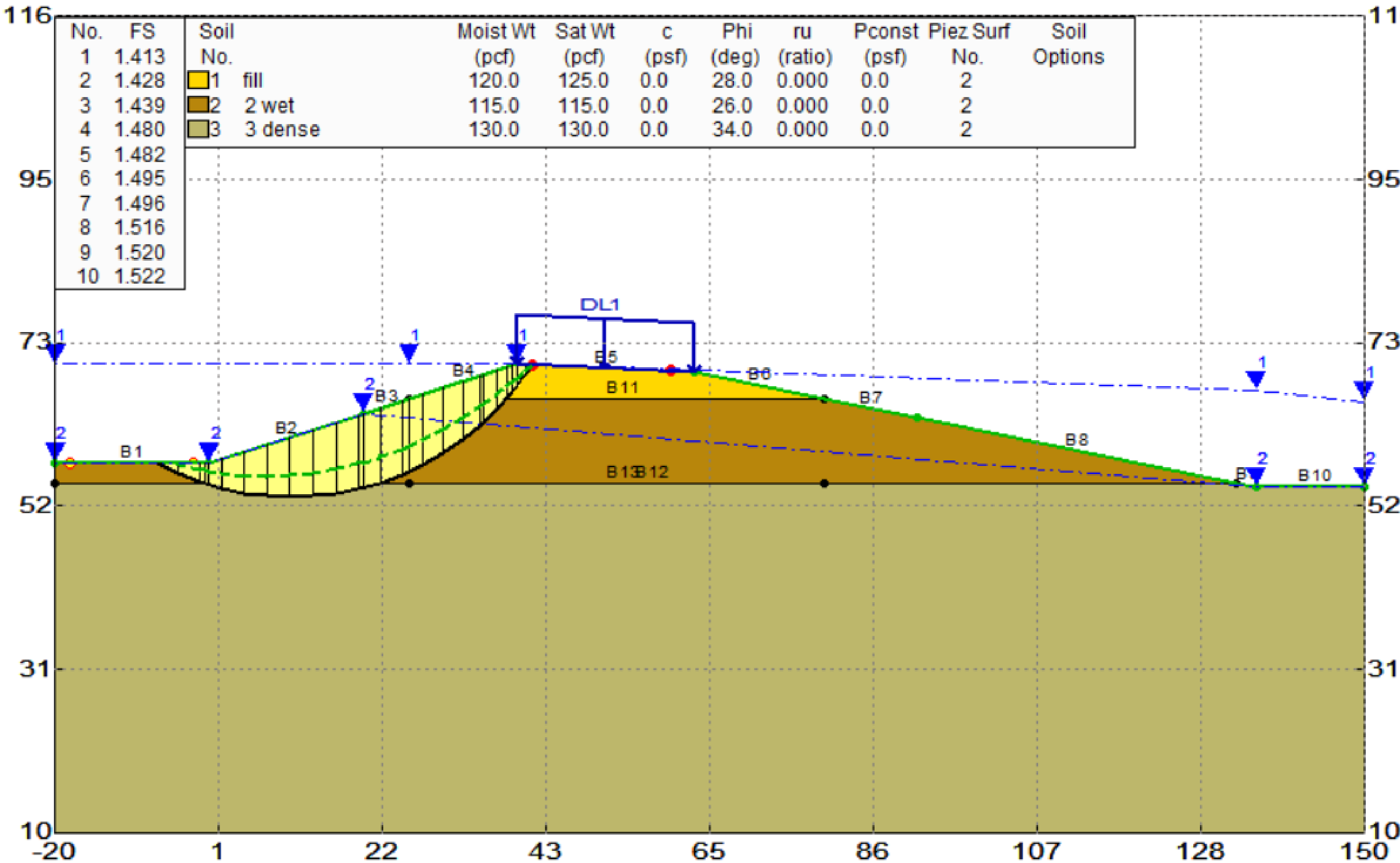
PLATE C.1

Lake George Dam Rehabilitation
Earthen Embankment Max Storage RDD

Pennoni / TH

\LG b2_max storage.gsd



