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22 May 2024

Kate Betsill
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Safe Dams Program
Georgia Department of Natural Resources
2 Martin Luther King, Jr. Drive
Atlanta, Georgia 30334

Subject: **Lake Petit Dam Permit Application**
Appendix A - Stability Analyses of Lake Petit Dam, Revision 1
Pickens County
Permit #112-009-00462

Dear Ms. Betsill:

On behalf of Big Canoe Property Owners Association (POA), Geosyntec Consultants, Inc. (Geosyntec) is providing this cover letter and submitting Revision 1 of the Appendix A - Stability Analyses of Lake Petit Dam from the April 2023 Revision 0 permit application for Lake Petit Dam (Dam).

The April 2023 Revision 0 permit application was revised and submitted to the Safe Dams Program (SDP) in March 2024 (Revision 1) to explicitly remove the portion of the original application related to the stability of the Dam (i.e., Appendix A) while the SDP finalized their review. The revised stability analyses address SDP comments provided in your letters dated 10 August 2023 and 02 April 2024.

If you have further questions, feel free to contact us at 423.385.2310.

Sincerely,

Vernon James Dotson, Jr., P.E. (GA, AL. NC, TN)
Senior Principal Engineer and Engineer of Record
Geosyntec Consultants, Inc.

cc: Scott Auer, Big Canoe Property Owners Association
John Barrett, P.E., Geosyntec Consultants, Inc.
Wesley MacDonald, P.E., Geosyntec Consultants, Inc.

APPENDIX A
Stability Analyses of Lake Petit Dam
Revision 1

LAKE PETIT DAM

Pickens County, Georgia

State ID No. 112-009-00462

NID No. GA00685

Stability Analyses of Lake Petit Dam

Revision 1

Prepared for:

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Project No: TCG10217

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May 2024

Geosyntec
consultants

CALCULATION PACKAGE COVER SHEET

Client: Big Canoe Property Owners Association

Project: New Seepage Collection System and Stability Analyses

Project No.: TCG10217 **Task #:** 03/02

TITLE OF COMPUTATION Stability Analyses of Lake Petit Dam Revision 1

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RECORD OF REVISIONS

Revision Number & Date	Description of Revision
Rev. 0 – 27 April 2023	Initial Submittal to Georgia Environmental Protection Division Safe Dams Program
Rev. 1 – 22 May 2024 Revisions in response to GA SDP comments on Rev. 0	Updates to Sections 3.1.1.3 and 4.2.1
	Addition of Sections 3.3, 4.2.3, 5.4, and 5.5
	Corresponding updates to Conclusions and References
	Addition of Table 4
	Addition of Figures 4 and 5
	Addition of Cyclic Strength Ratio Calculation to Attachment 1
	Updates to Attachment 2 and Figure 2-1

STABILITY ANALYSES OF LAKE PETIT DAM

1 PURPOSE AND SCOPE

This calculation package (Package) was prepared by Geosyntec Consultants, Inc. (Geosyntec) to document the stability of Lake Petit Dam (Dam) with respect to current stability criteria as defined by the Rules and Regulations of the State of Georgia, Rule 391-3-8-.09. This Package presents engineering calculations to evaluate seepage and slope stability of the Dam under the loading conditions described within the regulations described herein.

1.1 Background and Site Geometry

Lake Petit Dam is located within the Big Canoe development on Petit Creek, approximately 5.8 miles upstream of Marble Hill, Georgia (GA) and is owned and operated by Big Canoe Property Owners Association (POA). The reservoir formed by the Dam has a surface area of 107 acres (ac) at a normal pool elevation (El.) of 1,635.5 feet (ft) North American Vertical Datum of 1988 (NAVD88). Elevations reported in this Package are in relation to NAVD88 unless otherwise noted. The storage of the reservoir is approximately 4,235 ac-ft at normal pool elevation, as confirmed by the bathymetric survey conducted in March 2022 which was subsequently approved by GA Safe Dams Program (SDP) in August 2022 (Geosyntec 2022). The Dam has a maximum height of 126 ft measured vertically from the downstream toe, a crest length of approximately 908 ft, and a crest width of approximately 35 ft.

The downstream face of the Dam was designed with 2.5H:1V (horizontal to vertical) slopes, and with 10-ft wide benches at approximately 20-ft vertical intervals. The upstream face of the Dam was designed with a continuous 3.5H:1V slope.

The Dam has a trench drain system (i.e., internal drain system) under the downstream face and is located at approximate El. 1,520 ft. The internal drain system discharges into an outlet structure (i.e., impact basin) with an invert at El. 1,516.7 ft. Downstream of the Dam are the ballfields, which are estimated to be relatively free-draining downstream of the Dam.

1.2 1998 Evaluation of Stability and Rehabilitation Measures

In 1998, Geosyntec evaluated the stability of the Dam under static and seismic conditions. As part of the scope of work, Geosyntec conducted a subsurface investigation, installed dam safety instrumentation, and completed a laboratory testing program on soil samples of the Dam for strength and material characterization. Using the results of the field and laboratory investigation activities, Geosyntec developed a seepage and slope stability model of the Dam to evaluate its performance under normal and seismic loading conditions. The calculated slope stability factors of safety met the requirements of the GA SDP for the global steady-state and pseudostatic scenarios.

1.3 Objective

The 1998 report was submitted to and reviewed by the GA SDP; however, it was never formally accepted as the calculation of record. The purpose of this Package is to document an updated evaluation of the stability of the Dam under the loading conditions required by the Rules and Regulations of the State of Georgia, Rule 391-3-8-.09 for earthen embankments. Specifically, this Package documents an evaluation of the calculated factor of safety against instability for static and pseudostatic loading with steady-state seepage conditions, as well as rapid drawdown analysis.

The remainder of this Package is organized to present: (i) applicable rules and regulations; (ii) methodology; (iii) input data; (iv) analysis results; and (v) conclusions.

2 APPLICABLE RULES AND REGULATIONS

2.1 Loading Conditions

The criteria, defined on "*Rule 391-3-8-.09, Standards for the Design and Evaluation of Dams*", was considered in the slope stability calculations presented in this Package. The following minimum factors of safety can be considered as acceptable stability for the Dam:

- The calculated static factor of safety under the long-term steady-state seepage conditions (i.e., normal pool) must equal or exceed 1.5;

- The calculated pseudostatic (i.e., seismic or earthquake loading) factor of safety under the long-term steady-state seepage conditions must equal or exceed 1.1; and

- The calculated static factor of safety under the rapid drawdown conditions at the upstream side of the Dam must equal or exceed 1.3.

2.2 Normal Pool

Normal Pool is defined as the normal maximum operating range of the reservoir. For Lake Petit Dam, the Normal Pool is at El. 1,635.5 ft.

2.3 Earthquake Loading

The Engineer Guidelines (2015) for the Safe Dams Program in GA states that a dam "shall be able to withstand seismic acceleration defined in the most current map for peak acceleration from a 2 percent exceedance in 50 years (i.e., 2475-year return period) earthquake." and "the minimum required seismic acceleration is 0.05g."

The methodology utilized for development of the site-specific earthquake loading, prepared in accordance with the state regulations, is described in Section 3.2.2.

2.4 Rapid Drawdown

The Engineer Guidelines (2015) for the Safe Dams Program in GA states that the Dam, specifically the gated structure system, shall be designed to drain two-thirds of the reservoir volume at normal pool within 10 days, which constitutes the basis for selection of the lower reservoir level for a rapid drawdown analysis. As stated above, Normal Pool for the Dam is El. 1,635.5 ft and the elevation at which one-third of the reservoir is still impounded is El. 1,602.0 ft.

The GA SDP's rules also reference the rapid drawdown case for a submerged downstream toe. This analysis was not included in this Package because the toe of the Dam is not submerged nor is it interpreted to become submerged during the design flood. During a flood event or discharge of the reservoir through the Spillway, it is unlikely to inundate the downstream side of the Dam due to the discharge point location and local topography of the ballfields and topographic relief downstream of the Dam. The Dam's spillway discharges into Petit Creek at approximately El. 1,514 ft and approximately 250 ft downstream of the impact basin. The next controlled level downstream is Lake Sconti Dam, which is approximately one mile downstream and has an embankment top elevation and normal pool at approximately El. 1,470.0 ft and 1,464.0 ft, respectively.

2.5 End of Construction

The GA SDP's rules also reference the end of construction case for stability following completion of dam construction. Stability of the Dam at the end of construction was not evaluated, as this dam has been constructed and in service for approximately 50 years.

3 METHODOLOGY

Geosyntec evaluated the stability of the tallest cross-section using limit equilibrium calculation procedures to assess the factor of safety. The pore water pressure for Normal Pool was computed with a steady-state seepage analysis. The sections below outline the methodology adopted for analysis.

3.1 Seepage Analysis

Seepage analyses were performed using the computer program SEEP/W, version 2019 (Geo-Slope, 2019a). SEEP/W uses the finite element method (FEM) for analyzing groundwater seepage problems in soil and rock. SEEP/W is capable of modeling saturated and unsaturated flow under steady-state and transient conditions.

The solution procedure for the FEM seepage model consists of defining the geometry by drawing regions that identify distinct lithologic units, assigning material parameters, and defining boundary conditions. The seepage model includes the entire embankment cross-section and underlying foundation units. A global element size of 2 ft was used for developing the FEM mesh. Low-order elements (i.e., three-node triangles and four-node quadrilaterals) were considered adequate for the FEM seepage model.

For the materials in the Dam, the hydraulic conductivities were calibrated within the range previously defined by Geosyntec (1998) until reaching a reasonable representation of the steady-state seepage condition, as interpreted from piezometers within the embankment. Piezometric readings from G-1, G-1B, G-2, P-2, P-4, P-6, and P-7 were used to compare the obtained total head from the model and the defined target value shown in Table 1. The target was selected from the mean value of the data ranging from 2020 to 2022 plus one standard deviation computed using the Three Sigma Rule (Grafarend 2006). While calibrating the seepage model, more weight was given to the piezometers close to the ground surface as they were interpreted to provide a better representation of the phreatic surface; however, this resulted in conservative estimates of the total head (i.e., increased head) deeper within the Dam.

3.1.1 Boundary Conditions

3.1.1.1 Reservoir Loading Condition

The Normal Pool reservoir was simulated with a total head boundary condition set at El. 1,635.5 ft along the upstream face and reservoir of the Dam.

3.1.1.2 Far-Field Boundary Condition

The far-field (downstream) boundary condition for the seepage analyses was set approximately 130 ft downstream of the toe of the Dam. The downstream boundary condition was assumed to be equal to El. 1,516.7 ft and defined as a total head boundary at the far downstream edge of the seepage model. This elevation corresponds to the invert of the trench drain located at the impact basin.

3.1.1.3 Internal Drain System

An internal drain system is located beneath the downstream face of the Dam and collects seepage from the embankment which is connected to the downstream toe via pipes installed during the original construction. This internal drain has been modeled as a discrete point with a total head boundary condition within cross-section A-A, which is shown in Figures 1 and 2. The total head boundary condition allows seepage to exit the model at the location and appropriately represents the internal drain system.

The total head boundary condition assigned to the internal drain system was El. 1,535.0 ft. This boundary condition was selected based on calibration of the seepage model, in which the total head was varied until reaching a reasonable representation of the seepage model based on the target values shown in Table 1 for the piezometer readings. Based on the evaluation of construction records and the sensitivity analyses, we believe the drain is likely functioning and a gradient exists within the Dam towards the drain. The surrounding phreatic line is near approximate El. 1,560, so the capacity of the drain is likely governed by the hydraulic conductivity of the surrounding clay embankment (i.e., strong gradients exist close to the drain, but most seepage bypasses the drain and exits downstream).

3.2 Slope Stability Analysis

Limit-equilibrium slope stability analyses were performed using the computer program SLOPE/W, version 2019 (Geo-Slope, 2019b). SLOPE/W is a 2D slope stability computer program which can be used to employ limit-equilibrium analysis methods. SLOPE/W analyses uses the pore water pressures computed from the seepage analysis performed with SEEP/W.

The method described by Morgenstern-Price (1965) was used to conduct limit-equilibrium slope stability analyses. Morgenstern-Price's method utilizes interslice forces which consider both shear and normal interslice forces. Both moment and force equilibrium are satisfied for individual slices as well as the entire soil mass.

Circular failure surfaces were considered for limit-equilibrium slope stability analyses. For circular failure surfaces, ranges of entry and exit locations for potential slip surfaces were defined along the analyzed slope. The search for the critical slip surface was performed by initially selecting a large range of entry and exit locations, and then refining these ranges once the likely locations of critical entry and exit locations were identified. The entry and exit ranges were divided into 30 increments with four radius increments to evaluate potential failure surfaces.

The minimum sliding mass depth was set at 10 ft in order to avoid results of surficial, localized failures that are not likely to impair the overall embankment stability. These surficial failures can typically be corrected by routine maintenance activities and are not considered to pose a threat to the safety of the Dam. Because unsaturated shear strength is not assigned in these analyses, the effects of negative pore water pressures on shear strength are conservatively ignored.

3.2.1 Static Slope Stability Evaluation

Geosyntec performed static slope stability calculations for both downstream and upstream slopes, using the drained strength parameters for the defined materials and pore water pressures determined from steady-state seepage analyses described above.

3.2.2 Pseudostatic Slope Stability Evaluation

The pseudostatic analysis performed herein accounted for a horizontal seismic loading on the Dam, for both downstream and upstream slopes. The analysis was performed using the defined undrained strength parameters to account for rapid loading conditions within the cohesive soils and effective stress parameters were used for the free-draining materials. To conduct a pseudostatic analysis, a horizontal seismic coefficient (K_s) was computed. K_s was calculated using the method proposed by Bray and Travasarou (2009), an industry-accepted method for analyzing the seismic performance of embankments and slopes. This method utilizes simplified, semiempirical procedures to evaluate the performance of the Dam during earthquake loading.

Seismic coefficient calculations, presented in Attachment 1, are based on the following procedure.

Step 1: Estimate the Fundamental Period

The initial fundamental period (T_s) of the sliding mass was estimated using the following:

$$T_s = 2.6H/V_s \tag{1}$$

where H is the average height of the potential sliding mass, and V_s is the average shear wave velocity of the sliding mass. For this Package, the average height of the potential sliding mass was taken as the height of the Dam (i.e., 126 ft). V_s was calculated as 1,148 ft/s using shear wave velocity tests conducted in boring G-1B (Geosyntec 1998). This data is provided in Attachment 2. The computed T_s for the sliding mass is **0.28 s**.

Step 2: Estimate the Pseudostatic Seismic Coefficient

The K_s was calculated using the equations and relationships provided by Bray and Travasarou (2009):

$$K_s = \exp[(-a+b^{0.5})/0.665] \tag{2a}$$

where variables a and b are calculated using the following relationships:

$$a = 2.83 - 0.566\ln(S_a) \tag{2b}$$

$$b = a^2 - 1.33\{\ln(D) + 1.10 - 3.04\ln(S_a) + 0.244[\ln(S_a)]^2 - 1.5T_s - 0.278(M-7) - \varepsilon\} \tag{2c}$$

where:

- S_a is the 5 percent damped elastic spectral acceleration at the degraded period of $1.5T_s$ of the sliding mass;
- ε is the normally distributed variable to account for the probability of exceedance;
- M is the earthquake's moment magnitude; and
- D is the maximum allowable displacement in centimeters (cm) of the sliding mass.

The site's design spectra was estimated using the online National Seismic Hazard Model (NSHM) Hazard Tool made available by the United State Geological Survey (USGS), which presents a Uniform Hazard Response Spectra (UHRS) created from the National Seismic Hazard Model (USGS 2018). The UHRS analysis was performed using a Site Class D based on ASCE 7.16 (ASCE 2017) according to the V_s. Recent guidelines, such as ASCE 7.22 (ASCE 2021), provide boundary Site classes depending on the V_s. For the Dam, a Site Class C/D was estimated with the most recent guideline; however, Geosyntec conservatively adopted Site Class D in order to incorporate more conservative estimates of ground shaking at the site. The S_a at the degraded period ($1.5T_S$) of the Dam is **0.31 g** for a Site Class D. The estimated UHRS is presented in Attachment 1.

The normally distributed variable (ε) is estimated from a normal distribution function which accounts for the probability of exceedance of the selected displacement threshold (i.e., D). For example, a 50 percent probability of exceedance represents ε=0, while a 16 percent probability of exceedance represents ε=1. In this Package, a 10 percent probability of exceedance was selected (i.e., ε=1.32).

The estimated pseudostatic coefficient is modified based on the moment magnitude of the earthquake (M) selected for analysis. Selection of the magnitude is based upon regional sources of ground motions and typically ranges between 6.5 and 7.5. While the Site is in a region with relatively low seismic hazards, Geosyntec conservatively adopted an earthquake with a moment magnitude **7.0** for analysis and estimation of pseudostatic coefficients.

For embankments, the industry standard for the maximum allowable displacement of earthen dams is 60 cm (approximately 2 ft) during seismic events (FEMA 2005). Based on the Bray and Travasarou (2009) method, the allowable displacement selected herein (i.e., D=2 ft) corresponds to a K_s of 0.054. Multiple analyses were conducted for the pseudostatic stability to evaluate the sensitivity of the model to seismic loading, specifically for the downstream slope (i.e., most critical slope under an earthquake). Initially, the allowable displacement was varied from 10 to 100 cm to compute the K_s with the Bray and Travasarou (2009) method. Additionally, the GA SDP's minimum seismic acceleration of 0.05 g was evaluated as part of the sensitivity analysis. Then, slope stability analyses were performed to determine the factor of safety for each value of K_s. The analysis was also conducted to compute the yield coefficient (K_y) for the Dam. K_y is equal to a horizontal seismic acceleration coefficient that results in a factor of safety equal to one (i.e., the acceleration above which produce deformations in a Newmark analysis).

3.2.3 Rapid Drawdown Slope Stability Evaluation

Rapid drawdown conditions occur when a reservoir level drops rapidly, not allowing for relatively impermeable soils within the embankment to drain. Rapid drawdown decreases the stabilizing effect of the reservoir on the slope, while undrained strengths still govern slow-draining soils within the embankment, resulting in an extreme loading condition on the embankment. The three-stage procedure described by Duncan et al. (1990) is used for the analysis of the rapid drawdown condition:

- Stage 1: Prior to drawdown, steady-state seepage conditions are used to calculate effective consolidation stresses on a failure surface of interest.

- Stage 2: Following drawdown, stability analysis is performed on the failure surface of interest using undrained shear strengths and total-stress analysis. Interpolation is used to estimate undrained shear strength based on effective principal stress ratios after consolidation and at failure.

- Stage 3: If drained shear strengths are less than undrained shear strengths, stability analysis is performed using drained shear strengths, assuming excess pore water pressures induced due to drawdown have dissipated.

This process may then be repeated for other failure surfaces to determine the critical slip surface for sudden drawdown. SLOPE/W automatically performs the previously described stages and reports the critical factor of safety computed for the slope.

To conduct the rapid drawdown analysis, two piezometric lines were used: one for the pre-drawdown steady-state condition (i.e., at El. 1,635.5 ft) and one for the post-drawdown steady-state condition (i.e., at El. 1,602 ft), based on the requirement of draining two-thirds of the reservoir volume and then the procedure described above was implemented.

3.3 Post-Earthquake Deformations

The following sections describe the processes Geosyntec used to conduct a screening-level evaluation of potential for liquefaction and cyclic softening of the soils in the Dam and to demonstrate that the available freeboard for the Dam meets GA SDP requirements for minimum freeboard in the event of post-earthquake deformation.

3.3.1 Liquefaction and Cyclic Softening Screening

Geosyntec used the available shear wave velocity profiles and index properties presented in Attachment 2 to estimate the likelihood of liquefaction and cyclic softening of soils at the Dam due to an earthquake. Based on a procedure described by Boulanger and Idriss (2006), soils can be classified as either ‘sand-like’ or ‘clay-like’ based on the expected behavior during an earthquake depending on index properties and fines content (FC). For example, a fine-grained soil can exhibit ‘clay-like’ behavior if they have a Plastic Index (PI) larger than 7. Atterberg limits and gradation tests were performed in 1998 for both the core and the shell of the Dam.

A criterion developed by Bray and Sancio (2004, 2006) was used to screen for the potential of cyclic softening of the ‘clay-like’ soils. Using this procedure, soils with a ratio of water content (w_c) to liquid limit (LL) less than 0.8 are considered not susceptible to cyclic softening.

A criterion for liquefaction potential of ‘sand-like’ soils established by Andrus and Stokoe (2000) was used to screen for the potential of liquefaction of the shell of the Dam based on shear wave velocities. This criterion uses overburden stress-corrected shear wave velocities to screen for liquefaction potential considering FC and cyclic strength ratio (CSR) (which depends on overburden stress and the design earthquake). Although this criterion was chosen to screen liquefaction of the ‘sand-like’ soils, all available shear wave velocity data for the Dam was used to evaluate liquefaction potential of the Dam materials.

Overburden stress-corrected shear wave velocity is calculated using the following equation described by Sykora (1987), Kayen et al. (1992), and Robertson et al. (1992):

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25} \tag{3}$$

where:

- V_{s1} is the overburden-stress corrected shear wave velocity;
- P_a is atmospheric pressure approximated as approximately 2,116 psf; and
- σ'_v is the initial vertical effective stress at a chosen depth;

The CSR was calculated using the equations and relationships provided by Seed and Idriss (1967):

$$CSR = 0.65 \frac{\sigma_v}{\sigma'_v} \frac{a_{max}}{g} r_d \tag{4a}$$

where:

- σ_v is the vertical total stress and σ'_v is the vertical effective stress at a chosen depth;
- $\frac{a_{max}}{g}$ is the maximum horizontal acceleration as a fraction of gravity; and
- r_d is the shear stress reduction factor that accounts for the dynamic response of the soil profile. This factor can be determined using the equations developed by Idriss (1999):

$$r_d = exp[\alpha(z) + \beta(z)M] \tag{4b}$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \tag{4c}$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \tag{4d}$$

3.3.2 Post-Earthquake Deformation

A procedure described by Ishihara and Yoshimine (1992) was used to determine potential post-earthquake deformations under the assumption that the entire soil mass may exhibit strength reduction. Potential deformations can occur even if soils are deemed not susceptible to liquefaction and cyclic softening because excess pore water pressure will still be generated during an earthquake, which could temporarily reduce the strength of the materials producing permanent deformation at the Dam. The Ishihara and Yoshimine procedure is typically used to assess volume change due to pore water pressure dissipation in saturated sands after a seismic event. The methodology is used as a proxy to determine potential volumetric strain of the Dam, as some of the materials are expected to exhibit 'sand-like' behavior. These assumptions provide a conservative estimate for post-earthquake deformation, which can be used to demonstrate that the available freeboard at the Dam after post-earthquake densification or reconsolidation settlement will be sufficient according to the minimum acceptable freeboard established by the GA SDP (i.e., 3 ft).

4 INPUT DATA

4.1 Cross-Section Used for Analysis

One two-dimensional (2D) cross-section was developed for the seepage and slope stability analyses of the Dam. The cross-section A-A is located along the transverse centerline of the Dam as shown in Figure 1. Cross-section A-A is aligned with existing piezometers installed at the downstream face of the Dam (i.e., piezometers in boring locations G-1, G-1B, G-2, P-2, P-4, P-6, and P-7).

Figure 2 shows the cross-section adopted for the analysis. The surface elevations of the downstream face were developed from a survey of the Dam conducted in May 2021. The slopes of the downstream face were measured to range from 2.2H:1V to 2.5H:1V. The steeper slopes were observed close to the toe of the Dam and the crest. The surface elevation of the upstream face of the Dam was developed from a bathymetric survey of the reservoir conducted in March 2022. The overall slope of the upstream face was measured as 3.5H:1V.

The Dam consists of a shell and core with an underlying saprolite and bedrock. The ballfields are located at the downstream side of the Dam. These subsurface conditions at the Dam were established using information from the following historic sources:

(i) boring logs from the 1998 field investigation conducted by Geosyntec and Piedmont Geotechnical Consultants, Inc.; (ii) boring logs from field investigations prior to the construction of the Dam.; (iii) topographic map of the area prior to the construction of the Dam; and (iv) design drawings for the Dam.

4.2 Material properties

Geosyntec estimated material parameters for analysis based upon a review of previously defined material parameters (Geosyntec 1998) and laboratory test results. As part of the 1998 field investigation, samples collected from the shell and core of the Dam were analyzed in the laboratory for index properties and strengths using isotropic consolidated undrained triaxial compression (ICU-TXC) tests. This data is provided in Attachment 2. Table 2 presents a summary of the material properties selected for the evaluations performed herein. The following subsections present the properties for the subsurface conditions at the Dam used in the seepage and slope stability analyses.

4.2.1 Hydraulic Conductivity

Dam Shell

Based on results from the grain size analyses conducted on Dam shell material, the shell is a silty sand classified as SM based on the Unified Soil Classification System (USCS). The average unit weight (γ) of the shell is 125 pounds per cubic foot (pcf). A vertical hydraulic conductivity (k_V) of 1.6 $x10^{-5}$ ft/s ($4.9x10^{-4}$ cm/s) and an anisotropy ratio (k_V/k_x) of 0.5 for the Dam shell material were used. The hydraulic conductivity was calibrated from the seepage model to reasonably match the target total heads from the piezometers presented in Table 1.

Dam Core

Based on results from the grain size analyses conducted on Dam core material collected, the core is a sandy silt classified as ML based on the USCS. A γ=130 pcf, a k_V of 3.3 $x10^{-6}$ ft/s ($1.0x10^{-4}$ cm/s), and an anisotropy ratio of 0.1 for the Dam core material were used. Similar to the shell, the hydraulic conductivity was calibrated from the seepage model to reasonably match the total heads from the piezometers.

Saprolite

The upstream saprolite was assumed to be relatively impermeable compared to the Dam shell and core. k_V = 3.3 x10^{-9} ft/s (1.0x10^{-7} cm/s) for the upstream saprolite material was used while the downstream saprolite was modeled with k_V = 1.6 x10^{-6} ft/s (4.9x10^{-5} cm/s). The anisotropy ratio assumed for the material was 1.0.

A distinction in the hydraulic conductivity was modeled between the saprolite downstream of the core and the saprolite upstream of the core to capture the influence of an upstream excavation and cutoff trench. The cutoff trench is not explicitly modeled in the geometry; however, the influence of the cutoff trench and upstream excavation was modeled by assigning a relatively lower vertical and horizontal hydraulic conductivity for the saprolite upstream of the core relative to downstream.

Ballfield

In the stability analyses, the ballfield soils have been modeled with γ=125 pcf, k_V = 1.6 x10^{-3} ft/s (4.9x10^{-2} cm/s), and an anisotropy ratio of 1.0. The hydraulic conductivity properties were calibrated based on the seepage model to properly represent a free draining material typically for ballfields.

Bedrock

In the stability analyses, the bedrock was modeled as impenetrable. The bedrock was assumed to be relatively impermeable compared to the Dam shell and core. k_V = 3.3 x10^{-9} ft/s was used for this material. The assumed hydraulic conductivity is supported by the observation that no boils or other indications of upward seepage were observed in the tailwater creek below the Dam (Geosyntec 1998).

4.2.2 Drained and Undrained Strength Parameters

Dam Shell

Based on the dam shell ICU-TXC tests, the effective parameters at the ultimate strength condition were lower than the peak, with a range for the friction angle from 34 to 37 degrees (deg). Geosyntec selected effective friction angle (ϕ') of 34 deg and no cohesion (c') for analysis.

For the current evaluation, Geosyntec adopted the maximum effective principal stress ratio (i.e., maximum obliquity) as the failure criterion for individual laboratory tests results and re-interpreted the undrained strength characterization. Figure 3 presents failure points of individual triaxial laboratory tests based on the criterion of maximum obliquity. A linear relationship was used to define the undrained shear strengths for both the shell and core. A total stress friction angle (ϕ) of 23 deg and a cohesion (c) of 1,000 psf were selected.

Dam Core

The effective stress parameters, ϕ'=32 deg and c'=0 psf, were selected based on the evaluation of the ICU-TXC tests. The undrained parameters, ϕ=23 deg and c=1,000 psf, were obtained for the core as shown on Figure 3 and described in the previous section.

Saprolite

In the stability analyses, the saprolite has been modeled differently at the upstream and downstream of the Dam. The upstream saprolite was modeled as impenetrable, while the downstream saprolite was modeled with γ=125 pcf and drained shear strengths of ϕ'=35 deg and c'=0 psf. These parameters are considered conservative based on the high SPT blow counts measured in the material.

Ballfield

The drained shear strengths of ϕ'=32 deg and c'=0 psf were selected based on typical values of free draining materials judged to representative of fill common for roadway and ballfield construction.

Bedrock

Bedrock was assumed to be impenetrable for slope stability computations.

4.2.3 Index Properties for Liquefaction and Cyclic Softening Screening

Dam Shell

Based on results from the grain size analyses conducted on Dam shell material, the shell is a silty sand with a FC of 30 to 44 percent. Atterberg Limit tests suggest that the Dam shell material is either non-plastic or has a PI of 3. Based on the shear wave velocity profile in boring G-1B, which is predominantly in the shell of the Dam, the Dam shell material generally has a shear wave velocity over 800 ft/s.

Dam Core

Based on results from the grain size analyses conducted on Dam core material, the core is a sandy silt with a FC from 52 to 58 percent and a PI between of 9 and 15. Water content of the Dam core material ranged from 17.5 to 22.0 percent with a LL from 33 to 45. The w_c/LL ratio ranged from 0.49 to 0.53. Based on the shear wave velocity profile in boring G-5, which is predominantly in the core of the Dam, the Dam core material has a lower shear wave velocity of 400 to 600 ft/s in the unsaturated portion but generally has a shear wave velocity over 800 ft/s in the saturated portion of the Dam.

5 ANALYSIS RESULTS

The calculated phreatic surface and total head contours from the seepage analysis are presented in Attachment 3. For the steady-state seepage conditions analyzed, the calculated total heads were higher than the target values presented in Table 1 at several piezometer locations. The computed higher total heads represent a conservatively representative scenario of the Dam's internal seepage, and the results were considered appropriate for the stability analyses.

5.1 Static Slope Stability Evaluation Results

The calculated factor of safety for steady-state seepage slope stability analysis are summarized in Table 3 and the results are presented in Attachment 4. The calculated factor of safety, for both upstream and downstream slopes, are greater than the minimum required value for a long-term steady-state condition.

5.2 Pseudostatic Slope Stability Evaluation Results

The calculated factor of safety for steady-state seepage slope stability under seismic conditions (i.e., pseudostatic analysis) are summarized in Table 3 and the results are presented in Attachment 4.

For the allowable displacement of 60 cm (i.e., 2 ft), a K_s of 0.054 g caused a factor of safety of **1.5** and **2.4** for the downstream and upstream slopes, respectively. Based on the sensitivity analysis, a displacement equal to 100 cm (i.e., approximately 3 ft) resulted in seismic coefficients lower than the state-required seismic acceleration (i.e., 0.05g) for the design and evaluation of dams.

Geosyntec also evaluated a more conservative allowable displacement of 10 cm (i.e., 4 inches). With an allowable displacement of 4 inches, a K_s equal to 0.14 g was calculated, and on the calculated factor of safety was 1.2 for the downstream slope.

When using the GA SDP's minimum seismic acceleration of 0.05 g, a pseudostatic factor of safety of 1.5 was computed for the downstream slope of the Dam. The computed K_y was 0.2 g for a factor of safety equal to one. Note that the K_y is higher than the estimated peak ground acceleration at the site (from the UHRS) of 0.18 g. Therefore, the embankment is considered stable under the seismic loading conditions evaluated herein.

5.3 Rapid Drawdown Slope Stability Analysis

The calculated factor of safety for rapid drawdown condition at cross-section A-A is summarized in Table 3 and the results are presented in Attachment 4.

Assuming a sudden release of two-thirds of the reservoir volume, the calculated factor of safety of 2.1 at the upstream slope is greater than the minimum required value of 1.3. Therefore, the embankment is considered stable under rapid drawdown loading condition considered in this evaluation.

5.4 Liquefaction and Cyclic Softening Screening-Level Analysis

Based on the Boulanger and Idriss (2006) procedure, the core of the Dam is expected to exhibit 'clay-like' behavior, and the shell is expected to behave as a 'sand-like' material with potentially interbedded 'clay-like' materials.

The 'clay-like' soils of the Dam have w_c/LL ratios lower than 0.53 and PI of 9 to 15 and are therefore considered to be not susceptible to cyclic softening based on the Bray and Sancio (2004, 2006) criterion.

The 'sand-like' materials were further evaluated to estimate the potential for liquefaction using the chart proposed by Andrus and Stokoe (2000) that relates the normalized shear wave velocity and the CSR. The CSR was calculated using a maximum horizontal acceleration of 0.18 g based on the expected peak ground acceleration of the sliding mass and ranged from 0.11 to 0.13. Attachment 1 includes a table of calculated CSR values. Figure 4 shows the chart by Andrus and Stokoe (2000) and the normalized shear wave velocity profile values from borings G-1B and G-5 within the Dam. As shown in the figure, liquefaction is not expected for the Dam because the normalized shear wave velocities are mostly larger than 200 m/s (i.e., approximately 656 ft/s) and FC over 20 percent. Note that one data point falls within the liquefaction zone; however, this point is not saturated as it is located above the phreatic level within the Dam and is therefore not susceptible to liquefaction.

5.5 Post-Earthquake Deformation Analysis

The soils which comprise the embankment shell and core are not susceptible to liquefaction and cyclic softening and therefore, significant seismic densification and post-liquefaction reconsolidation settlement are not anticipated. However, potential deformations can occur even if soils are not susceptible to liquefaction as some excess pore water pressure may be generated during an earthquake, which would temporarily reduce the strength of the materials producing permanent deformation at the Dam.

The graphical procedure proposed by Ishihara and Yoshimine (1992) was utilized as a conservative approach to estimate post-earthquake deformations. Since the soils are not susceptible to liquefaction, a factor of safety against liquefaction of 1.1 was adopted. Additionally, the lower bound value of the relative density, D_R, was assumed for this deformation analysis. The estimated volumetric strain following the earthquake is 0.8 percent. The results are presented in Figure 5. Based on the estimated volumetric strain, the vertical settlement is conservatively estimated to be 1 to 2 ft at the crest of the 126-foot-tall Dam. The current freeboard is 11.5 ft (i.e., dam crest El. 1,647.0 ft. less normal pool El. 1,635.5 ft) and therefore, up to 2 ft of settlement will maintain approximately 9.5 ft of freeboard.

This screening level evaluation was used to demonstrate that Lake Petit Dam could maintain a freeboard larger than the minimum acceptable freeboard of 3 ft according to GA SDP in the event of post-earthquake deformations.

6 CONCLUSION

Geosyntec performed seepage and slope stability analyses to evaluate and document the stability of Lake Petit Dam and predicted performance during an earthquake and following a rapid drawdown of the reservoir. Geosyntec reviewed the existing geotechnical and instrumentation data at the Site and updated the geotechnical characterization of the respective geologic and dam units. Additionally, Geosyntec developed seismic loading parameters in accordance with current guidelines for conducting pseudostatic analyses and assessed the potential for liquefaction, cyclic softening, and post-earthquake deformations.

The calculated factors of safety exceed the minimum required values for all load cases as described herein and meets the slope stability criteria established within the GA SDP Guidelines. There are currently no known issues or concerns from a slope stability perspective.

The screening level evaluation of liquefaction potential concluded that the Dam is unlikely to experience liquefaction or cyclic softening. However, under the assumption that the entire soil mass of the Dam experiences a strength reduction, a conservative 1 to 2 ft of settlement could occur at the crest, reducing freeboard to 9.5 ft, which demonstrates that Lake Petit Dam could maintain a freeboard larger than the minimum acceptable freeboard of 3 ft according to GA SDP in the event of seismic-induced deformations.

7 REFERENCES

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TABLES

Table 1 – Piezometer Target Values for Model Calibration

Data Analysis	Mean	Std. Dev.	Target[1]
P-2A	1626.2	0.5	1626.7
P-2B	1611.1	0.9	1611.9
P-2C	1596.1	0.6	1596.7
P-4A	1588.5	2.8	1591.3
P-4B	1573.0	2.1	1575.1
P-4C	1570.6	1.4	1571.9
P-6A	1555.1	0.9	1556.0
P-6B	1538.9	0.8	1539.8
P-6C	1554.2	1.0	1555.1
P-7A	1536.1	0.5	1536.6
P-7B	1522.6	0.4	1523.0
P-7C	1527.6	0.4	1528.0
G-1A Shallow	1598.4	1.9	1600.3
G-1A Deep	1579.5	1.6	1581.0
G-1B	1585.3	1.3	1586.6
G-2 Shallow	1570.5	2.7	1573.2
G-2 Intermediate	1559.9	1.5	1561.4
G-2 Deep	1553.4	0.8	1554.2

Notes:

1. Target total head for the piezometers was selected as the Mean + 1 standard deviation of the piezometers' measured data over the last three years, which represents the upper range of 68% of the data using the Three Sigma Rule (Grafarend 2006).

Table 2 – Summary of Selected Geotechnical Parameters

Material Type	Total Unit Weight	Effective Shear Strength Parameters		Undrained Shear Strength Parameters		Hydraulic Conductivity		
	γ (pcf)	c' (psf)	ϕ' (deg)	c (psf)	ϕ (deg)	k_h (ft/s)	k_v (ft/s)	k_v / k_h
Bedrock	Impenetrable					3.3E-09	3.3E-09	1.0
Ballfield	125	0	32	-	-	1.6E-03	1.6E-03	1.0
Dam Core	130	0	32	1,000	23	3.3E-05	3.3E-06	0.1
Dam Shell	125	0	34	1,000	23	3.3E-05	1.6E-05	0.5
Saprolite D/S	125	0	35	-	-	1.6E-06	1.6E-06	1.0
Saprolite U/S	Impenetrable					3.3E-09	3.3E-09	1.0

Acronyms:

D/S: Downstream
U/S: Upstream

Table 3 – Summary of Calculated Factors of Safety for Slope Stability

Loading Condition	Required Minimum Factor of Safety[1]	Calculated Factor of Safety[2]
Steady-State Seepage Stability (Downstream)	1.5	1.6
Steady-State Seepage Stability (Upstream)	1.5	2.5
Steady-State Seepage Pseudostatic Stability (Downstream)	1.1	1.5 (D=60 cm) [3]
Steady-State Seepage Pseudostatic Stability (Upstream)	1.1	2.4 (D=60 cm) [3]
Rapid Drawdown (Upstream) Stability	1.3	2.1

Acronyms:

None.

Notes:

1. Required minimum factor of safety are from the GA SDP Rules for Dam Safety, Rule 391-3-8-.09.
2. Results of stability analysis for the loading conditions are presented in Attachment 2.
3. The pseudostatic slope stability for the upstream slope was computed for an allowable displacement of 60 cm for a K_s equal to 0.054 g.

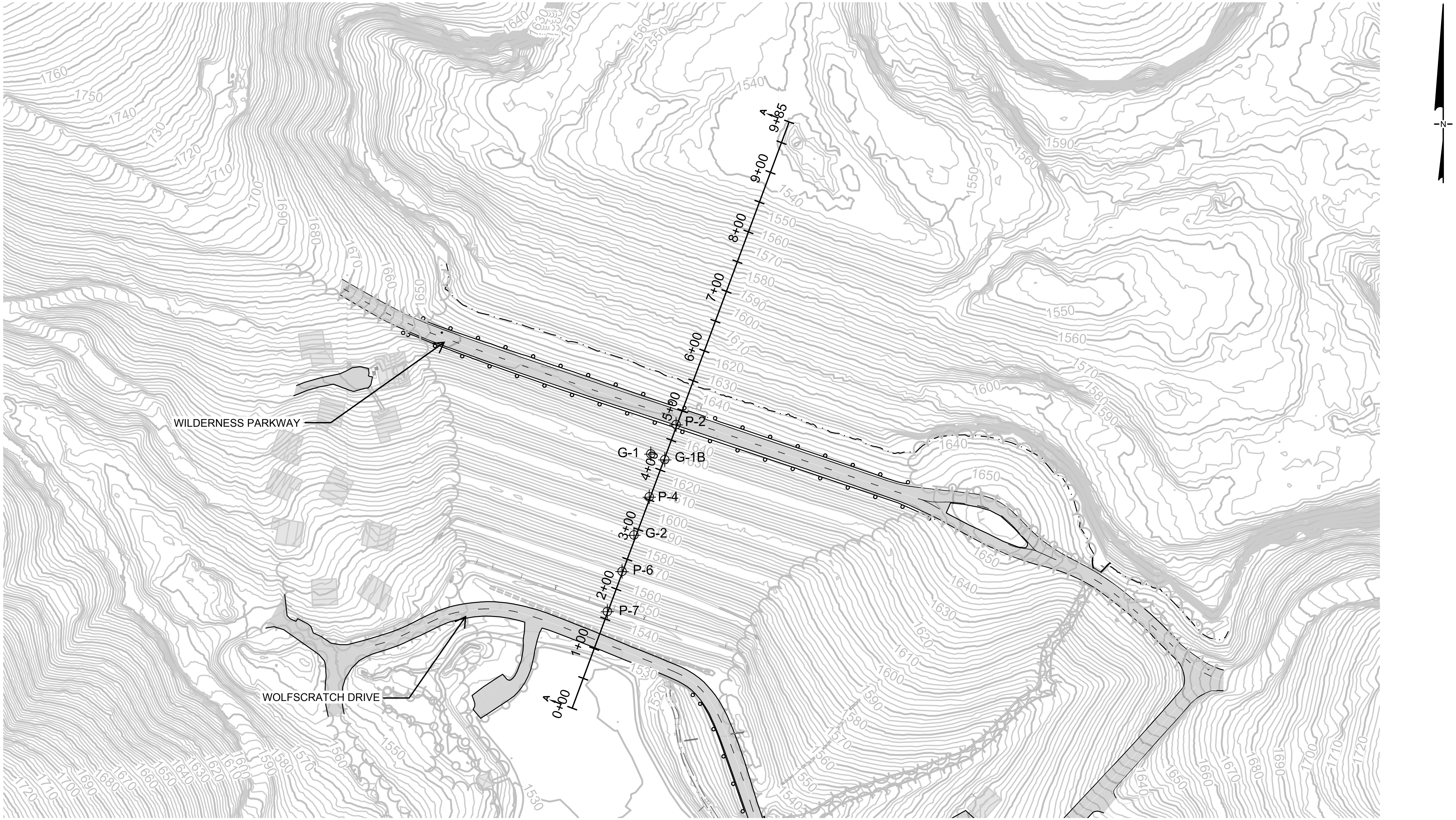
Table 4 – Summary of Soil Characteristics for Liquefaction Potential Screening

Boring	Laboratory Test Number	Material	Water Content, w_c (%)	Liquid Limit, LL	w_c/LL	Plasticity Index, PI	Percent Fines (%)	Sand-Like or Clay-Like
G-4	A	Dam Shell	25.9	NP	--	NP	30	Sand-Like
G-1B	E	Dam Shell	19.8	33	0.60	3	44	Sand-Like
G-1B	F	Dam Shell	16.5	NP	--	NP	36	Sand-Like
G-1B	G	Dam Core	20.7	41	0.50	9	52	Clay-Like
G-5	H	Dam Core	17.5	33	0.53	9	52	Clay-Like
G-5	J	Dam Core	22.0	45	0.49	15	58	Clay-Like

Acronyms:
None.
Notes:

1. Clay-like and sand-like designations using Boulanger and Idriss (2006).

FIGURES



LEGEND

1570	EXISTING GROUND MAJOR CONTOUR (10')
	EXISTING GROUND MINOR CONTOUR (2')
⌖	APPROXIMATE BORING LOCATION OF PIEZOMETERS USED IN THIS CALCULATION PACKAGE.