GEOSTASE FS =1.413

Spencer Method

GEOSTASE Slope Stability Analysis

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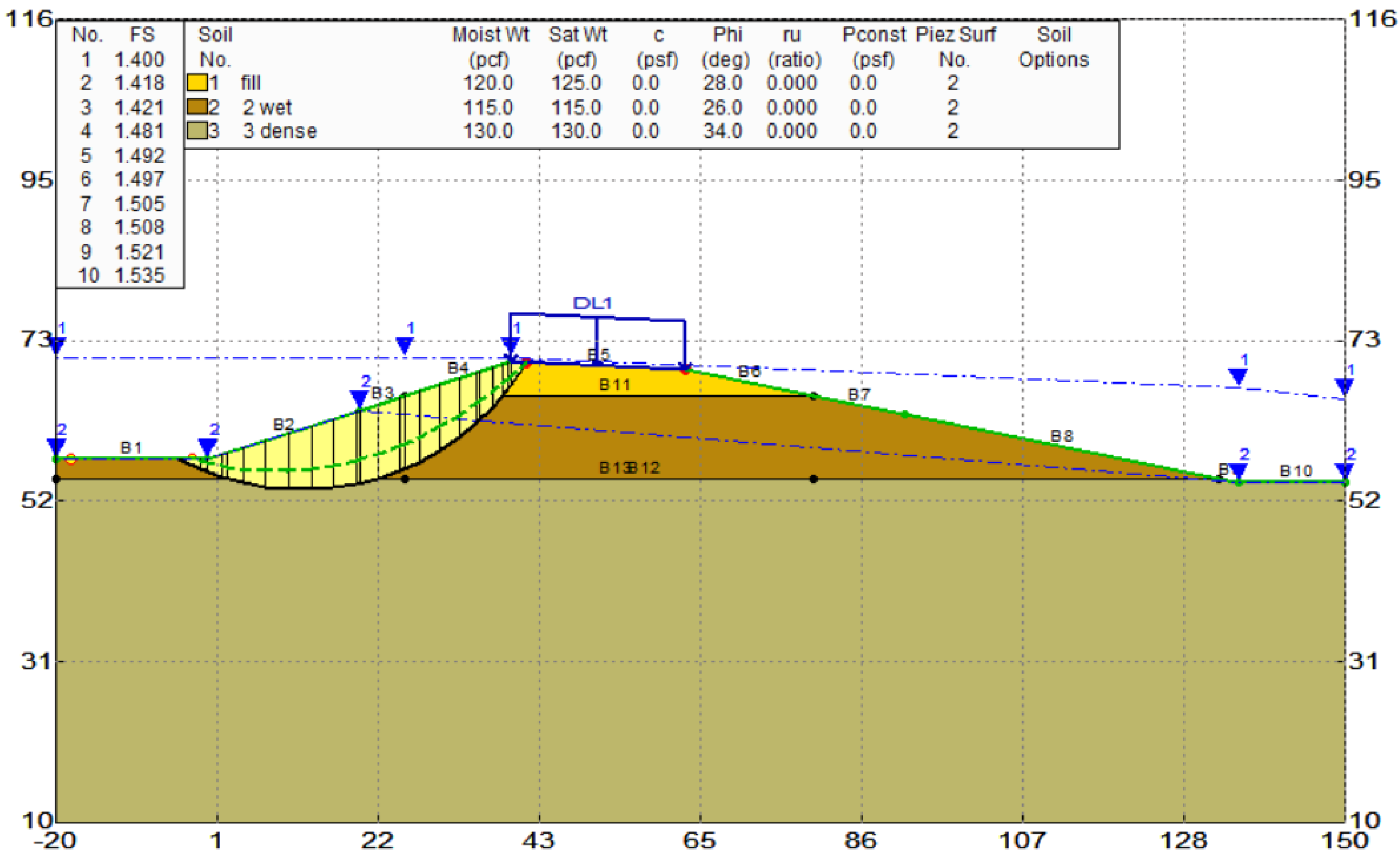
PLATE C.1

Lake George Dam Rehabilitation
Earthen Embankment Max Surcharge RDD

Pennoni / TH

\LG b2_overtop.gsd



GEOSTASE FS =1.400

Spencer Method

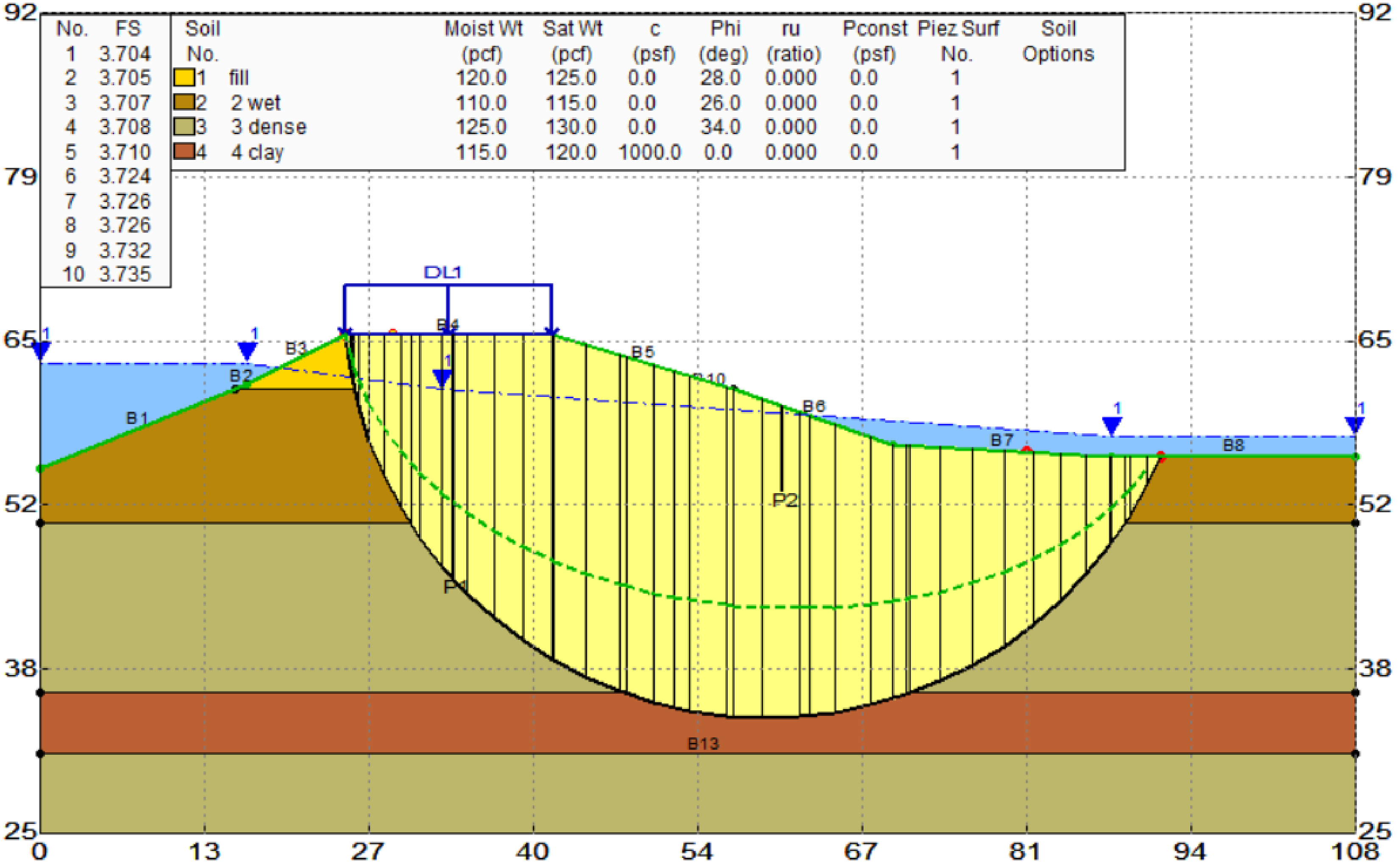
GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