0 150' 300'
SCALE IN FEET

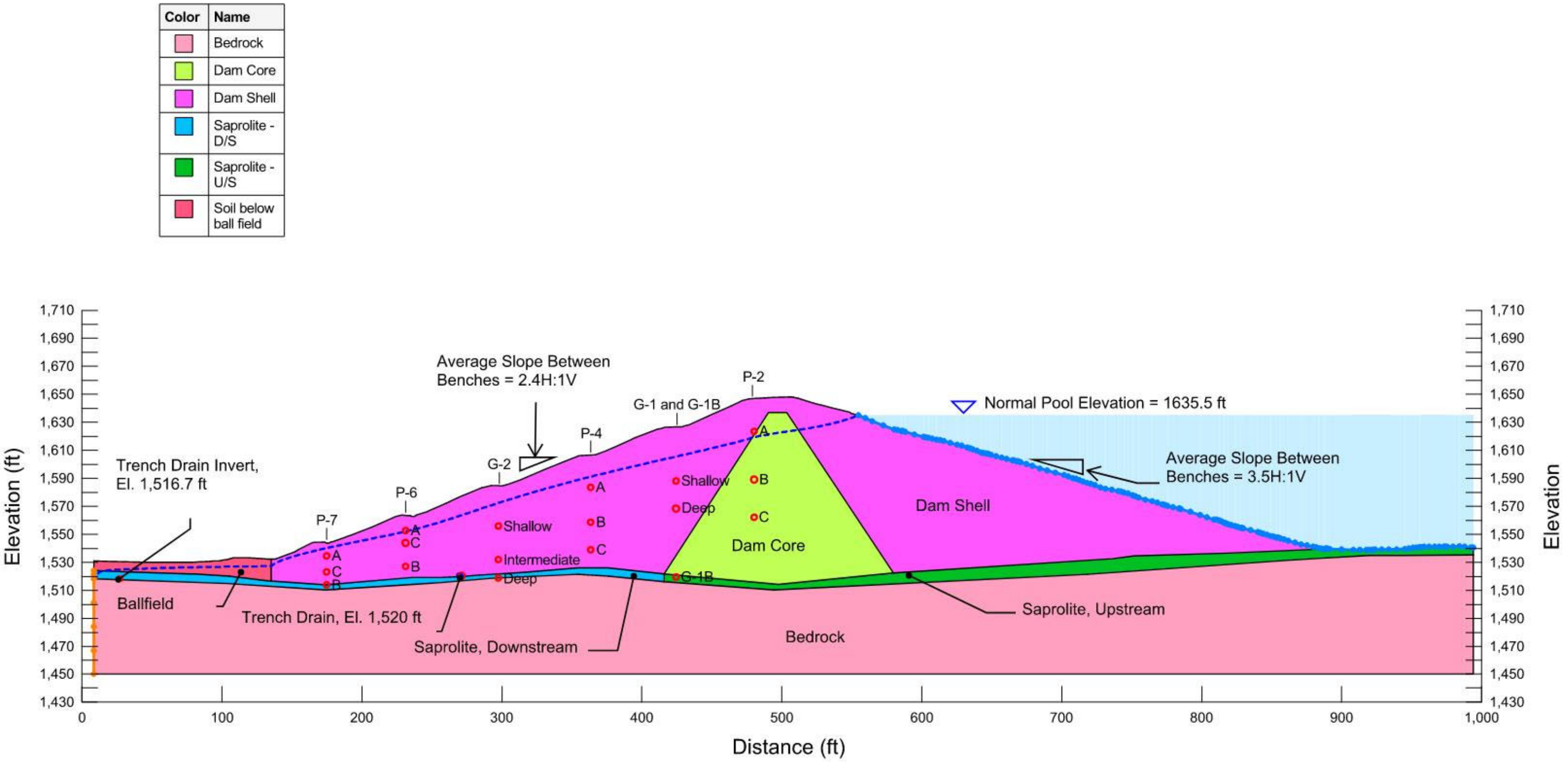
PLAN VIEW CROSS-SECTION A-A
LAKE PETIT DAM

Geosyntec consultants

FIGURE
1

PROJECT NO: TN9418
FEBRUARY 2023

C:_GEOPWDS01\DMS14761\TN9418.03-MD-EXPORT (1998 XSECTION) - Last Saved by: Rodney.Noble on 1/25/23



Notes:
Trench drain is located at elevation 1520 ft.; however, the trench drain is modeled with a total water head set at 1535 ft. to account for the efficiency of the trench drain.

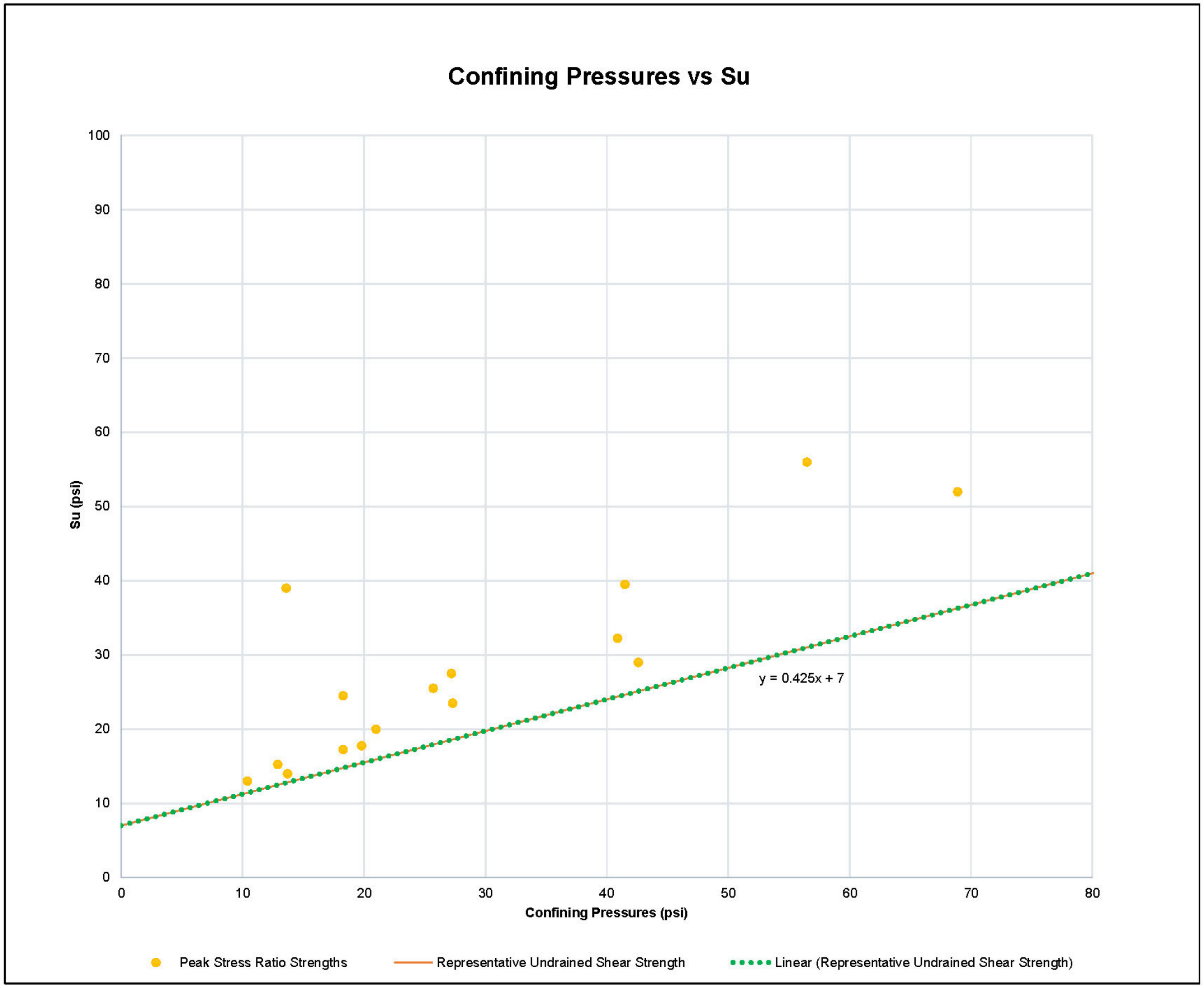
STEADY-STATE SEEPAGE ANALYSIS
LAKE PETIT DAM

Geosyntec
consultants

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure
2



Notes:
Representative undrained shear strength, c = 1000 psf and φ = 23 deg.

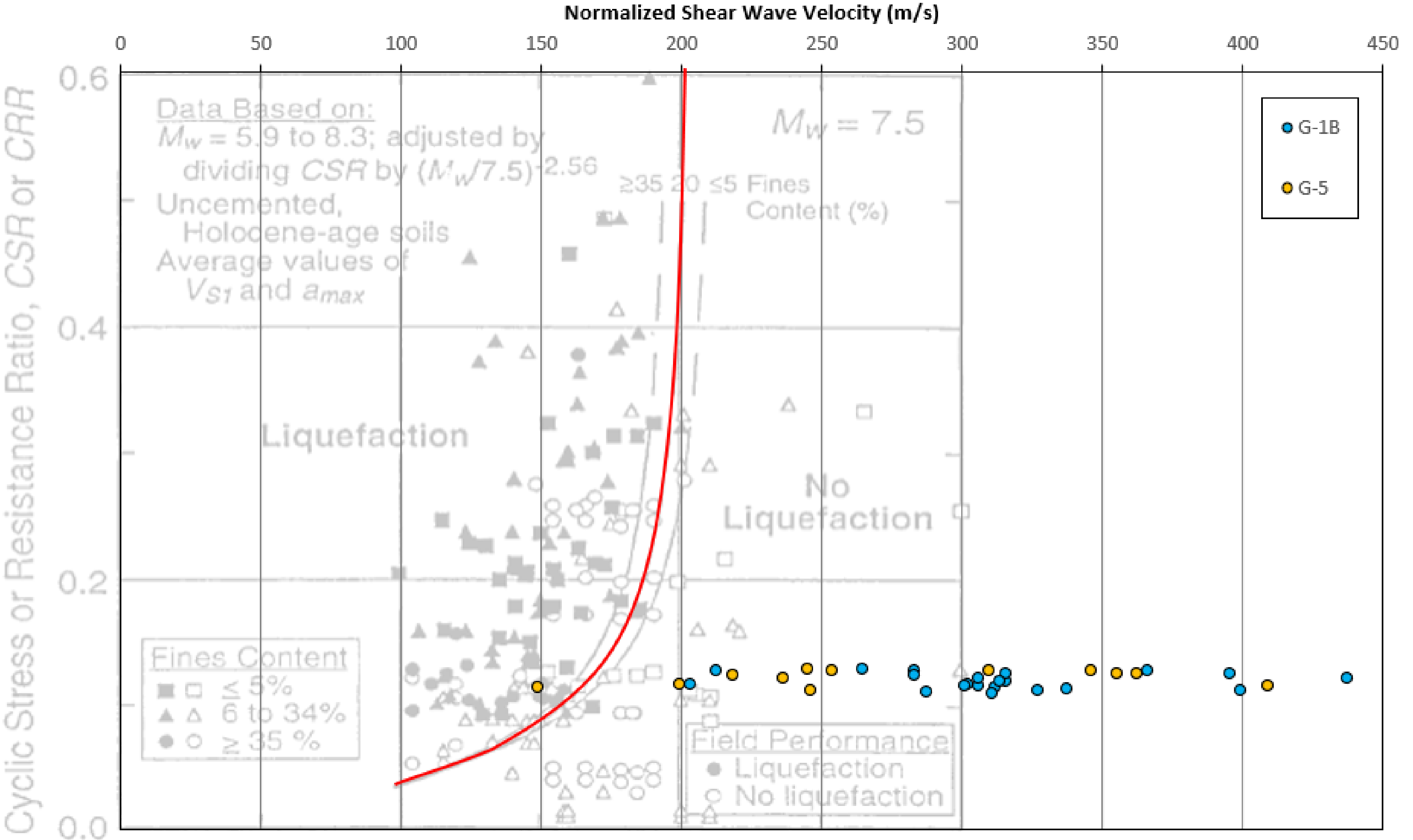
STEADY-STATE SEEPAGE ANALYSIS
LAKE PETIT DAM

Geosyntec consultants

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3



Notes:
Background graphic from Andrus and Stokoe (2000)
Single point plotting in liquefaction zone is in unsaturated portion of the Dam and is therefore not expected to liquefy

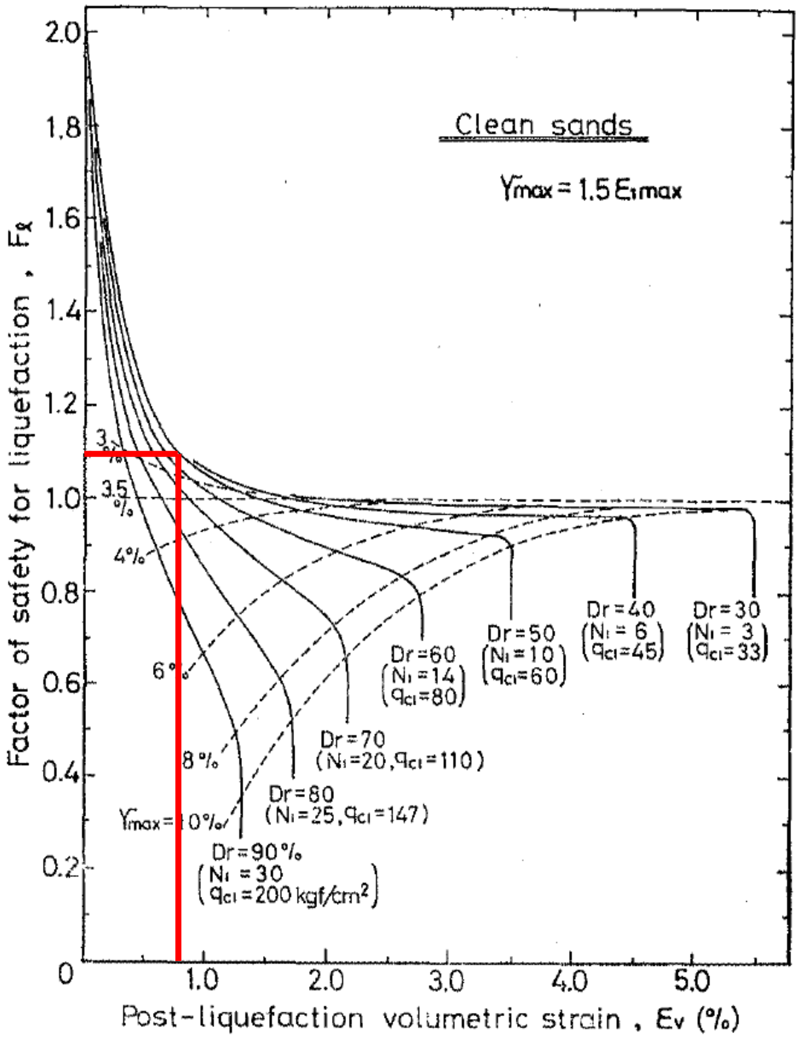
LIQUEFACTION SCREENING CRITERIA
LAKE PETIT DAM

Geosyntec consultants

PROJECT NO. TCG10217

DATE: MAY 2024

Figure 4



Assumptions:

- Entire Dam soil mass experiences post-earthquake strength reduction
- Factor of safety against liquefaction at least 1.1 since liquefaction is not anticipated to occur
- Dam Height: 126 ft

Using graphical procedure:

Post-liquefaction volumetric strain ≈ 0.8%
Settlement = 126 ft (0.008) ≈ 1 ft

Notes:
Graphic from Ishihara and Yoshimine (1992)

POST-EARTHQUAKE DEFORMATIONS
LAKE PETIT DAM

Geosyntec consultants

PROJECT NO. TCG10217

DATE: MAY 2024

Figure
5

ATTACHMENT 1
Site Seismic Evaluation

Written by and Date:	EOA; 02/18/2023
Computation Title:	Average Shear Wave Velocity Calculation
Project Title:	New Seepage Collection System and Stability
Project No.:	TN9418 Task No: 03/02

Average Shear Wave Velocity Calculation

Shear Wave Velocity (ft/sec)	Depth (ft)	Material Description	Shear Wave Velocity by Layer (Denominator of EQ 20.4-1)*
--	0	--	--
648	2.5	SILT	0.00386
816	7.5	SILT	0.00613
957	12.5	SILT and fine to medium sand	0.00522
1333	17.5	SILT and fine to medium sand	0.00375
1074	22.5	SILT and fine to medium sand	0.00466
1105	27.5	SILT and fine to medium sand	0.00452
1466	32.5	SILT and fine to medium sand	0.00341
805	37.5	SILT and fine to medium sand	0.00621
1025	42.5	SILT and fine to medium sand	0.00488
1447	47.5	SILT and fine to medium sand	0.00346
1140	52.5	SILT, very fine sand and gravel	0.00439
1293	57.5	SILT, very fine sand and gravel	0.00387
1178	62.5	SILT, very fine sand and gravel	0.00424
1846	67.5	SILT, very fine sand and gravel	0.00271
1342	72.5	SILT, very fine sand and gravel	0.00373
882	77.5	SILT, very fine sand and gravel	0.00567
1324	82.5	SILT, very fine sand and gravel	0.00378
1501	87.5	SILT, very fine sand and gravel	0.00333
1471	92.5	SILT, very fine sand and gravel	0.00340
1305	96.5	SILT	0.00307
1422	100.5	SILT	0.00281

Low:	648	ft/sec
Max:	1846	Data Source:
Average ($\overline{v}_s$)*:	1148	ft/sec
Median:	1293	ft/sec
Depth:	100.5	ft

Notes:

*Average Shear Wave Velocity, EQ 20.4-1, page 204, ASCE 7-16.

$$\overline{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad (20.4\text{-}1)$$

1) The values for the shear wave velocity and depth have been exported from the Law 1998 report, boring G-1B.

2) Based on the Average Shear Wave Velocity ($\overline{v}_s$) the site would be classified as Stiff Soil (Class D). Please see Table 20.3.1 (ASCE 7-16) for Site Classification based on the average shear wave velocity.

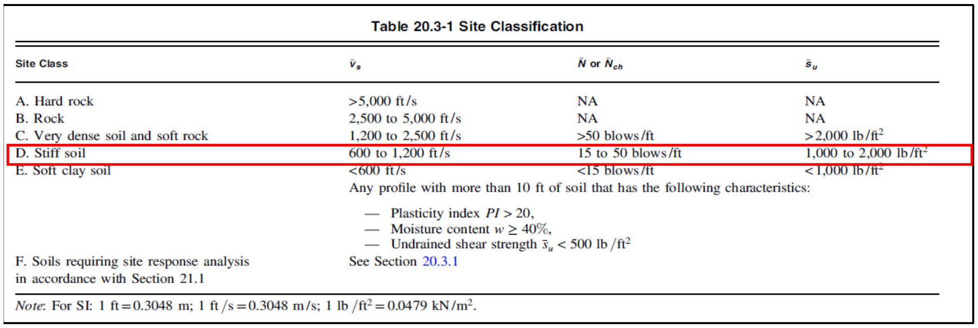


Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50 blows/ft	>2,000 lb/ft^2
D. Stiff soil	600 to 1,200 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
E. Soft clay soil	<600 ft/s	<15 blows/ft	<1,000 lb/ft^2
	Any profile with more than 10 ft of soil that has the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\bar{s}_u < 500$ lb /ft^2		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

Note: For SI: 1 ft = 0.3048 m; 1 ft /s = 0.3048 m/s; 1 lb /ft^2 = 0.0479 kN/m^2.