Lake Albert Dam Rehabilitation
Lake Albert Embankment Normal Pool

Pennoni / TH

\LA b3A.gsd



GEOSTASE FS =3.704

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

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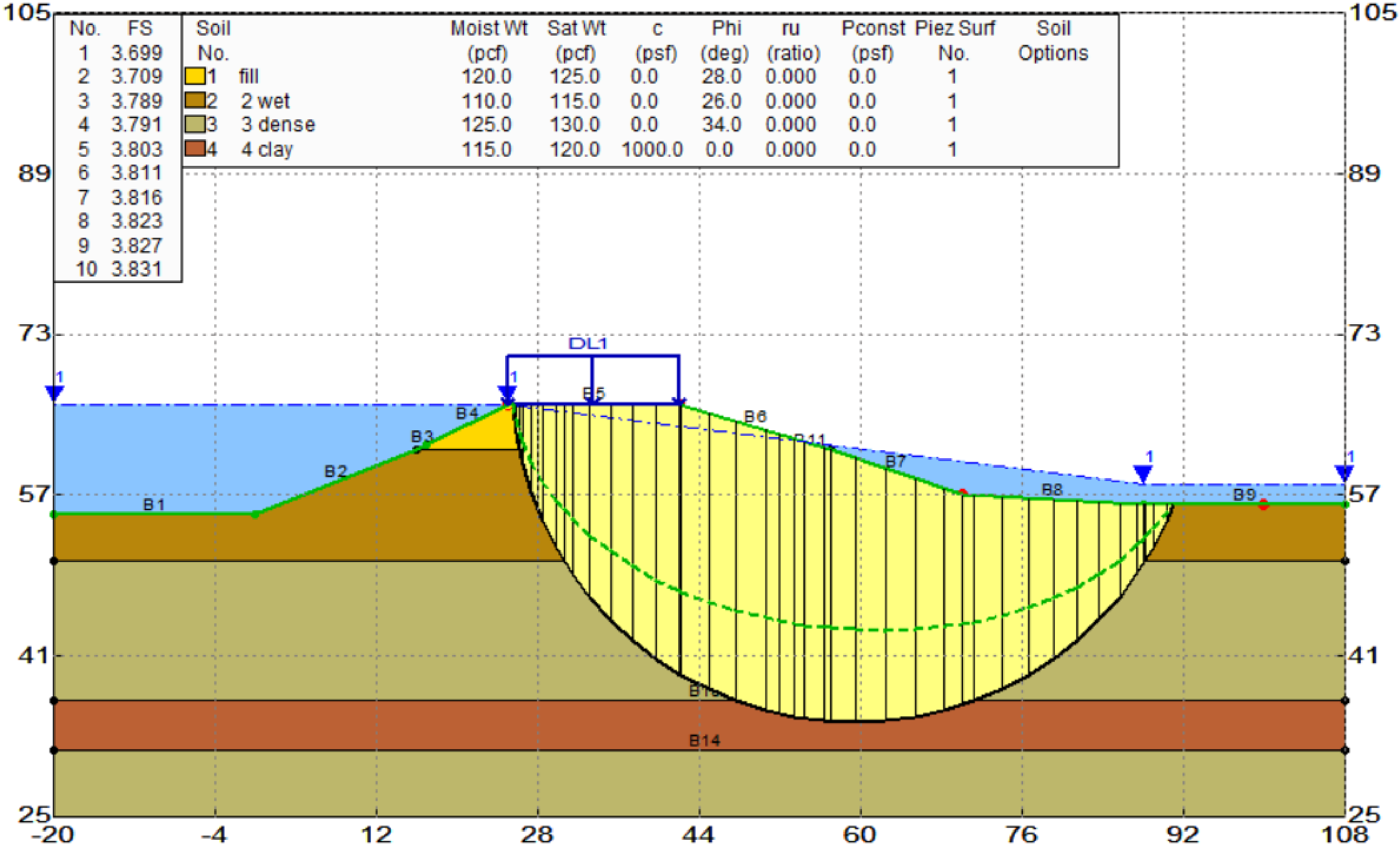
PLATE C.1