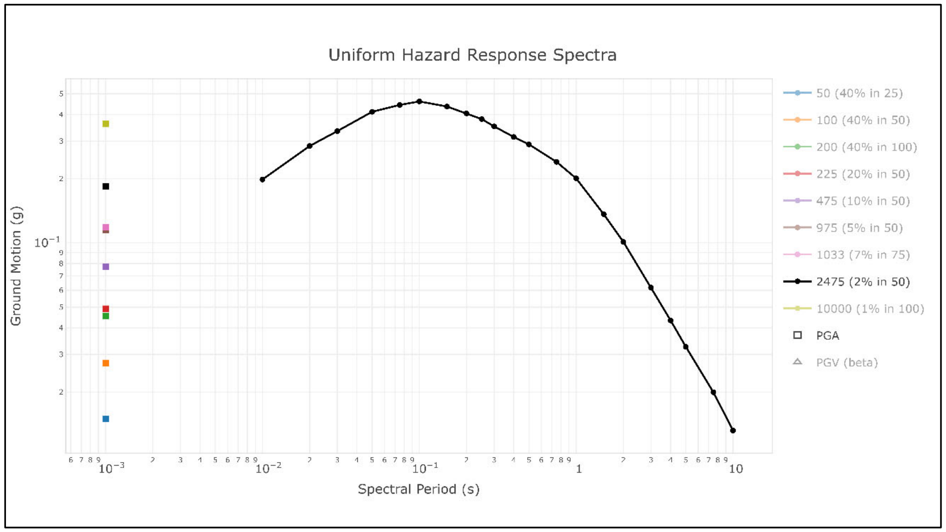
Written by and Date: EOA; 02/18/2023
Title: Uniform Hazard Response Spectra
Project Title: New Seepage Collection System and Stability
Project No.: TN9418 Task No: 03/02

Uniform Hazard Response Spectra Data

Intensity Measure Type (IMT): 2475 (2% in 50)
Peak Ground Acceleration (PGA): 0.184 g

Spectral Period (s)	Ground Motion (g)
0.01	0.198
0.02	0.285
0.03	0.334
0.05	0.412
0.075	0.443
0.1	0.460
0.15	0.436
0.2	0.405
0.25	0.380
0.3	0.352
0.4	0.314
0.5	0.290
0.75	0.240
1	0.201
1.5	0.136
2	0.101
3	0.062
4	0.043
5	0.033
7.5	0.020
10	0.013

Site Location



Notes:
1) Data Source: NSHM (USGS 2018).

Written by and Date: EOA; 02/18/2023
Title: Seismic Coefficient Calculations
Project Title: New Seepage Collection System and Stability
Project No.: TN9418 Task No: 03/02

Seismic Coefficient Calculation

Step 1:

Calculation of Initial Fundamental Period (T_s)

Pseudostatic Analysis in 1D or 2D:

1D: The case of a relatively wide potential sliding mass that is shaped like a trapezoid where:

$T_S = 4H/V_S$

$H =$ 126 ft *<- Height of Dam.*

$V_S =$ 1148 ft/sec *<- Average shear wave velocity.*

$T_S =$ 0.439 s

2D: The case of a triangular-shaped sliding mass that largely has a 2D response, where:

$T_S = 2.6H/V_S$

$H =$ 126 ft *<- Height of Dam.*

$V_S =$ 1148 ft/sec *<- Average shear wave velocity.*

$T_S =$ 0.285 s *<- Due to the geometry of the dam and 2D response expected, this T_s value is used.*

Step 2:

Calculation of the Seismic Coefficient (K_s)

$K_S = \exp[(-a + b^{0.5})/0.665]$

$a = 2.83 - 0.566 \ln(S_a)$

S_a at $1.5T_S$

$1.5T_s =$ 0.428

$S_a =$ 0.306948

Spectral Ground Motion

0.4	0.313671
0.5	0.289592
0.428	0.306948

<- Linear interpolation between 0.4 and 0.5 Spectral Periods.

$a = 3.498$

$b = a^2 - 1.33\{\ln(D) + 1.10 - 3.04\ln(S_a) + 0.244[\ln(S_a)]^2 - 1.5T_s - 0.278(M - 7) - \varepsilon\}$

$a =$ 3.498

$D =$ 20 cm *<- Maximum Allowable Displacement.*

$S_a =$ 0.306948

$T_s =$ 0.285 s

$M =$ 7 *<- Magnitude (M) = 7 moderate event; M = 9 major event.*

$\varepsilon =$ 1.32 *<- Normally distributed random variable with zero mean and standard deviation of 0.66 for 86th percentile, and 1.32 for 95th percentile.*

$b =$ 3.889

$K_S =$ 0.101

Notes:

🟦 Input values/data.

🟧 Output results.

1) The seismic coefficients used in the Pseudostatic Analyses were calculated using a simplified semiempirical predictive procedure (Bray & Travasarou, 2009).

2) The example seismic coefficient calculation presented above was conducted with the assumption of a maximum allowable displacement of 20 cm (approximately 7.9 inches).

3) For the Pseudostatic Analyses, the following parameters are used when calculating the seismic coefficients: H, V_s, T_s, S_a, M, and ε.

3) S_a at a degraded $1.5T_s$ procured from the NSHM Hazard Tool (USGS, 2018).

4) A summary table with calculated seismic coefficients for D = 100, 75, 60, 30, 20, and 10 cm is presented below.

D (cm)	K_s
100	0.038
75	0.047
60	0.054
30	0.081
20	0.101
10	0.140

Written by and Date: KRB; 05/16/2024
Computation Title: Cyclic Strength Ratio Calculation
Project Title: New Seepage Collection System and Stability
Project No.: TCG10217 Task No: 03/02

Cyclic Strength Ratio Calculation

Shear Wave Velocity (ft/sec)	Depth (ft)	Overburden-Corrected Shear Wave Velocity (ft/s)	Shear Stress Reduction Factor r_d	Cyclic Strength Ratio
Boring G-1B				
648	2.5	1035	1.000	0.117
816	7.5	990	0.983	0.115
957	12.5	1022	0.964	0.113
1333	17.5	1309	0.941	0.110
1074	22.5	1004	0.917	0.113
1105	27.5	1004	0.891	0.120
1466	32.5	1298	0.864	0.124
805	37.5	696	0.837	0.126
1025	42.5	868	0.809	0.127
1447	47.5	1201	0.781	0.127
1140	52.5	929	0.754	0.125
1293	57.5	1036	0.727	0.124
1178	62.5	929	0.702	0.122
1846	67.5	1434	0.678	0.120
1342	72.5	1028	0.656	0.118
882	77.5	667	0.636	0.116
1324	82.5	988	0.618	0.114
1501	87.5	1107	0.601	0.112
1471	92.5	1072	0.587	0.110
1305	96.5	943	0.578	0.109
1422	100.5	1018	0.569	0.108
Boring G-5				
1344	2.5	2147	1.000	0.117
539	7.5	654	0.983	0.115
457	12.5	488	0.964	0.113
822	17.5	807	0.941	0.110
1436	22.5	1343	0.917	0.113
854	27.5	776	0.891	0.120
1316	32.5	1165	0.864	0.124
1313	37.5	1136	0.837	0.126
949	42.5	803	0.809	0.127
1223	47.5	1015	0.781	0.127
1021	52.5	832	0.754	0.125
1484	57.5	1189	0.727	0.124
908	62.5	716	0.702	0.122

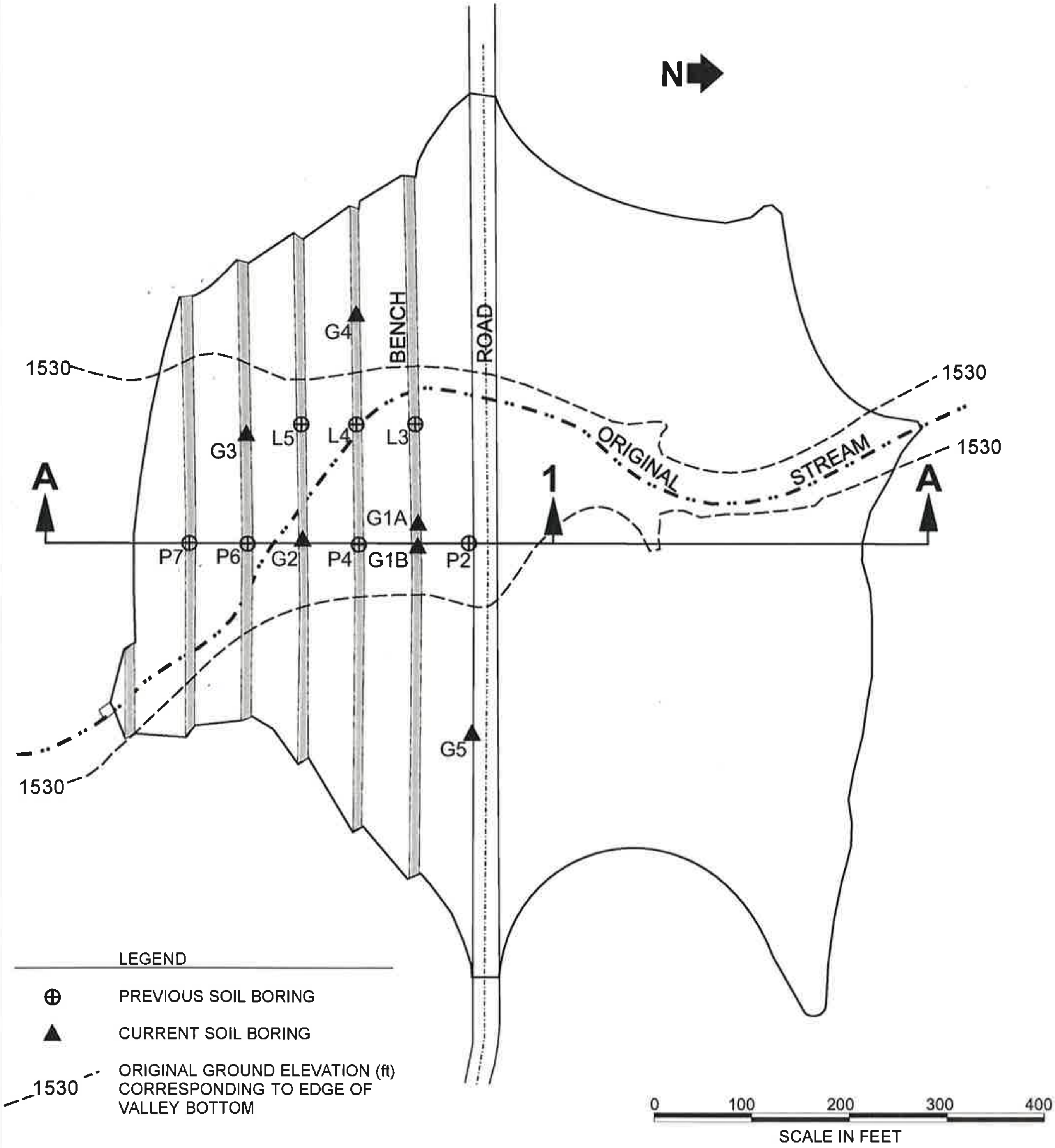
Low:	0.108
Max:	0.127

Notes:

1) The values for the shear wave velocity and depth have been exported from the Law 1998 report, borings G-1B and G-5.

ATTACHMENT 2
Geotechnical Data

PETIT COVE DAM
PLAN VIEW



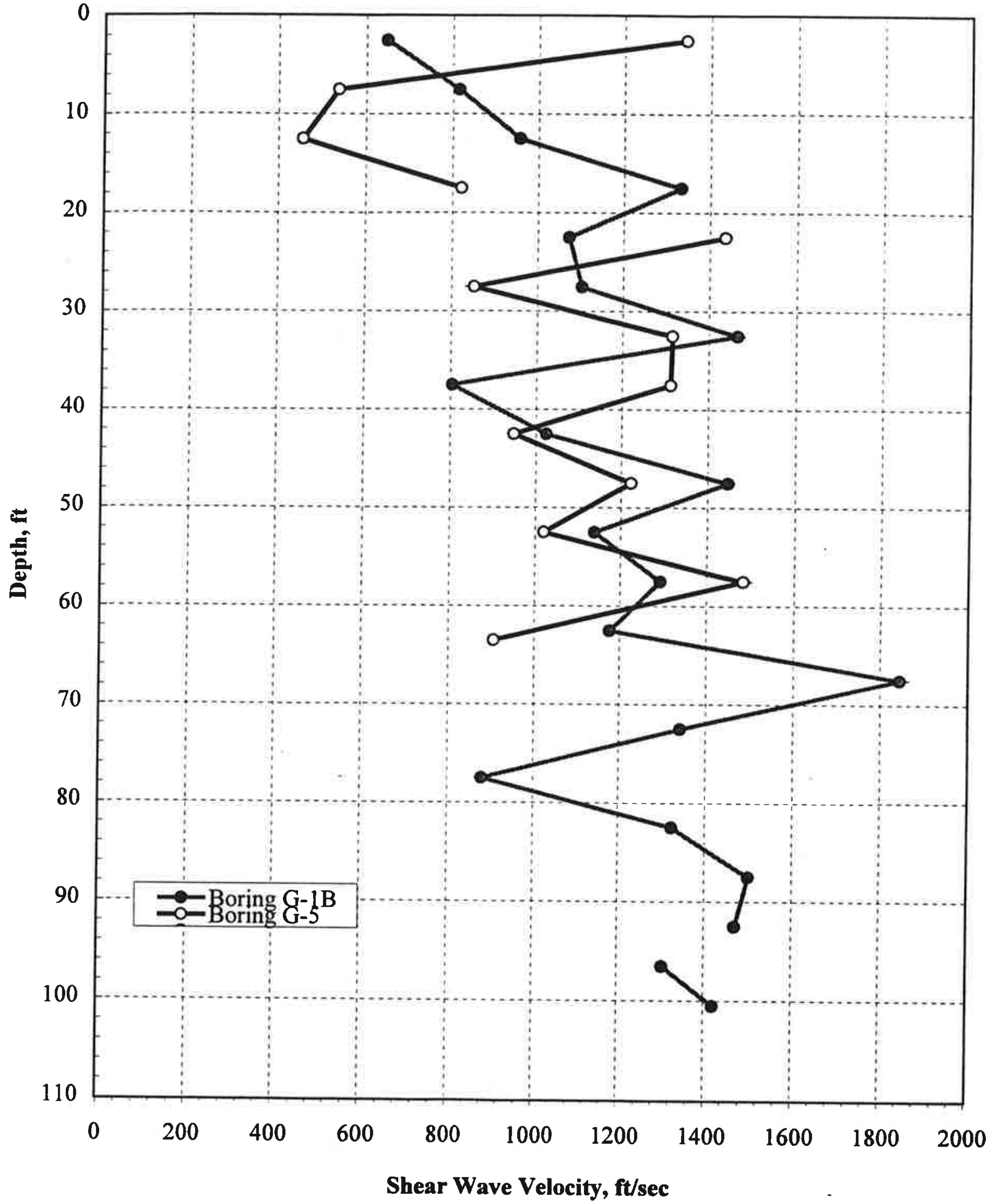
GEOSYNTEC CONSULTANTS

ATLANTA, GEORGIA

FIGURE NO.	2-1
PROJECT NO.	GL0625-15
DOCUMENT NO.	GA981181
FILE NO.	Plan.cdr

Shear Wave Velocity Profile

SHEAR WAVE VELOCITY PROFILES



GEOSYNTEC CONSULTANTS

ATLANTA, GEORGIA

FIGURE NO.	2-4
PROJECT NO.	GL0625-15
DOCUMENT NO.	GA981181
FILE NO.	FIGS.cdr/df

Summary of Standard Penetration Test, Triaxial Shear Test, and Index Property Test Results

TABLE 2-1

SUMMARY OF OCTOBER 1998 GEOSYNTEC FIELD INVESTIGATION PROGRAM

Boring No.	**Drilling**				**Sampling**				**Instrumentation and Additional Testing**	
	Location (Figure 2-1)	**Total Depth**	**Method**	**Terminate**	**Approximate Sequence**	**No. Shelby Tubes**	**No. Pitcher Barrel**	**No. SPT Tests**	**Piezometers**	**D-hole Shear Wave**
G-1A	Dam centerline (offset 10 ft from G-1B)	60 ft	8" bent. mud rotary	Within dam fill	None	0	0	0	1 in. PVC casing (2 installed)	
G-1B	Dam centerline	114 ft	8" bent. mud rotary	At bedrock surface	SPT - 5' intervals Tubes - 20' intervals	4-shell	1-shell 1-core	12-shell 2-core 1-saprolite	4 in. PVC casing (1 installed)	Within 4 in PVC casing
G-2	Dam centerline	68 ft	8" rotary	At bedrock surface	SPT - 5' intervals Tubes - 20' intervals	2-shell	3-shell	4-shell 1-saprolite	1 in. PVC casing (3 installed)	
G-3	115 ft west of dam centerline, above valley bottom	47 ft	HSA - 4.25" ID	Within dam fill	SPT - 5' intervals Tubes - 15' intervals	5-shell	0	6-shell	1 in. PVC casing (1 installed)	
G-4	235 ft west of dam centerline, above right abutment	55 ft	HSA - 4.25" ID (upper 30 ft) and 4" bent. mud rotary (lower 25 ft)	Within natural soil below dam fill	SPT - 5' intervals Tubes - 15' intervals	2-shell	3-shell	6-shell		
G-5	200 ft east of dam centerline, above left abutment	67 ft	8" bent. mud rotary	Within dam fill	SPT - 5' intervals Tubes - 15' intervals	5-core	1-core	2-shell 7-core		Within 4 in PVC casing

HSA = hollow stem auger, bent. = bentonite, PVC =polyvinyl chloride

TABLE 2-2

SUMMARY OF SPT N-VALUE CORRELATION TO EFFECTIVE STRESS FRICTION ANGLE

Material	φ' from N - Kulhaway and Mayne, 1990				φ' from $(N_1)_{60}$ - Hatanaka and Uchida, 1996			
Boring	no. tests	minimum	average.	st. deviation	No. tests	minimum	average.	st. deviation
Shell								
G-1B	14	38	41	1	14	38	41	2
G-2	2	36	39	3	2	37	39	3
G-3	5	38	42	3	7	37	40	2
G-4	5	37	40	3	7	35	38	2
G-5	-	-	-	-	1	43	43	-
	total 26	weighted avg. 37.7	weighted avg. 40.8	range 1 to 3	total 31	weighted avg. 37.2	weighted avg. 40.0	range 2 to 3
Core								
G-1B	4	34	35	1	4	35	36	1
G-5	14	29	34	3	14	31	35	2
	total 18	weighted avg. 30.1	weighted avg. 34.2	range 1 to 3	total 18	weighted avg. 31.9	weighted avg. 35.2	range 1 to 2
Saprolite								
G1-B	2	44	42	0	2	44	44	0

TABLE 3-1

LABORATORY TESTING RESULTS

SPECIMEN IDENTIFICATION				TRIAXIAL SHEAR TESTING							INDEX PROPERTY TESTING						
				Specimen Initial Conditions			Peak Strength Condition		Ultimate Strength Condition		Atterberg Limits		Grain Size Analysis (percent)				USCS Class.
Test No.	Boring No.	Sample Depth (ft)	Core or Shell Material	Water Content (%)	Dry Unit Weight (pcf)	Effective Consolidation Stress(1) (psi)	Deviator Stress(2) (psi)	Pore Pressure(3) (psi)	Deviator Stress(2) (psi)	Pore Pressure(3) (psi)	Liquid Limit	Plasticity Index	gravel	sand	silt	clay	
A	G-4	47-50	shell	25.9	103.1	41.5	79.0	17.5	113.1	1.6	NP	NP	12	58	23	7	SM
B	G-4	15-16	shell	17.7	97.9	13.6	78.0(4)	-4.0	81.3(4)	-7.3							
C	G-4	30-32	shell	27.8	97.2	27.2	55.0	14.0	101.2	-8.6							
D	G-1B	20-22	shell	19.1	103.5	18.3	34.5	8.5	48.6	0.3							
E	G-1B	38-40	shell	19.8	104.8	25.7	51.0	10.5	88.3	-7.5	33	3	7	49	41	3	SM
F	G-1B	80-81.5	shell	16.5	108.1	56.5	112.0	24.5	162.6	-7.1	NP	NP	3	61	34	2	SM
G	G-1B	105-107	core	20.7	109.3	68.9	104.0	39.5	165.3	4.0	41	9	4	44	42	10	ML
H	G-5	27-30	core	17.5	114.4	21.0	40.0	10.5	84.8	-8.1	33	9	6	42	35	17	ML
I	G-5	13-15	shell	24.2	105.1	12.9	30.5	4.5	63.6	-9.0							
J	G-5	60-62	core	22.0	104.8	40.9	64.5	24.0	97.8	6.5	45	15	2	40	40	18	ML
K	G-3	15-17	shell	22.5	107.4	13.7	28.0	60.0	63.3	-7.9							
L	G-3	28-30	shell	24.1	98.5	19.8	35.5	10.5	60.7	-0.6							
M	G-2	18-20	shell	23.8	98.3	10.4	26.0	3.5	55.3	-8.1							
N	G-2	38-40	shell	18.7	106.5	27.3	47.0	15.5	81.7	-1.1							
O	G-2	58-60	shell	21.6	106.0	42.6	58.0	25.5	84.7	11.4							
P	G-1B	20-22	shell	16.9(5)	102.8(5)	18.3(6)	49.0	5.0	87.7	-12.7							

Notes:
(1) Effective consolidation stress was achieved using back pressures ranging from 49 to 79 psi.
(2) Deviator stress is equal to the vertical stress applied to the specimen during shearing.
(3) Reported pore pressure is the change in pore water pressure during shearing.
(4) During this test excess friction developed in the loading system and reported deviator stresses are believed to be larger than actual values.
(5) Test performed on recompacted material.
(6) Test specimen initially consolidated to an effective stress of 23.8 psi, then overconsolidated to an effective stress of 18.3 psi.

Boring Logs

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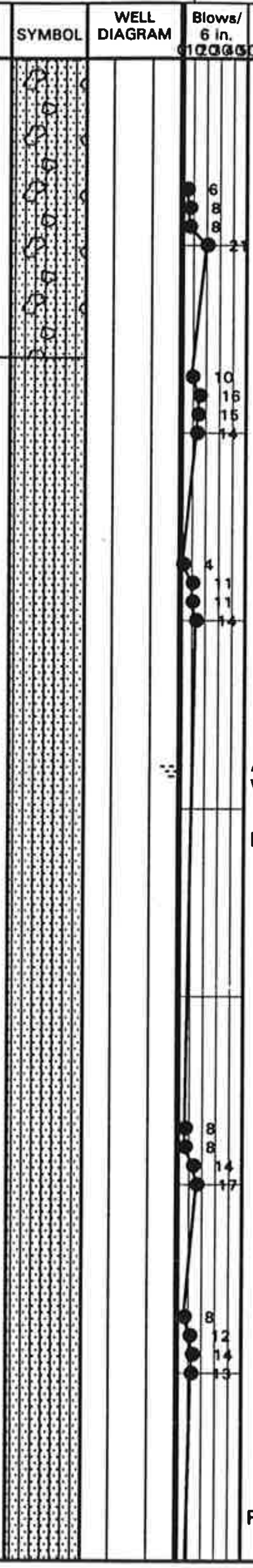
TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-1B**
LOCATION: **G-1**	N:	E:	GROUND ELEV.: **1627.0**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**			LOGGED BY: **J.Titus**
DATE: STARTED- **6 Oct 98**	COMPLETED- **12 Oct 98**		CHECKED BY: **G. Schmertmann**

ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	DRILLING LOG
1627	0		Begin Boring at 09:50hrs.
		SILT, micaceous, with coarse gravel, trace fine grained sand. Color: yellowish red (5YR4/6)	
1622	5		
		SILT and fine to medium sand, some clay @ 9.75 -10 feet. Weathered gneiss fragments sampled as fine grained sand.	
1617	10		
		SILT, micaceous, trace very fine sand. Color yellowish red (5YR5/8) Some coarse gravel (gneiss fragments) and trace organics (root) encountered @ 14-15 feet	
1612	15		
			Attempt shelby tube. Would not push (rock)
1607	20		
			Push shelby tube, 16" recovery
1602	25		
		SILT and very fine grained sand, micaceous. Color: dark reddish brown to very dark gray.	
1597	30		
		SILT, trace very fine sand, occassional lenses of weathered gneiss sampling as medium sand, trace organic material (bark/root)	
1592	35		
			Push shelby tube, 16" recovery
1587	40		



REMARKS:
3-WELL PIEZOMETER CLUSTER CONSTRUCTED AS FOLLOWS:
SHALLOW - 1-INCH PVC CASING SCREENED @ 20-40
MIDDLE - 1-INCH PVC CASING SCREENED @ 55-60
DEEP - 4-IN. PVC CASING SCREENED @ 105.5-110.5

SEE ATTACHED FIGURE FOR CONSTRUCTION DETAILS

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-1B**
LOCATION: **G-1**	N:	E:	GROUND ELEV.: **1627.0**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**			LOGGED BY: **J.Titus**
DATE: STARTED- **6 Oct 98**	COMPLETED- **12 Oct 98**		CHECKED BY: **G. Schmertmann**

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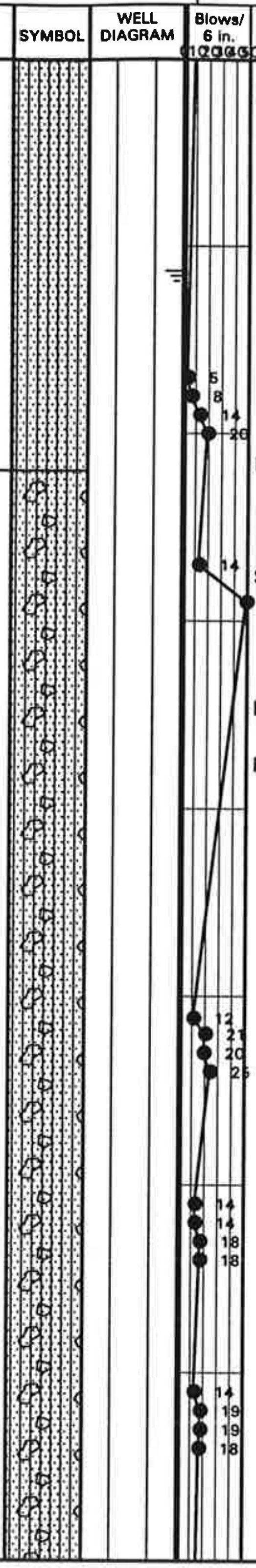

ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	Blows/ 6 in.	DRILLING LOG
1587	40			Push shelby tube, 16" recovery.
1582	45			
		SILT, trace very fine sand, micaceous (muscovite). Color: red (2.5YR4/8) and very dark gray (10YR3/1)	5 8 14 20	
1577	50			
		SILT, some very fine sand, some medium to coarse gravel (weathered gneiss and schist fragments).		Hard drilling @ 51-52 feet.
			14 50/5	Split spoon bouncing on wood
1572	55			
				Hard drilling (rock) @ 57.5-58 feet
				Push shelby tube, 16" recovery
1567	60			
1562	65	SILT, some very fine sand, some medium to coarse gravel (weathered gneiss and schist). Increasing size and number with depth in the spoon.	12 21 20 25	
1557	70		14 14 18 18	
1552	75	SAND, very fine to fine grained, and silt, some fine to medium gravel (weathered gneiss and schist fragments). Silty clay in end of spoon.	14 19 19 18	
1547	80			

REMARKS:

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**	BORING ID: **G-1B**
LOCATION: **G-1**	N: E:	GROUND ELEV.: **1627.0**
DRILLING CO.: **AT&E**	RIG: **CME 750**	DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**		LOGGED BY: **J.Titus**
DATE: STARTED- **6 Oct 98**	COMPLETED- **12 Oct 98**	CHECKED BY: **G. Schmertmann**

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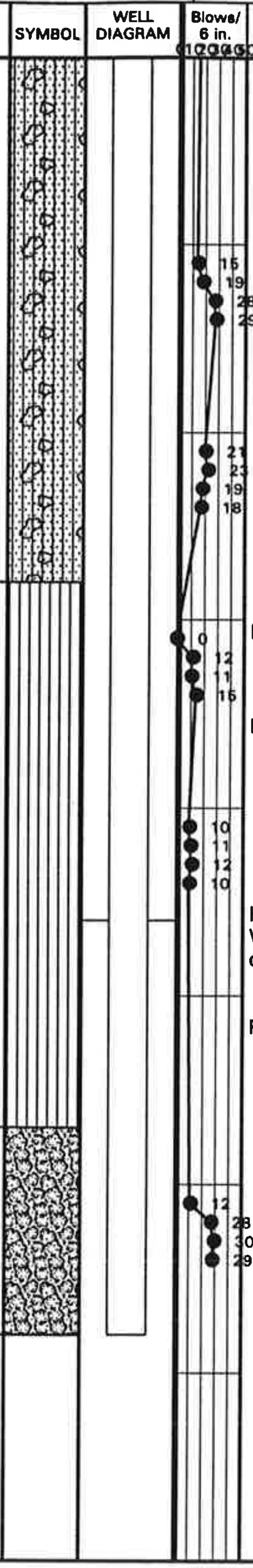
ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	Blows/ 6 in.	DRILLING LOG
1547	80			Pitcher barrel sample, 16" recovery.
1542	85	SILT, some very fine sand, micaceous. Trace gneiss gravel in end of spoon.	15 19 28 29	
1537	90	SILT and very fine sand, some weathered schist and gneiss fragments (1/2-1 inch. diam). Lower 5 inches of spoon has strong banding of mafics, quartz, and feldspars. Extensively weathered.	21 23 19 18	
1532	95	SILT, trace clay, micaceous. Mottled slightly. Color: dark red (2.5YR3/6) and yellowish brown (10YR5/8).	0 12 11 16	Drop by weight of rods Drilling becoming much harder
1527	100	SILT, some clay, micaceous. Color is very dark grayish brown (10YR3/2) @ 100-101 feet and red (2.5YR4/8) @ 101-102 feet.	10 11 12 10	Hit rock while drilling past 102.5. Wood fragment came out of hole while drilling past 103 feet.
1522	105			Pitcher barrel sample, 11" recovery.
		Saprolite		
1517	110		12 28 30 29	
1512	115	Boring terminated at 114.00 feet		
1507	120			



REMARKS:

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-2**
LOCATION: **G-2**	N:	E:	GROUND ELEV.: **1584.8**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**			LOGGED BY: **J.Titus**
DATE: STARTED- **14 Oct 98**	COMPLETED- **15 Oct 98**		CHECKED BY: **G. Schmertmann**

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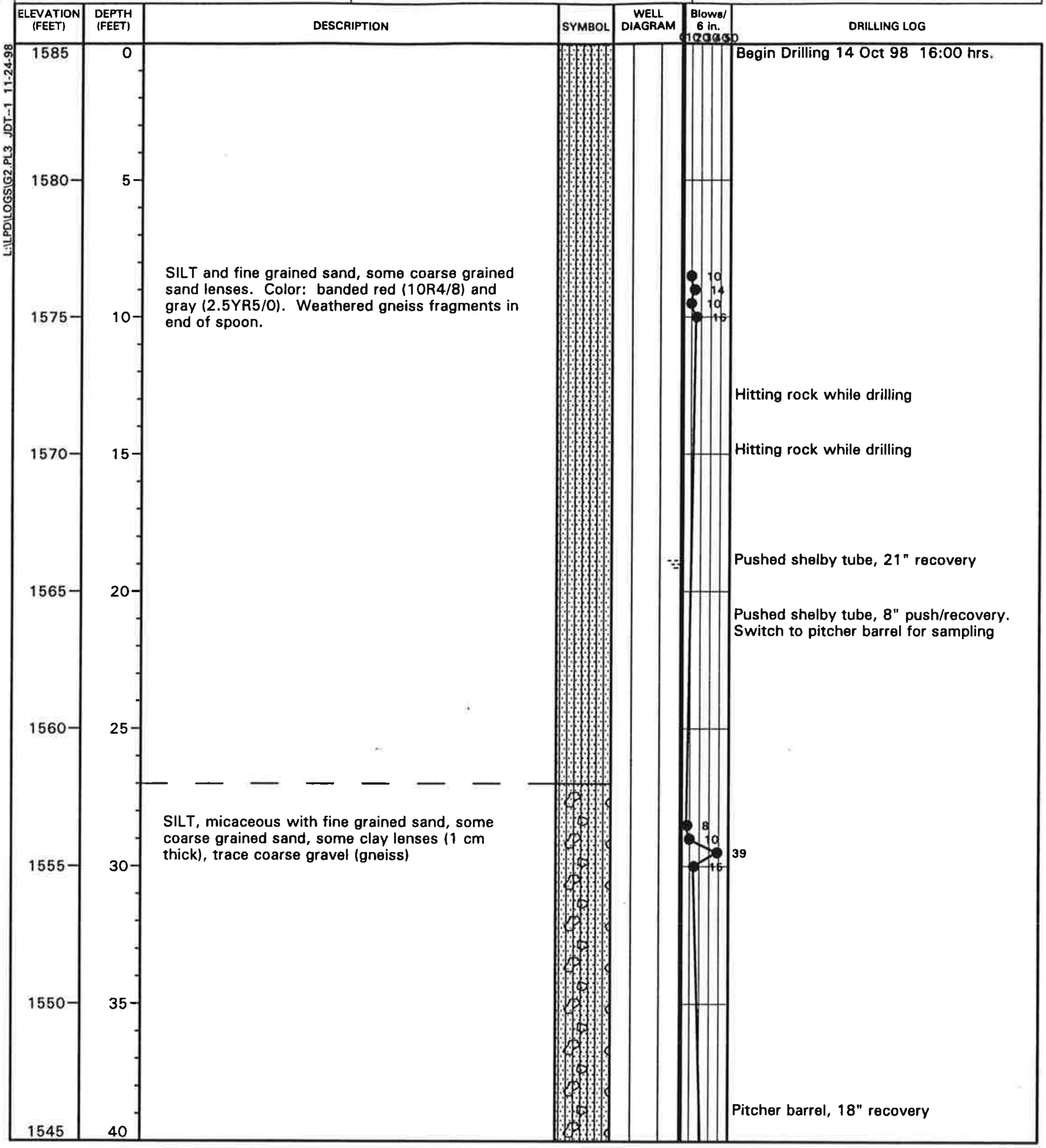


SILT and fine grained sand, some coarse grained sand lenses. Color: banded red (10R4/8) and gray (2.5YR5/0). Weathered gneiss fragments in end of spoon.

SILT, micaceous with fine grained sand, some coarse grained sand, some clay lenses (1 cm thick), trace coarse gravel (gneiss)

Begin Drilling 14 Oct 98 16:00 hrs.

Hitting rock while drilling

Hitting rock while drilling

Pushed shelby tube, 21" recovery

Pushed shelby tube, 8" push/recovery. Switch to pitcher barrel for sampling

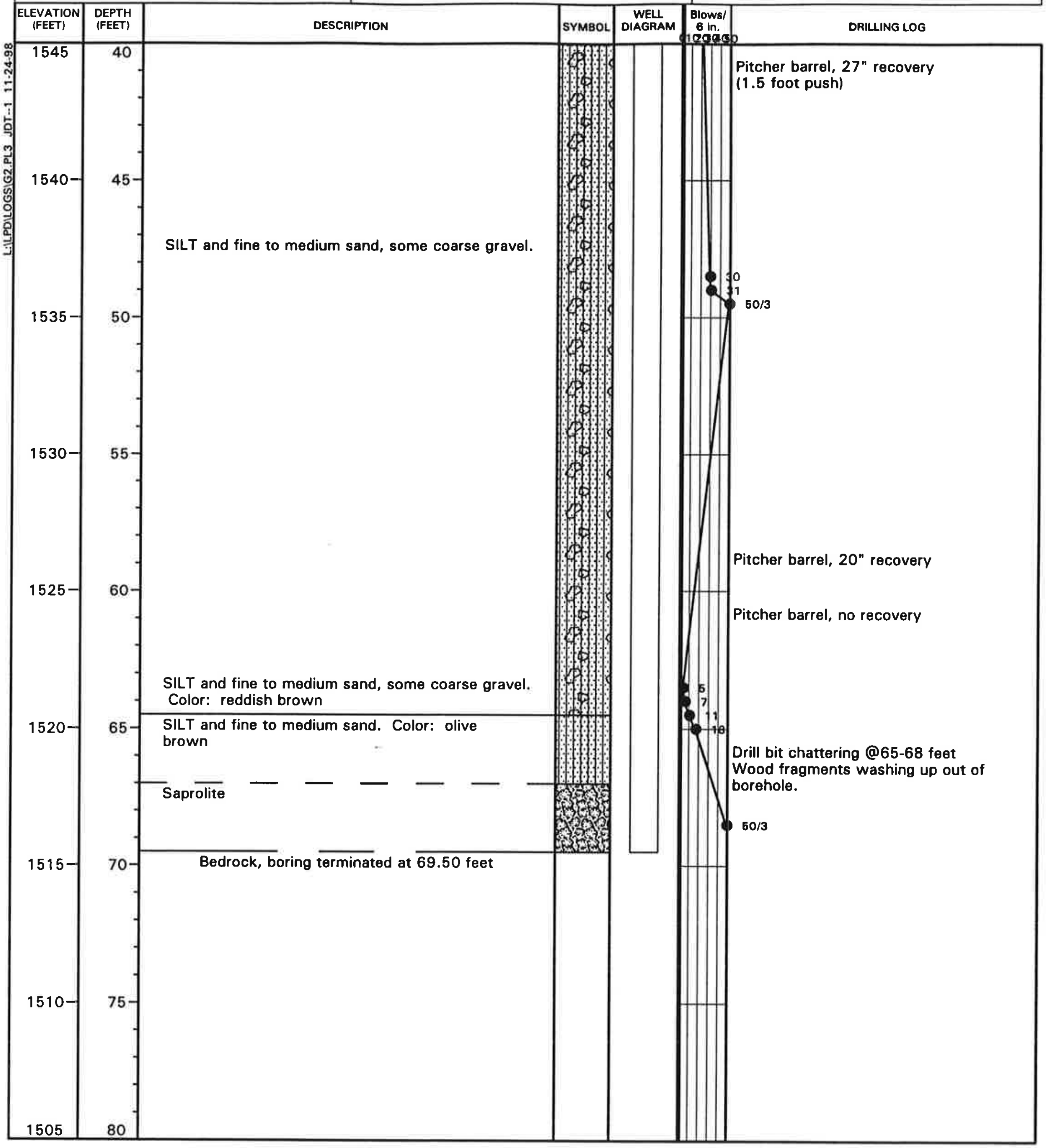
Pitcher barrel, 18" recovery

REMARKS:
3-WELL PIEZOMETER CLUSTER CONSTRUCTED AS FOLLOWS:
SHALLOW - 1-IN. PVC CASING SCREENED @ 10-30
MIDDLE - 1-IN. PVC CASING SCREENED @ 50-55
DEEP - 1-IN. PVC CASING SCREENED @ 65.5-68.5

SEE ATTACHED FIGURE FOR CONSTRUCTION DETAILS

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-2**
LOCATION: **G-2**	N:	E:	GROUND ELEV.: **1584.8**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**			LOGGED BY: **J.Titus**
DATE: STARTED- **14 Oct 98**	COMPLETED- **15 Oct 98**		CHECKED BY: **G. Schmertmann**



SILT and fine to medium sand, some coarse gravel.

SILT and fine to medium sand, some coarse gravel. Color: reddish brown

SILT and fine to medium sand. Color: olive brown

Saprolite

Bedrock, boring terminated at 69.50 feet

Pitcher barrel, 27" recovery (1.5 foot push)

Pitcher barrel, 20" recovery

Pitcher barrel, no recovery

Drill bit chattering @65-68 feet
Wood fragments washing up out of borehole.