Lake Albert Dam Rehabilitation
Embankment Max Storage

Pennoni / TH

\LA b3A_maxStorage.gsd



GEOSTASE FS =3.699

Spencer Method

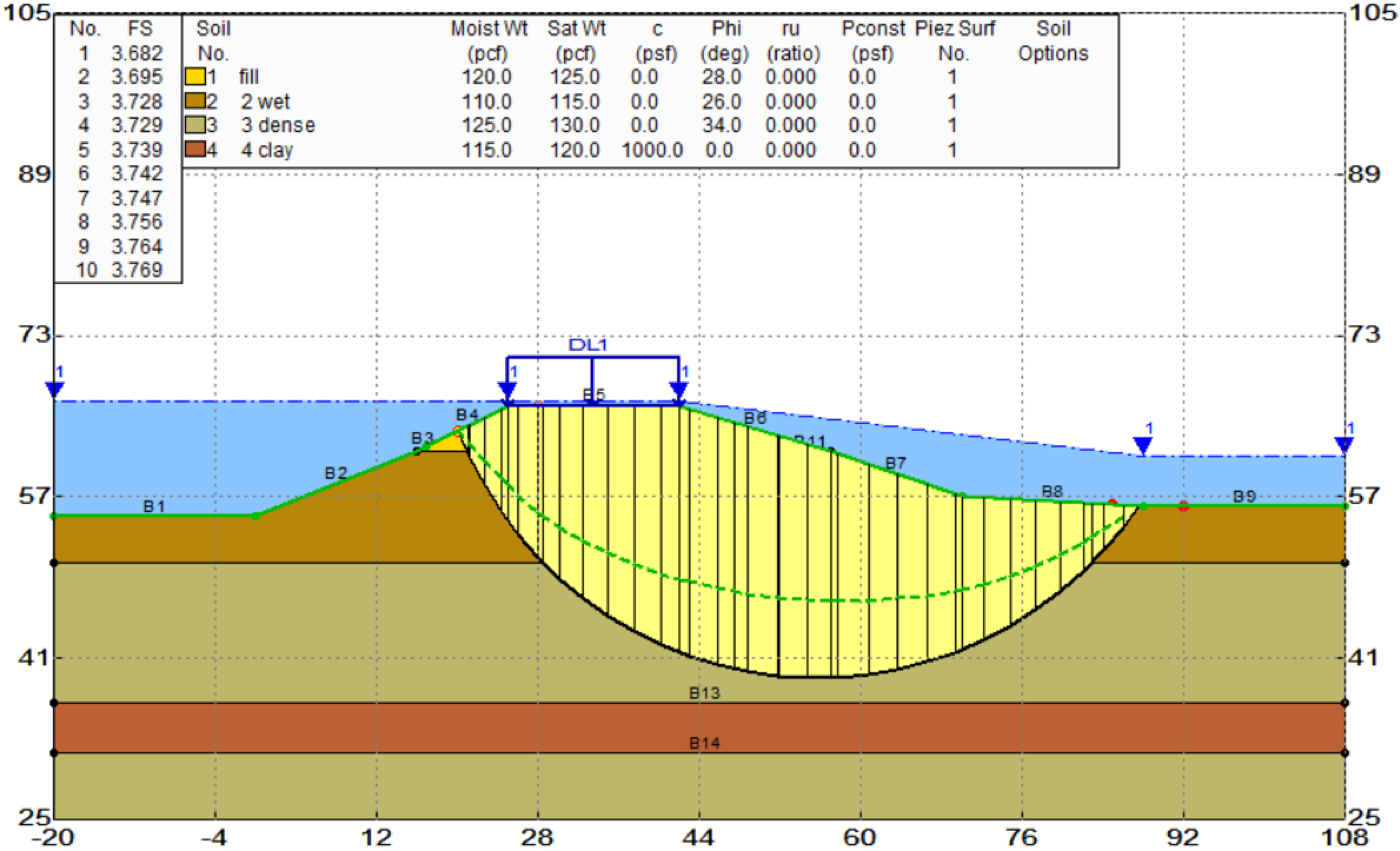
GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

Lake Albert Dam Rehabilitation
Lake Albert Embankment Surcharged

Pennoni / TH

\LA b3A_overtop.gsd



No.	FS
1	3.682
2	3.695
3	3.728
4	3.729
5	3.739
6	3.742
7	3.747
8	3.756
9	3.764
10	3.769

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	fill	120.0	125.0	0.0	28.0	0.000	0.0	1	
2	2 wet	110.0	115.0	0.0	26.0	0.000	0.0	1	
3	3 dense	125.0	130.0	0.0	34.0	0.000	0.0	1	
4	4 clay	115.0	120.0	1000.0	0.0	0.000	0.0	1	

GEOSTASE FS = 3.682

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

PLATE C.1

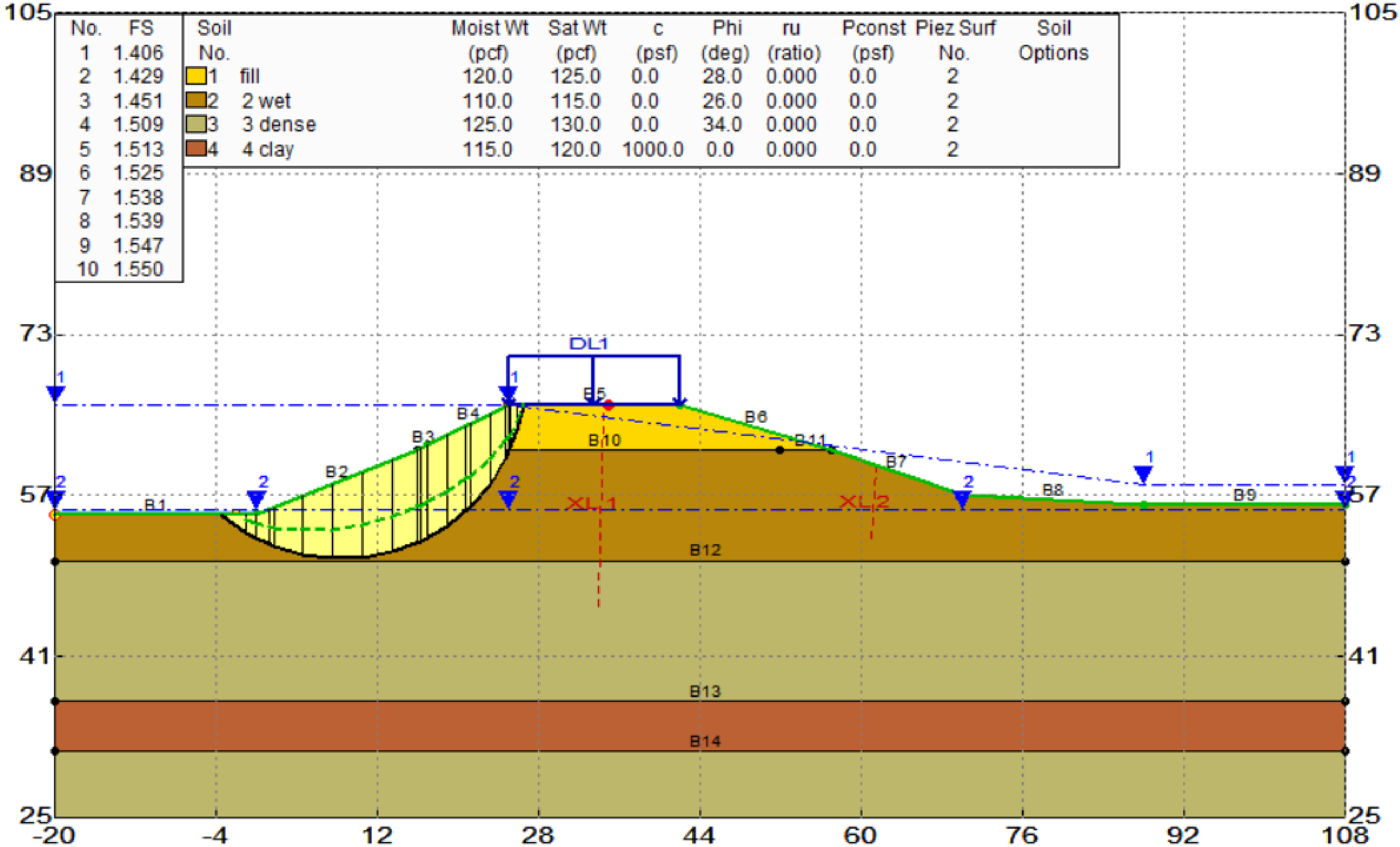
Lake Albert Dam Rehabilitation
Embankment Max Storage RDD

Pennoni / TH

\LA b3A_maxStorage.gsd

No.	FS
1	1.406
2	1.429
3	1.451
4	1.509
5	1.513
6	1.525
7	1.538
8	1.539
9	1.547
10	1.550

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	fill	120.0	125.0	0.0	28.0	0.000	0.0	2	
2	2 wet	110.0	115.0	0.0	26.0	0.000	0.0	2	
3	3 dense	125.0	130.0	0.0	34.0	0.000	0.0	2	
4	4 clay	115.0	120.0	1000.0	0.0	0.000	0.0	2	