REMARKS:

TEST BORING RECORD

PAGE 1 OF 1

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**	BORING ID:	**G-3**
LOCATION: **G-3**	N:10281.952 E:9904.224	GROUND ELEV.:	**1564.03**
DRILLING CO.: **AT&E**	RIG: **CME 750**	DRILLER:	**P. Bergman**
METHOD & DIAMETER: **4-1/4 ID HSA**		LOGGED BY:	**J.Titus**
DATE: STARTED- **13 Oct 98**	COMPLETED- **13 Oct 98**	CHECKED BY:	**G. Schmertmann**

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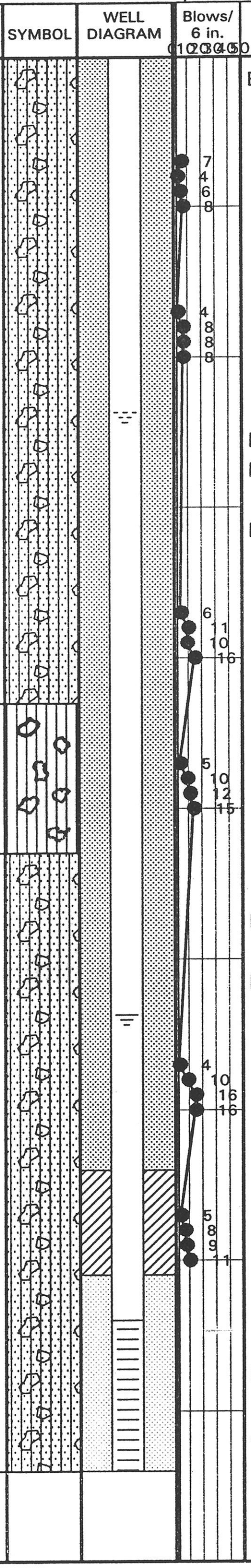
ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	DRILLING LOG
1564	0		Begin drilling 13 Oct 98: 1545 hrs.
1559	5	SILT, micaceous, trace very fine sand, trace coarse gravel (gneiss). Color: yellowish brown (10YR5/4). Gneiss weathers to fine sand. Color is banded gray and white.	
1554	10	Occasional lenses of silty clay. Color red (2.5YR5/8)	Hit water while drilling Pushed shelby tube, 18" recovery
1549	15		Pushed shelby tube, 22" recovery
1544	20	Becoming more banded in texture. Gneiss weathers to fine to medium sand.	
1539	25	SILT and coarse gravel (quartz and gneiss)	
1534	30		Pushed shelby tube, 23" recovery Pushed shelby tube, 12" recovery
1529	35	SILT, micaceous, trace medium sand (quartz), trace very fine sand. Coarse gravel (gneiss) in end of spoon.	
1524	40		
1519	45		Pushed shelby tube, 16" recovery Attempted shelby tube, would not push.
		Boring terminated at 47.00 feet	
1514	50		



REMARKS:
SHALLOW PIEZOMETER SCREENED IN FILL MATERIAL

CONTRUCTION: 1-INCH ID PVC PIPE WITH 5-FEET OF 0.010-INCH SLOTTED SCREEN.

GeoSyntec Consultants

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-4**
LOCATION: **G-4**	N:	E:	GROUND ELEV.: **1605.8**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **HSA/4" Mud Rotary**			LOGGED BY: **GS / JDT**
DATE: STARTED- **2 Oct 98**	COMPLETED- **5 Oct 98**		CHECKED BY: **G. Schmertmann**

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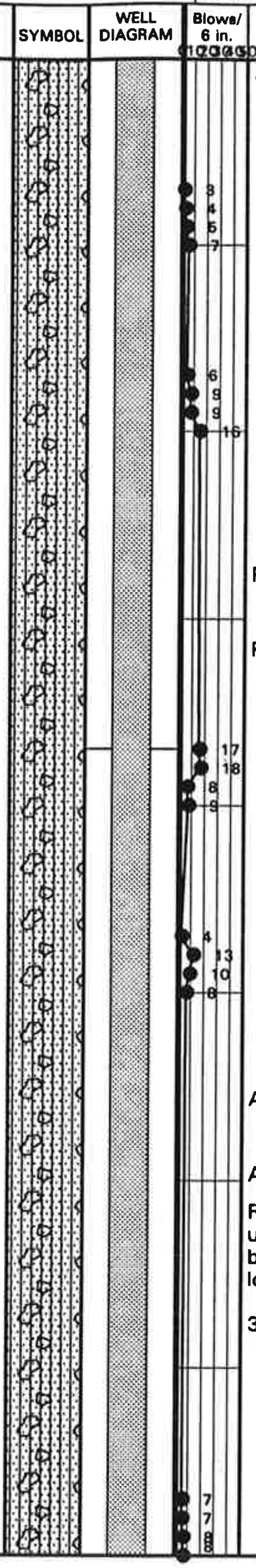

ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	DRILLING LOG
1606	0		2OCT98 Beging drilling using 4-1/4 ID HSA
		SILT, some sand. Color: brown	
1601	5		
		SILT, some sand, some medium gravel, micaceous, dry	
1596	10		
			Push shelby tube, 15" recovery
1591	15		Push shelby tube, 7" recovery
		SILT, some sand, some gravel. Medium gravel (weathered gneiss and schist) concentrated in upper 6" spoon, more silt in lower 9". dry. Color: dark brown.	
1586	20		
1581	25		
			Attempted shelby tube, would not push
1576	30		Attempted shelby tube, would not push Resume drilling on 5OCT98 at 10:45 hrs using 4-3/4 OD mud rotary. Boring has been offset by 5 feet from original location. 30-32 ft. - Pitcher barrel sample
1571	35		
		SILT, some sand. Trace gravel in upper 3" of spoon. micacoeus, dark brown.	
1566	40		

REMARKS:

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**	BORING ID: **G-4**
LOCATION: **G-4**	N: E:	GROUND ELEV.: **1605.8**
DRILLING CO.: **AT&E**	RIG: **CME 750**	DRILLER: **P. Bergman**
METHOD & DIAMETER: **HSA/4" Mud Rotary**		LOGGED BY: **GS / JDT**
DATE: STARTED- **2 Oct 98**	COMPLETED- **5 Oct 98**	CHECKED BY: **G. Schmertmann**

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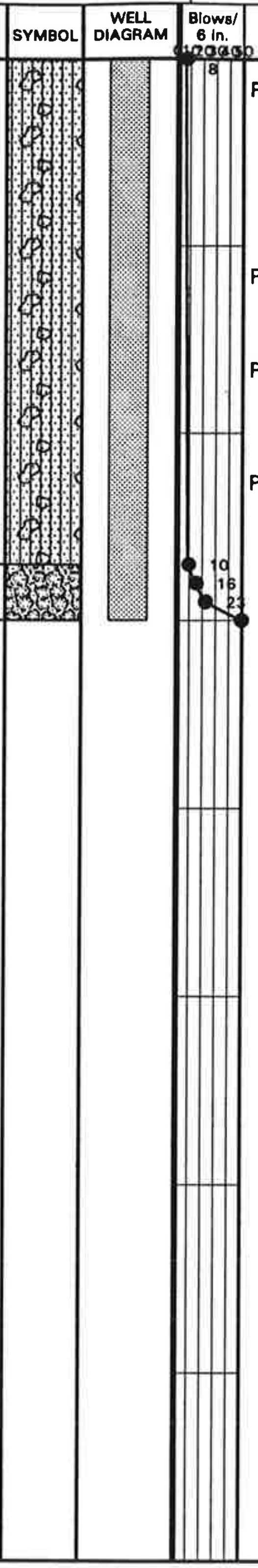


ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	DRILLING LOG
1566	40		Push shelby tube, 5" recovery.
1561	45	SILT, some sand, some fine to meduim gravel, micaceous	Push shelby tube, no recovery
			Pitcher barrel sample, 20" recovery
1556	50		Pitcher barrel sample, 8" recovery
		SILT, some sand, trace gravel (FILL)	
		Saprolite	
1551	55	Boring terminated at 55.00 feet	
1546	60		
1541	65		
1536	70		
1531	75		
1526	80		

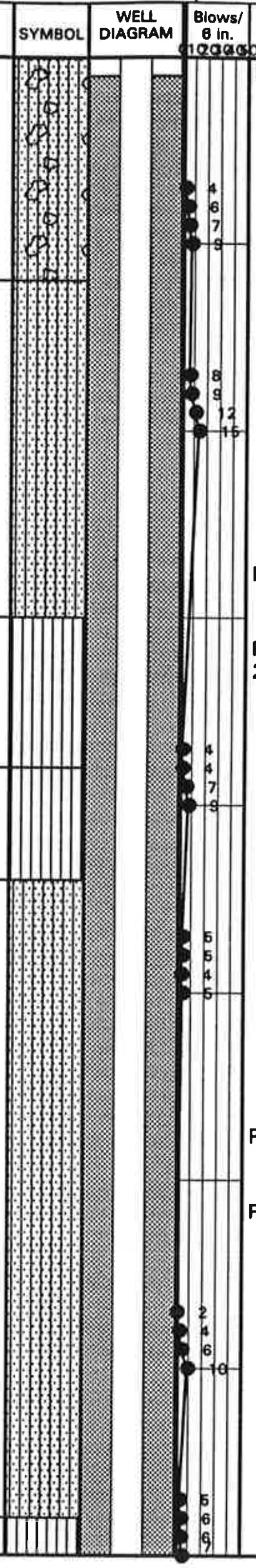
REMARKS:

TEST BORING RECORD

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-5**
LOCATION: **G-5**	N:	E:	GROUND ELEV.: **1646.72**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**			LOGGED BY: **J.Titus**
DATE: STARTED- **12 Oct 98**	COMPLETED- **14 Oct 98**		CHECKED BY: **G. Schmertmann**

L:\LPD\LOGS\G5.PL3 JDT-1 11-24-98

ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	DRILLING LOG
1647	0		Begin drilling on 12OCT98: 13:50 hrs.
		SILT some fine to medium sand, some fine to medium gravel. Dry, Color: brown (7.5YR4/4)	
1642	5		
		SILT, some very fine to fine sand, micaceous. Color: banded strong brown (7.5YR5/6) and dark gray (7.5YR4/0).	
1637	10		
			Push shelby tube, 17" recovery
1632	15		Pushed shelby tube 1 foot, 24" recovery (wash out)
		SILT, trace very fine sand. Color: dark gray to very dark gray (10YR4/1 - 3/1)	
		SILT, trace clay, micaceous, Color: red (10R4/8) Extremely weathered schist (to silt) in end of spoon.	
1627	20		
1622	25	SILT, some very fine sand, micaceous. Trace clay in lower 6" of spoon.	
			Push shelby tube, 19" recovery
1617	30		Push shelby tube, no recovery
		SILT, some fine to medium quartz sand, trace clay, micaceous. Color: red (2.5YR4/8).	
1612	35		
		SILT and sand (weathered gneiss)	
1607	40	SILT, and clay. Wood fragments at 29 feet.	



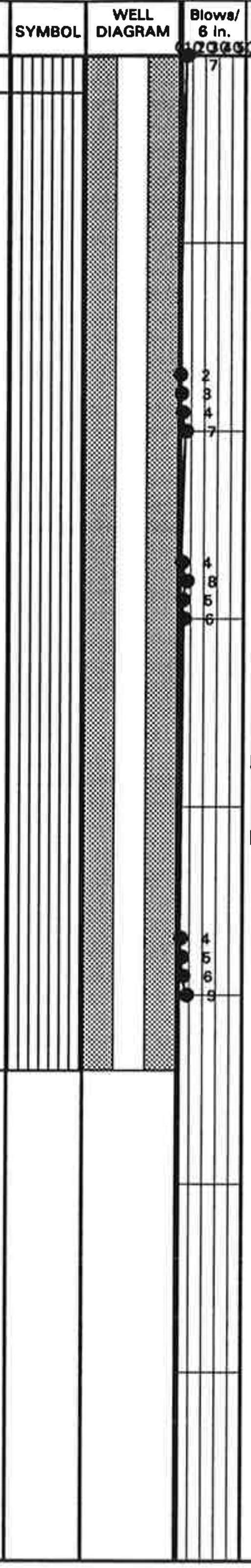
REMARKS:
Blank casing installed (no screen) for downhole geophysics applications.

TEST BORING RECORD

PAGE 2 OF 2

PROJECT NAME: **Lake Petit Dam**	PROJECT NO.: **GL0625**		BORING ID: **G-5**
LOCATION: **G-5**	N:	E:	GROUND ELEV.: **1646.72**
DRILLING CO.: **AT&E**	RIG: **CME 750**		DRILLER: **P. Bergman**
METHOD & DIAMETER: **Mud Rotary (8-in.)**			LOGGED BY: **J.Titus**
DATE: STARTED- **12 Oct 98**	COMPLETED- **14 Oct 98**		CHECKED BY: **G. Schmertmann**

L:\LPD\LOGS\G5.PL3 JDT-1 11-24-98



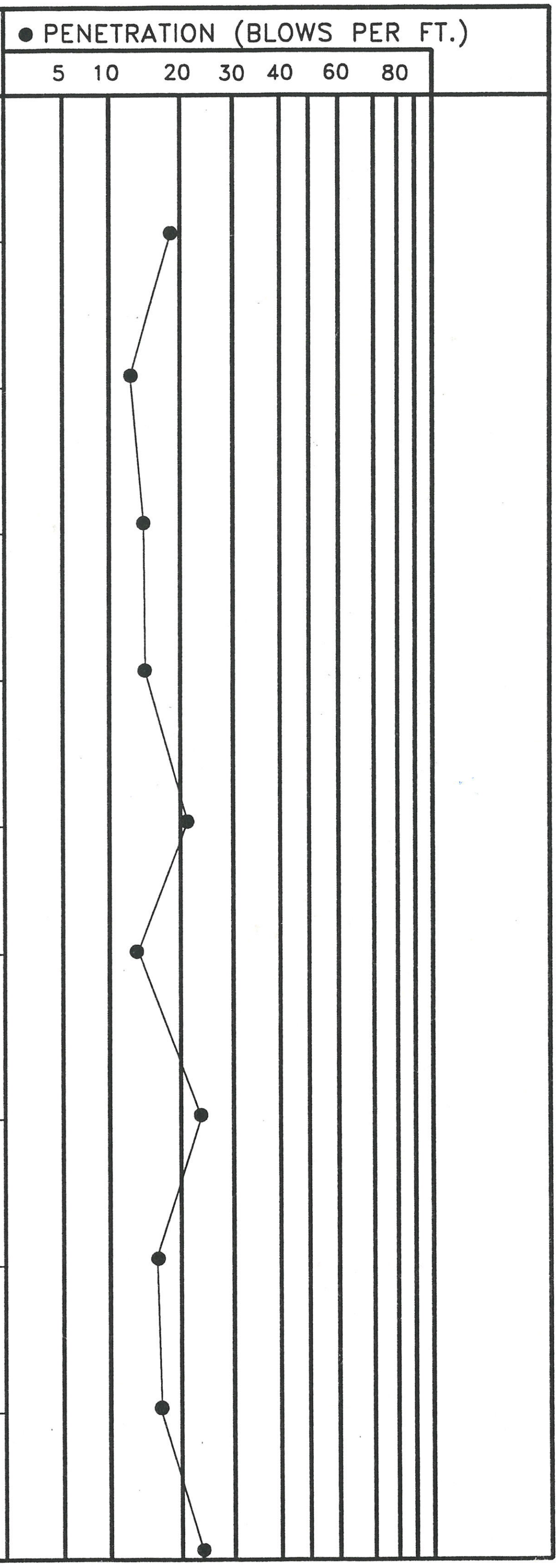
ELEVATION (FEET)	DEPTH (FEET)	DESCRIPTION	Blows/ 6 in.	DRILLING LOG
1607	40		7	Wood debris washing up out of borehole
				Push shelby tube, 15" recovery
1602	45			Pushed shelby tube 6". No recovery. wood debris in end of tube
		SILT, micaceous, trace clay, trace very fine sand, trace wood/roots. Color: red (10R4/8)	2 3 4 7	
1597	50			
		Increasing wood fragments up to 1" diam.	4 8 5 6	
1592	55			
				Push shelby tube, 22" recovery
1587	60			Push shelby tube, 14" recovery
		SILT, some clay, trace very fine sand. Color: @ 63-64.5 - red (10R4/8) @ 64.5-65 - dark gray (5YR4/1)	4 5 6 9	
1582	65			
		Boring terminated at 67.00 feet		
1577	70			
1572	75			
1567	80			

REMARKS:

logs of PGC hole

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.

DEPTH (FT.)	DESCRIPTION		ELEV. 1647
	FILL: Medium dense gray brown medium to fine SAND (SM), with rock fragments		
5			1642
	Stiff red gray brown fine sandy SILT (ML-SM), trace clay, topsoil, with rock fragments NOTE: Large rock encountered in borehole from 12.5 to 13.5 feet		
10			1637
		■	
15			1632
	Stiff to very stiff red brown clayey fine sandy SILT (ML-SM), with topsoil, organics and rock fragments NOTE: Rock encountered in borehole from 20 to 25 feet. Wood pieces in return mud.		
20			1627
25			1622
		■	
	Stiff red brown silty CLAY (CL), with topsoil, organics and rock fragments		
30			1617
	Very stiff red brown fine sandy clayey SILT (ML-CL), with topsoil and organics		
35			1612
	Topsoil with organics		1607
40	Very stiff red brown fine sandy clayey SILT (ML-CL), with trace topsoil, small organics, rock fragments		
		■	
45			1602
	Topsoil		1597



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH – 24 HRS.

GROUNDWATER LEVEL – 24 HRS.

■ UNDISTURBED SAMPLE

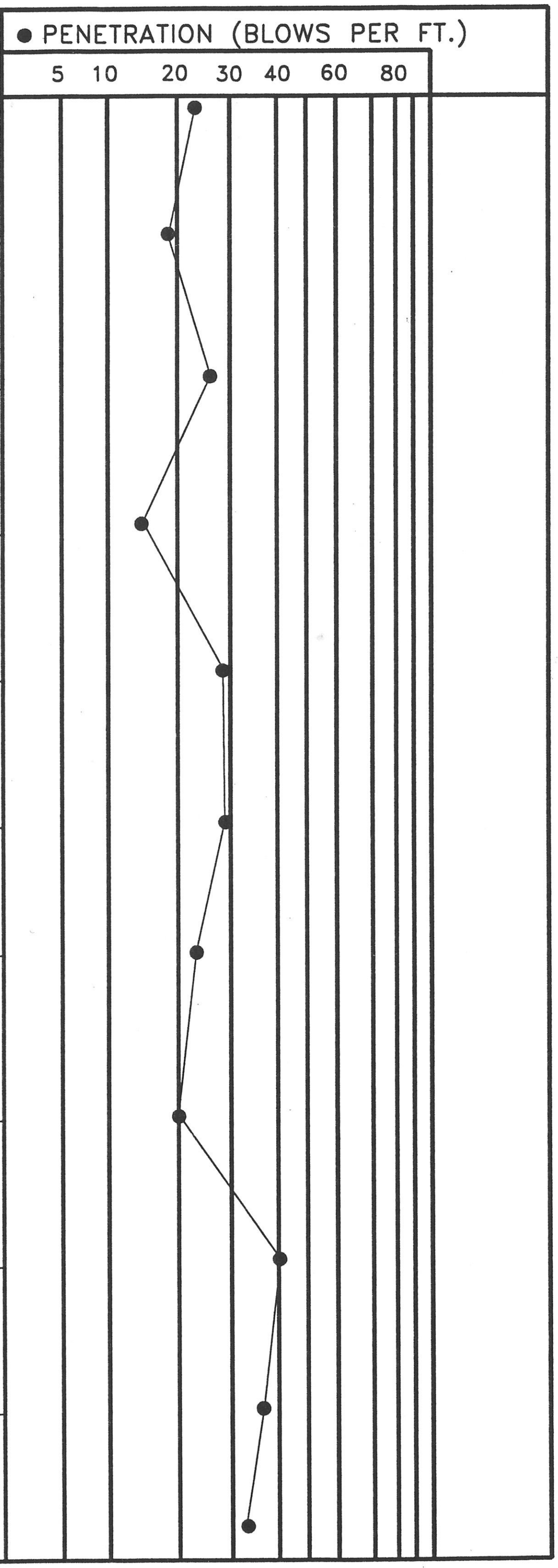
GROUNDWATER LEVEL AT TIME OF BORING

(XX) – DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-2
DATE DRILLED 10/7/97-10/9/97
JOB NO. 97089
PAGE 1 OF 3

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.

DEPTH (FT.)	DESCRIPTION	ELEV.
50	Very stiff red brown fine sandy clayey SILT (ML-CL) NOTE: Wood pieces in return mud stopping up supply hoses.	1597
55		1592
60	Very stiff red brown and gray fine sandy SILT (ML), trace clay, trace topsoil, small rock fragments	1587
65	NO SAMPLE RECOVERED FROM 63 TO 73 FEET NOTE: Wood pieces observed in mud return.	1582
70		1577
75	Large rock encountered in borehole from 76 feet to 77 feet.	1572
80	Very stiff tan brown clayey fine sandy SILT (ML-SM)	1567
85	Topsoil Very stiff red brown fine sandy clayey SILT (ML) NOTE: Temporarily lost circulation from 85-86 feet	1562
90	Dense gray brown silty medium to fine SAND (SM), with rock fragments **	1557
95	Hard red brown and gray clayey fine sandy SILT (ML-SM), with partially weathered rock fragments, topsoil, organics	1552
		1547



SOIL BORING RECORD

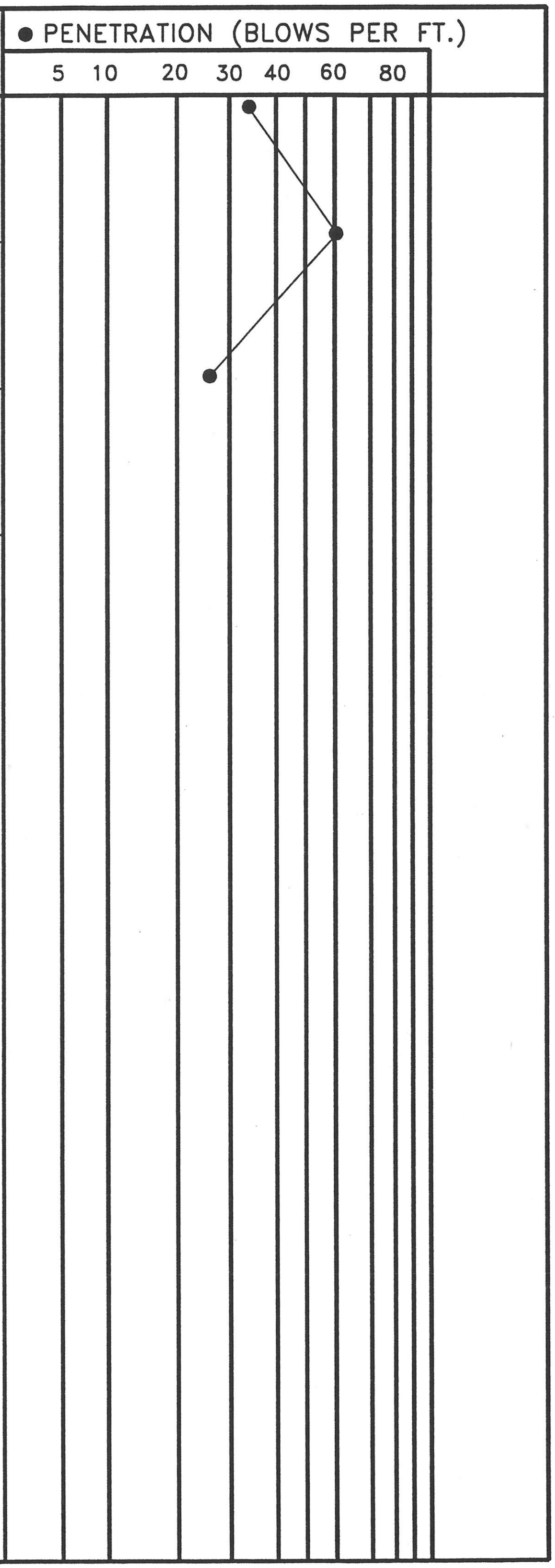
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH - 24 HRS.
▬ GROUNDWATER LEVEL - 24 HRS.
▮ UNDISTURBED SAMPLE
▬ GROUNDWATER LEVEL AT TIME OF BORING
(XX) - DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-2
DATE DRILLED 10/7/97-10/9/97
JOB NO. 97089
PAGE 2 OF 3

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.

DEPTH (FT.)	DESCRIPTION		ELEV.
100			1547
	Hard to very hard red brown and gray clayey fine sandy SILT (ML-SM) **	■	
	NOTE: Large rock encountered in borehole from 104 to 104.5 feet		
105			1542
	Very stiff brown fine sandy SILT (ML-SM), with rock fragments		
110			1537
	Approximate top of rock		
	Hard Drilling		
115	Boring terminated at 114 feet		1532
120			
	**SPT value amplified due to presence of rock.		
125			
130			
135			
140			
145			



SOIL BORING RECORD

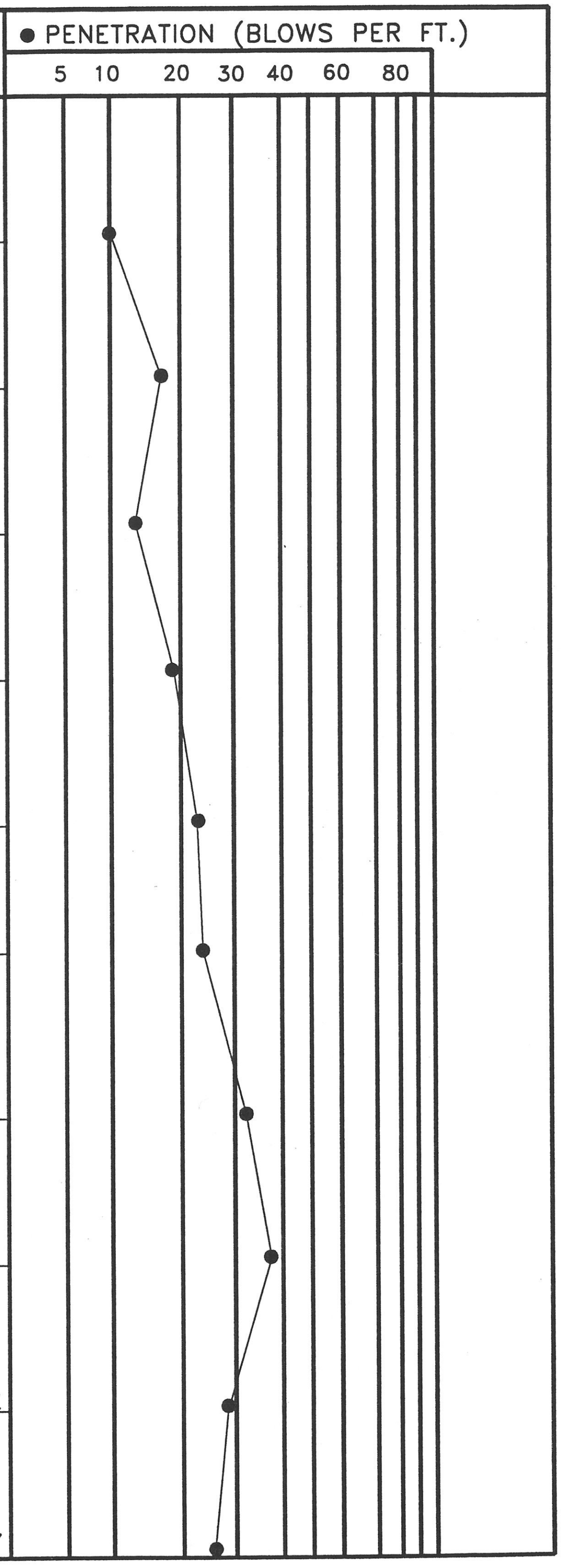
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH - 24 HRS.
GROUNDWATER LEVEL - 24 HRS.
■ UNDISTURBED SAMPLE
GROUNDWATER LEVEL AT TIME OF BORING
(XX) - DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-2
DATE DRILLED 10/7/97-10/9/97
JOB NO. 97089
PAGE 3 OF 3

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.

DEPTH (FT.)	DESCRIPTION		ELEV. 1607
	FILL: Loose tan brown gray silty fine SAND (SM), with rock fragments		
5			1602
	Medium dense tan brown silty fine SAND (SM), with rock fragments		
10			1597
		▮	
	Stiff to very stiff red tan brown fine sandy clayey SILT (ML)		
15			1592
20			1587
	Medium dense red brown to tan brown silty fine SAND (SM-ML), with rock fragments, trace topsoil		
25			1582
		▮	
30			1577
	Dense to medium dense red gray brown silty fine SAND (SM), with rock fragments		
35			1572
40			1567
		NR ▮	
45			1562
	NOTE: Large rock encountered in borehole from 46 to 48 feet.		
			1557



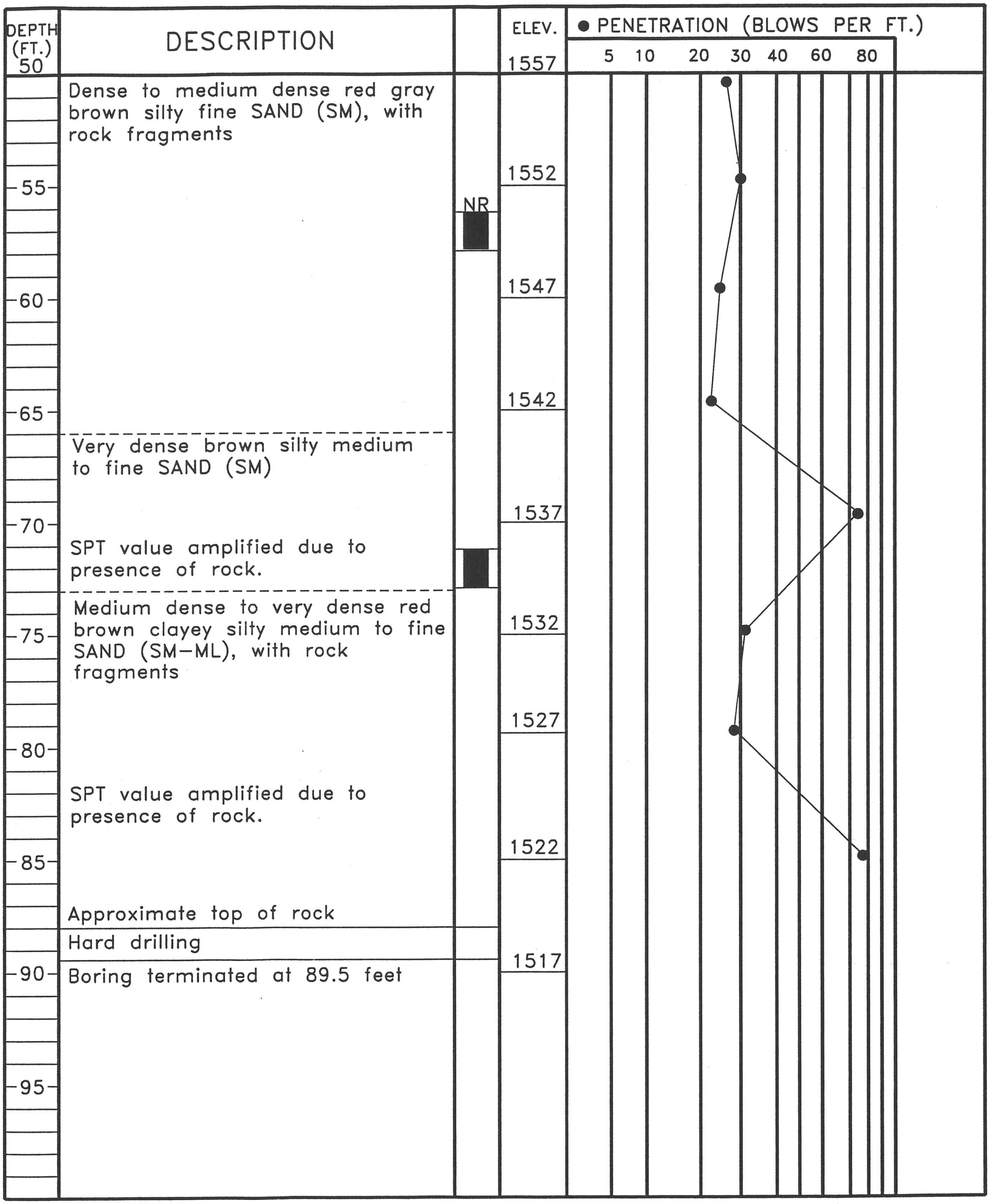
SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH - 24 HRS.
▮ UNDISTURBED SAMPLE
GROUNDWATER LEVEL - 24 HRS.
GROUNDWATER LEVEL AT TIME OF BORING
(XX) - DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-4
DATE DRILLED 10/2/97-10/6/97
JOB NO. 97089
PAGE 1 OF 2

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.



Dense to medium dense red gray brown silty fine SAND (SM), with rock fragments

Very dense brown silty medium to fine SAND (SM)

SPT value amplified due to presence of rock.

Medium dense to very dense red brown clayey silty medium to fine SAND (SM–ML), with rock fragments

SPT value amplified due to presence of rock.

Approximate top of rock

Hard drilling

Boring terminated at 89.5 feet

SOIL BORING RECORD

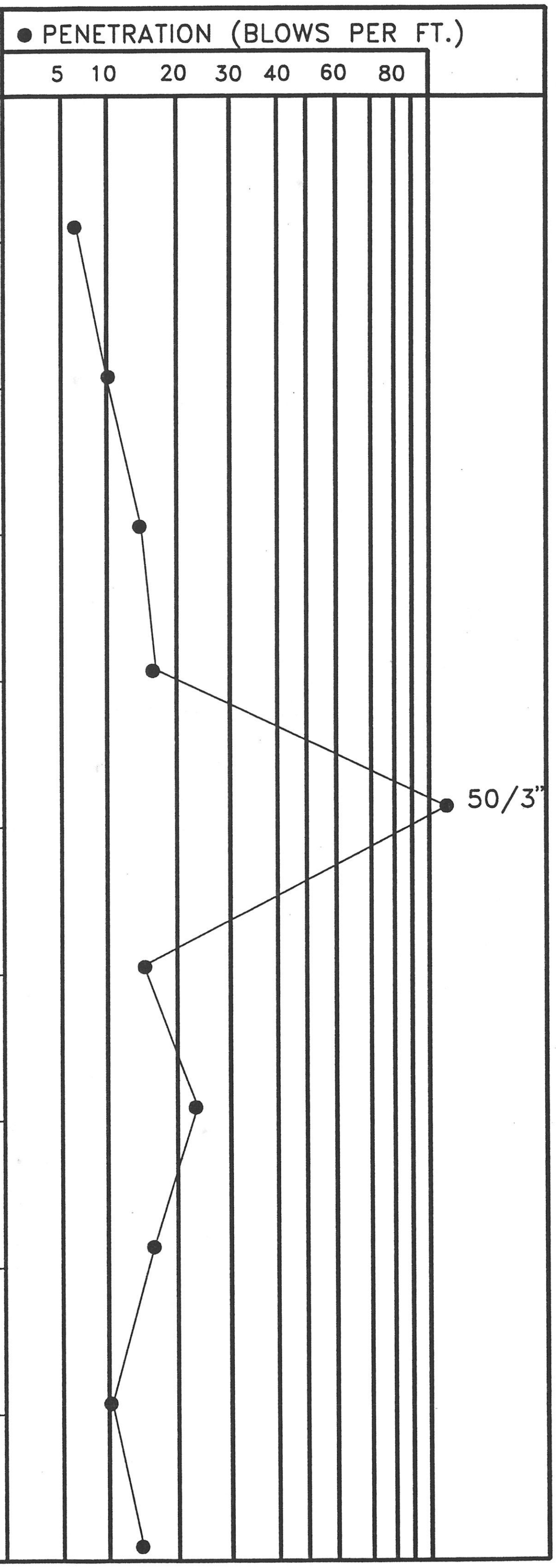
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH – 24 HRS.
GROUNDWATER LEVEL – 24 HRS.
UNDISTURBED SAMPLE
GROUNDWATER LEVEL AT TIME OF BORING
(XX) – DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-4
DATE DRILLED 10/2/97–10/6/97
JOB NO. 97089
PAGE 2 OF 2

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.

DEPTH (FT.)	DESCRIPTION		ELEV.
	FILL: Loose tan brown silty fine SAND (SM), with rock fragments		1562
5	Loose gray brown to tan brown silty medium to fine SAND (SM), with rock fragments		1557
10	Moist at 10 feet		1552
15	Medium dense red tan brown silty medium to fine SAND (SM), wet, with organics and rock fragments		1547
20	Becomes drier at 20 feet	▽	1542
25	Rock fragments at 23.5 feet No recovery Very wet between 25-30 feet		1537
30	Tan gray brown silty medium to fine SAND (SM), with rock fragments Drier at 30 feet		1532
35	Medium dense gray brown silty medium to fine SAND (SM), with rock fragments Very wet		1527
40			1522
45	Tan brown to red brown silty fine SAND (SM), and silty CLAY (CL), alternating seams 4" to 6" thick		1517
	Boring terminated at 50 feet		1512



SOIL BORING RECORD

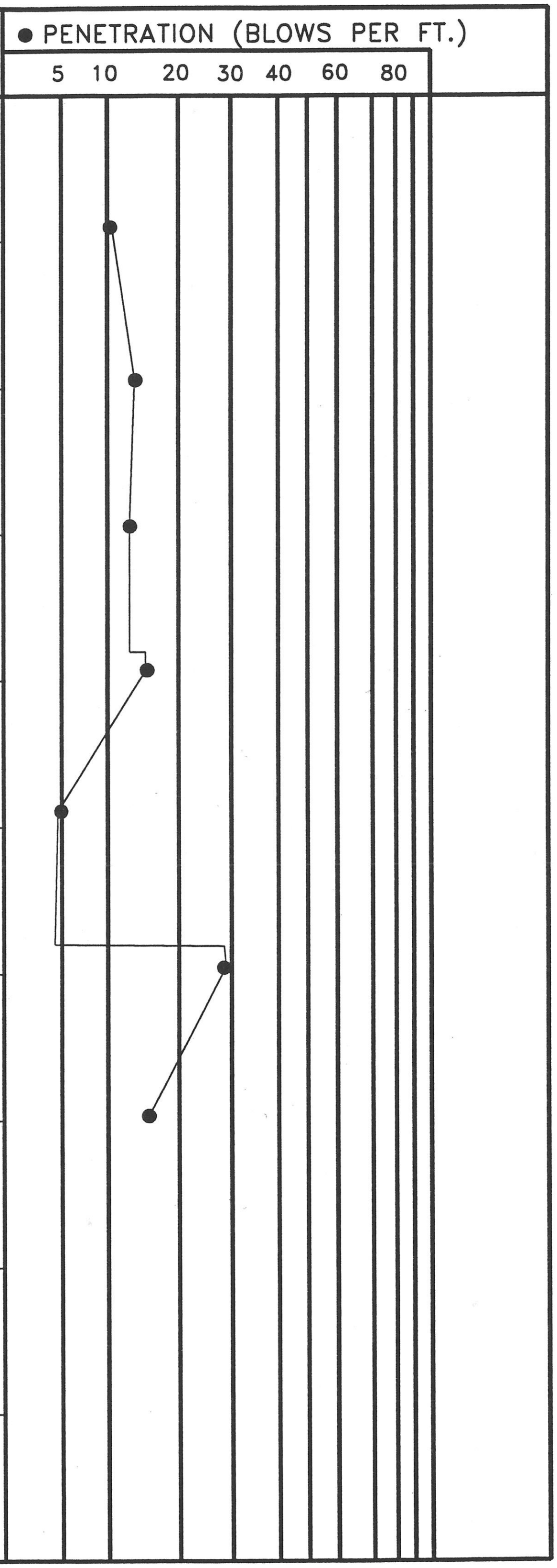
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH - 24 HRS.
▽ GROUNDWATER LEVEL - 24 HRS.
▮ UNDISTURBED SAMPLE
▽ GROUNDWATER LEVEL AT TIME OF BORING
(XX) - DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-6
DATE DRILLED 05/11/98
JOB NO. 97089
PAGE 1 OF 1

PIEDMONT GEOTECHNICAL CONSULTANTS, INC.