GEOSTASE FS =1.406

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

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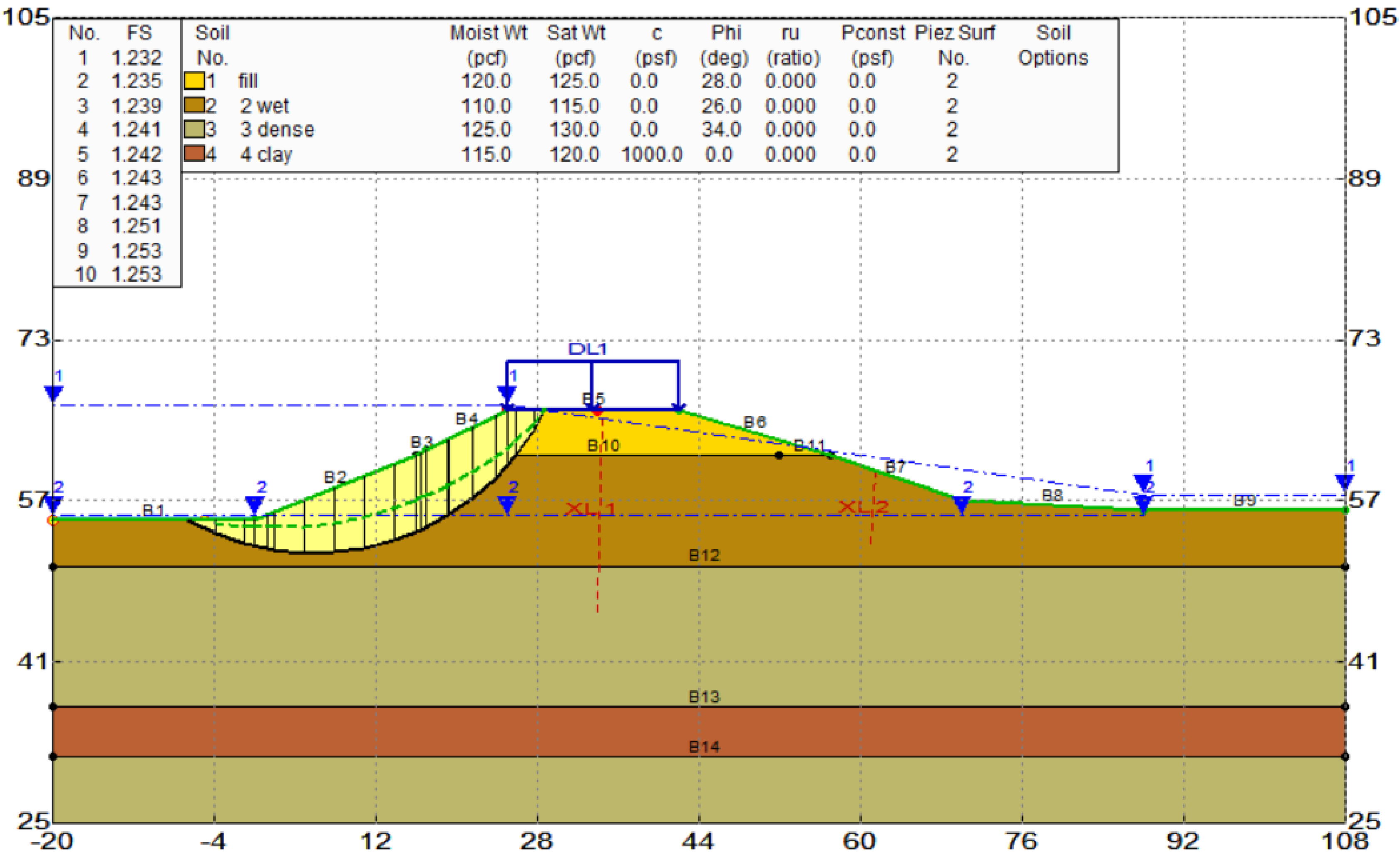
PLATE C.1

Lake Albert Dam Rehabilitation
Embankment Surcharged RDD

Pennoni / TH \LA b3A_overtop.gsd



GEOSTASE FS =1.232

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

PLATE C.1