DEPTH (FT.)	DESCRIPTION		ELEV.
	FILL: Loose tan gray brown silty medium to fine SAND (SM), with rock fragments		1544
5			1539
10	Medium dense tan brown silty fine SAND (SM), with rock fragments		1534
15			1529
20	POSSIBLE ALLUVIUM: Medium dense dark gray brown silty medium to fine SAND (SM), with small organics (Harder drilling at 20.5–23.5 feet with wood pieces)		1524
25	Loose dark gray brown silty medium to fine SAND (SM), with organics (drill bit hitting possible large organics)		1519
30	RESIDUUM: Medium dense tan brown micaceous silty medium to fine SAND (SM)		1514
35	Boring terminated at 35 feet		1509
40			
45			



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LBS.HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

C CAVED DEPTH – 24 HRS.
GROUNDWATER LEVEL – 24 HRS.
UNDISTURBED SAMPLE
GROUNDWATER LEVEL AT TIME OF BORING
(XX) – DENOTES UNIFIED SOIL CLASSIFICATION SYSTEM

BORING NO. P-7
DATE DRILLED 05/12/98
JOB NO. 97089
PAGE 1 OF 1

Summary of Triaxial Compression Testing Results, Particle Size Distribution, and Physical Properties

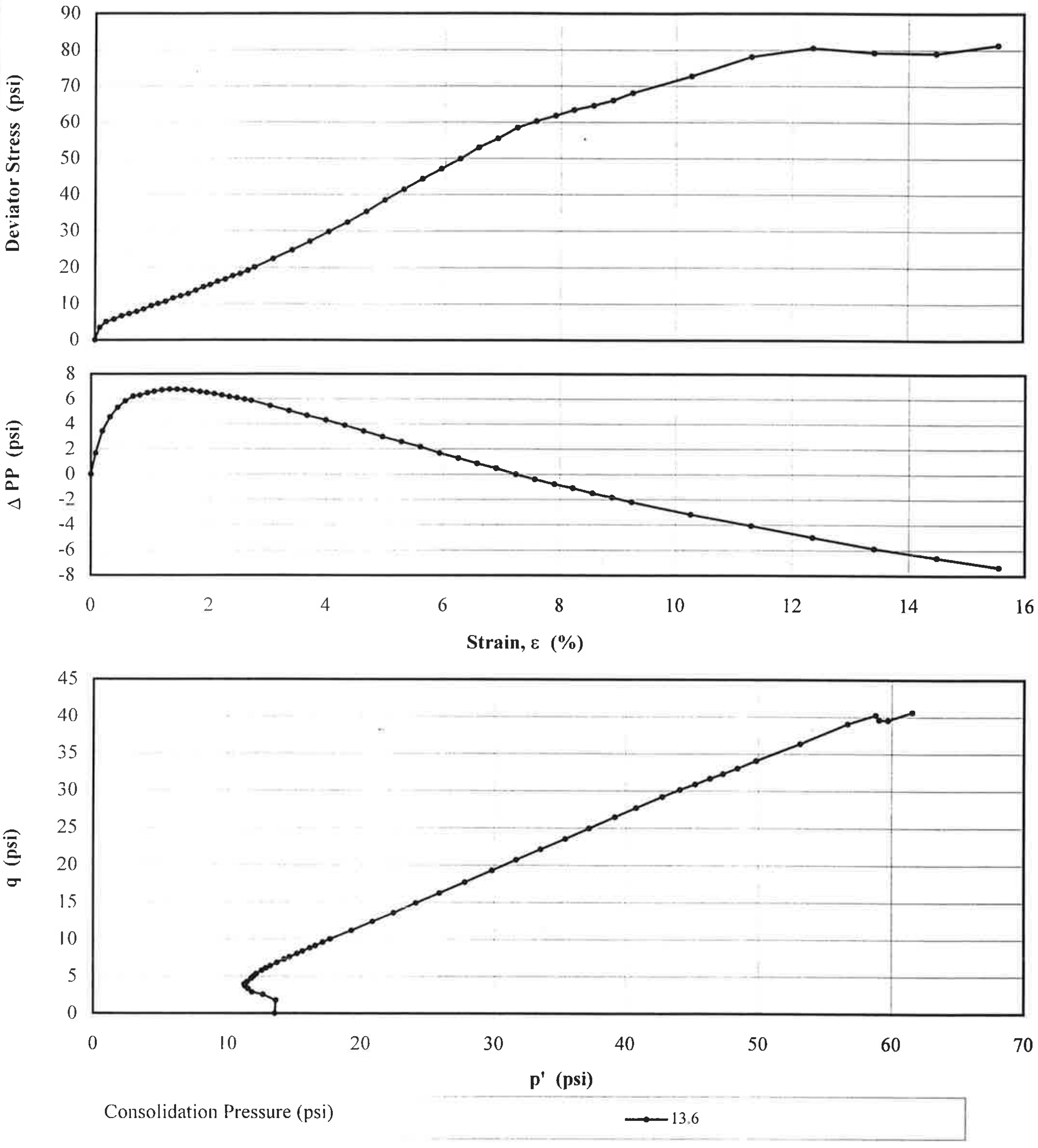
GEOSYNTEC CONSULTANTS
Geomechanics and Environmental Laboratory

Sample ID: **G-4 (D) (15'-16')**
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 1



Note:

1. Due to equipment malfunctioning, axial load piston generated friction forces beyond the recommended standard practice resulting in very high zero load correction.

98J21

TABLE 1

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-4 (D) (15'-16')	98J21.1	6.19	2.85	17.7	97.9	56.4	13.6					81.3	102.2	15.6	49.1	1	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1. Due to equipment malfunctioning, axial load piston generated friction forces beyond the recommended standard practice resulting in very high zero load correction.

GEOSYNTEC CONSULTANTS
Geomechanics and Environmental Laboratory

Sample ID: G-4 (L) (30'-32')

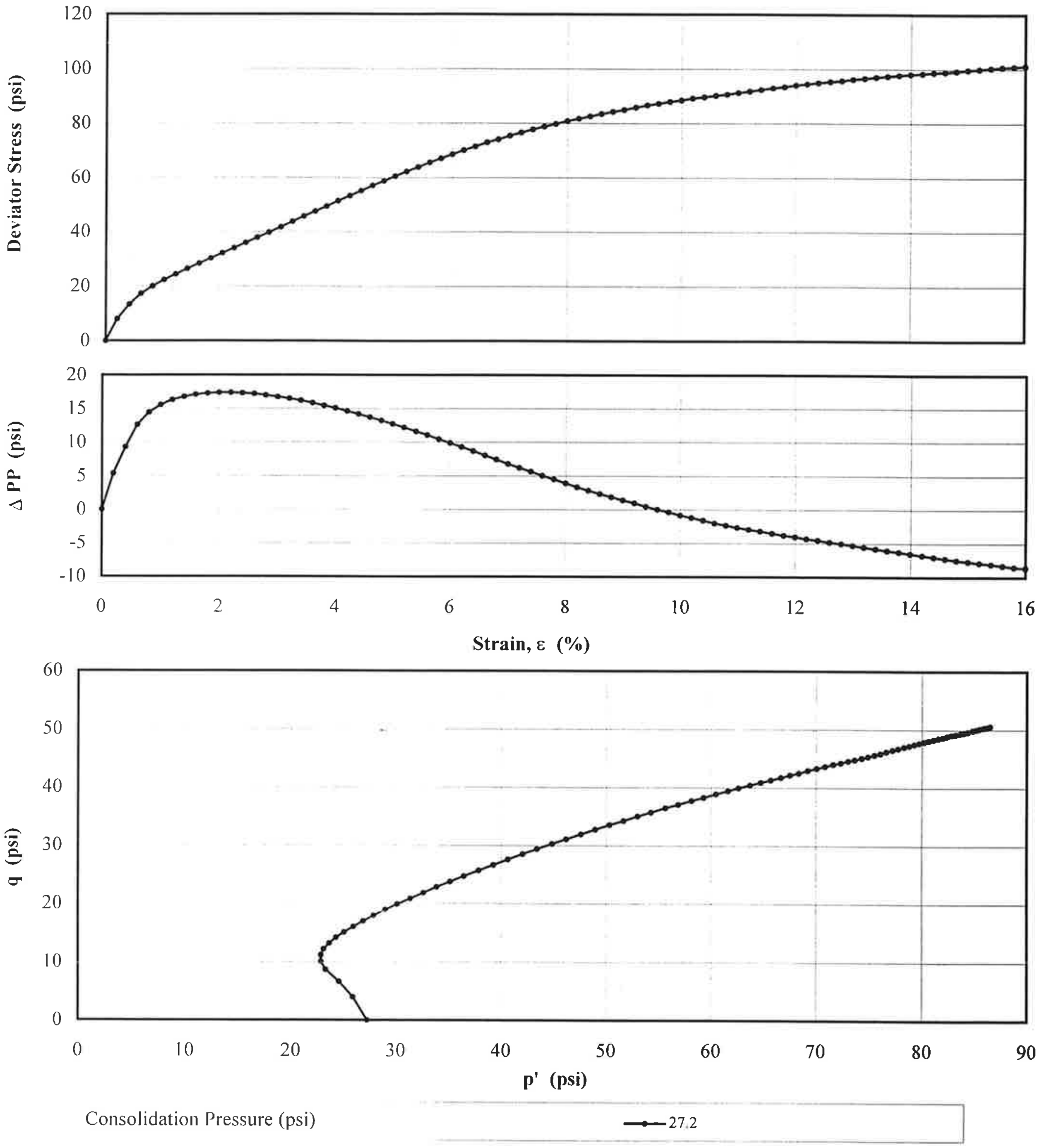
Project Name: LAKE PETIT DAM

Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 2



Note:

98J41

TABLE 2

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-4 (L) (30'-32')	98J41.1	6.73	2.89	27.8	97.2	51.2	27.2					101.2	137.1	16.0	42.6	2	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

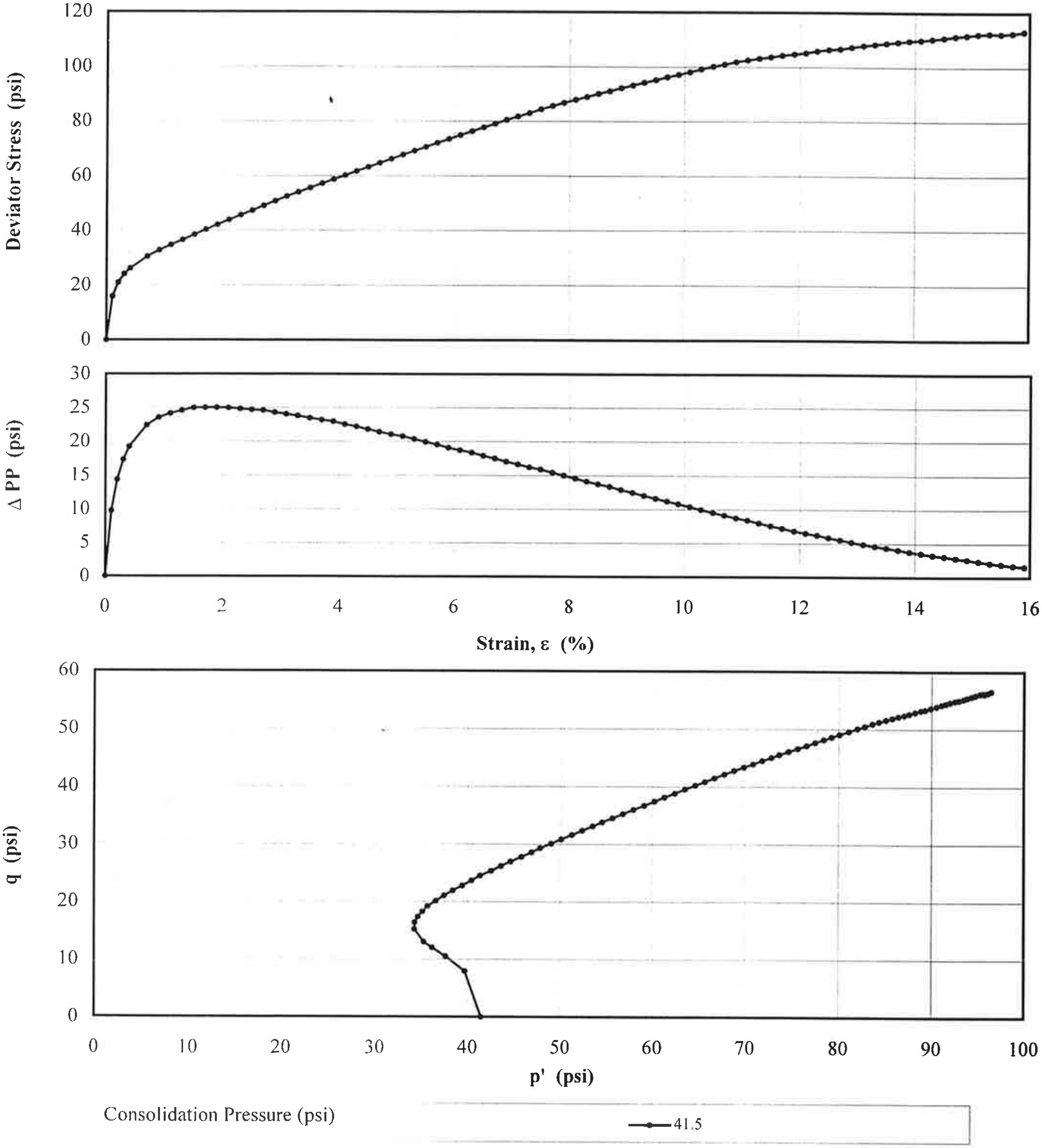
GEOSYNTEC CONSULTANTS
Geomechanics and Environmental
Laboratory

Sample ID: G-4 (H) (47'-50')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 3



Note:

TABLE 3

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-4 (H) (47'-50')	98J42.1	6.93	2.80	25.9	103.1	49.2	41.5					113.1	153.0	15.9	50.8	3	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

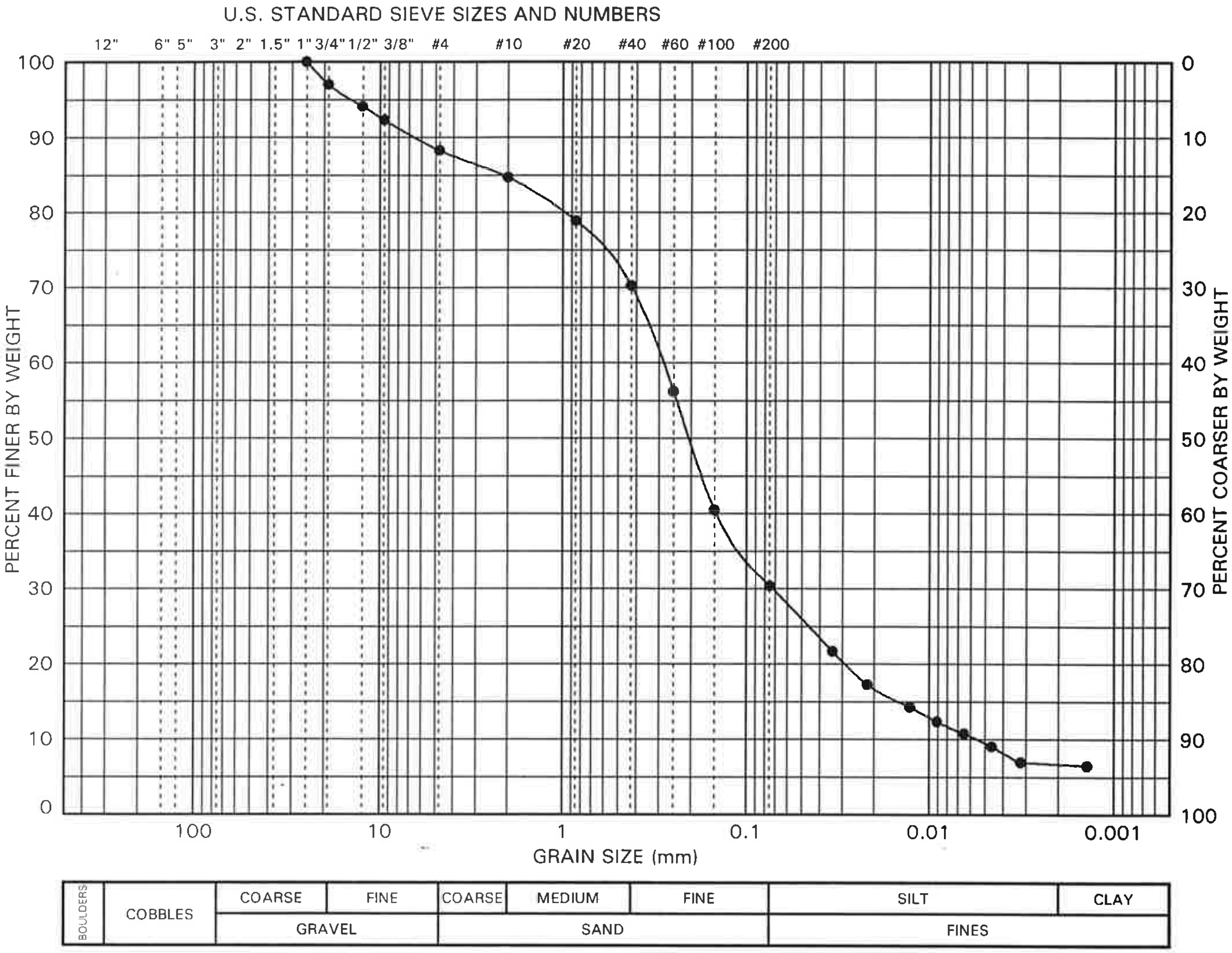
GEO SYNTEC CONSULTANTS
Geomechanics and Environmental Laboratory
Atlanta, Georgia

FIGURE
PROJECT: Lake Petit Dam
PROJECT NO.: GL0625
DOCUMENT NO.:

GS FORM:
4PS2 11/05/98

PARTICLE SIZE DISTRIBUTION AND PHYSICAL PROPERTIES

ASTM C 136, D 422, D 2487
D 3042 AND D 4318



SITE SAMPLE ID	*	LIQUID LIMIT (%)	NP
LAB. SAMPLE NO.	98J42	PLASTIC LIMIT (%)	NP
SAMPLE DEPTH (ft)		PLASTICITY INDEX	NP

SOIL CLASSIFICATION:

SM - Silty Sand

SOIL FRACTIONS		
	GRAVEL (%)	11.7
	SAND (%)	57.9
	FINES (%)	30.4
	SILT (%)	23.7
	CLAY(%)	6.7
COEFF. UNIFORMITY (Cu)		
COEFF. CURVATURE (Cc)		

PERCENT PASSING U.S. STANDARD SIEVE SIZES AND NUMBERS														PERCENT FINER THAN HYDROMETER PARTICLE DIAMETER (mm)				
3"	2"	1.5"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200					
PERCENT PASSING SIEVE SIZES (mm)																		
75	50	37.5	25	19	12.5	9.5	4.75	2.00	0.850	0.425	0.250	0.150	0.075	0.050	0.020	0.005	0.002	0.001
100	100	100	100	97	94	92	88	85	79	70	56	41	30	26	17	10	7	

NOTES: * G-4(H) (47-50)

GeoSyntec Consultants

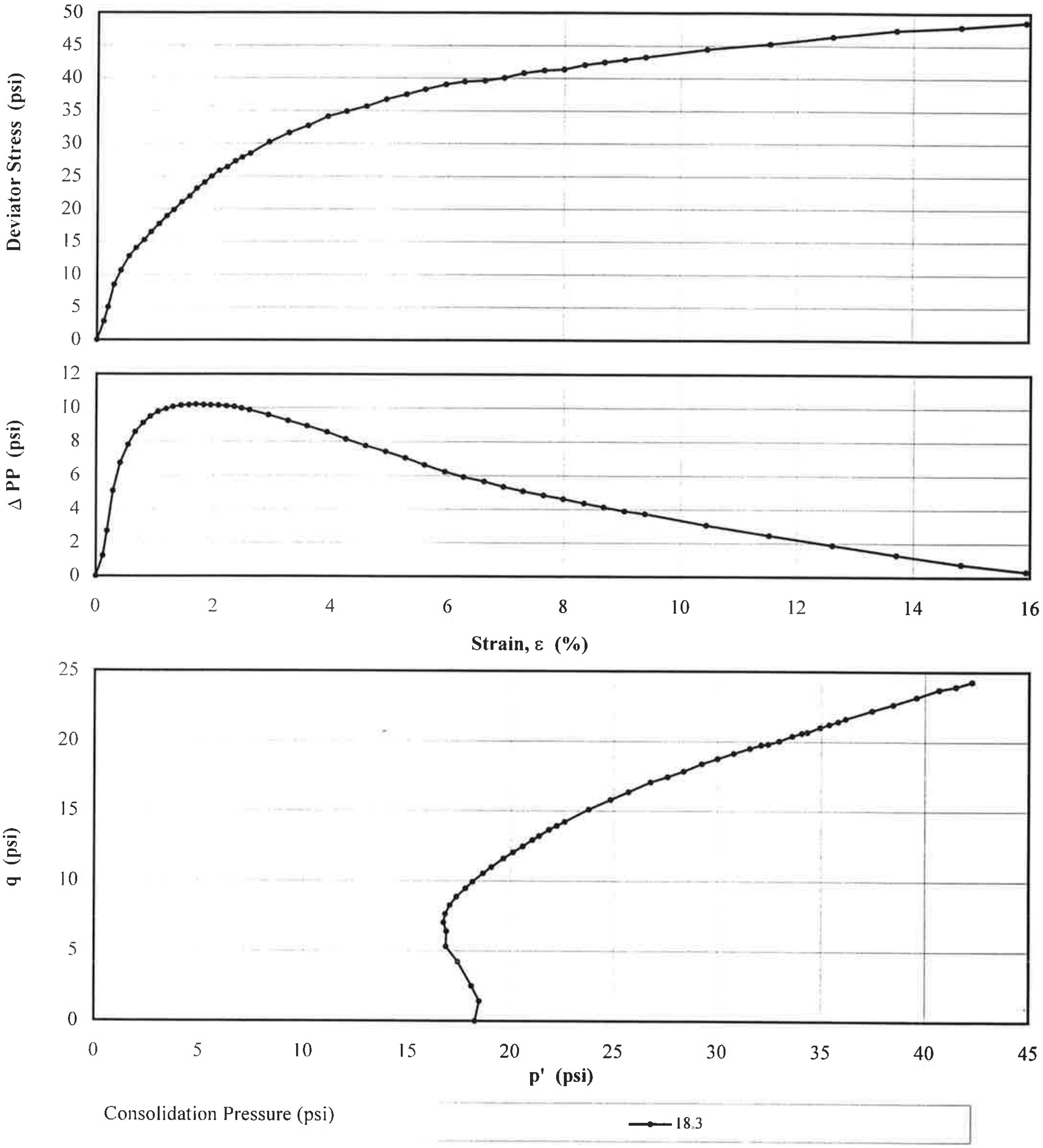
Geomechanics and Environmental Laboratory

Sample ID: G-1B (E) (20'-22')

Project Name: LAKE PETIT DAM

Project No.: GL0625

ASTM D 4767 | **TRIAXIAL COMPRESSION TESTING** | **Figure 4**



Note:

98J67

GL0625/46156388.XLS

TABLE 4

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-1B (E) (20'-22')	98J67.1	5.91	2.86	19.1	103.5	50.6	18.3					48.6	66.5	15.9	50.9	4	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

GeoSyntec Consultants
Geomechanics and Environmental Laboratory

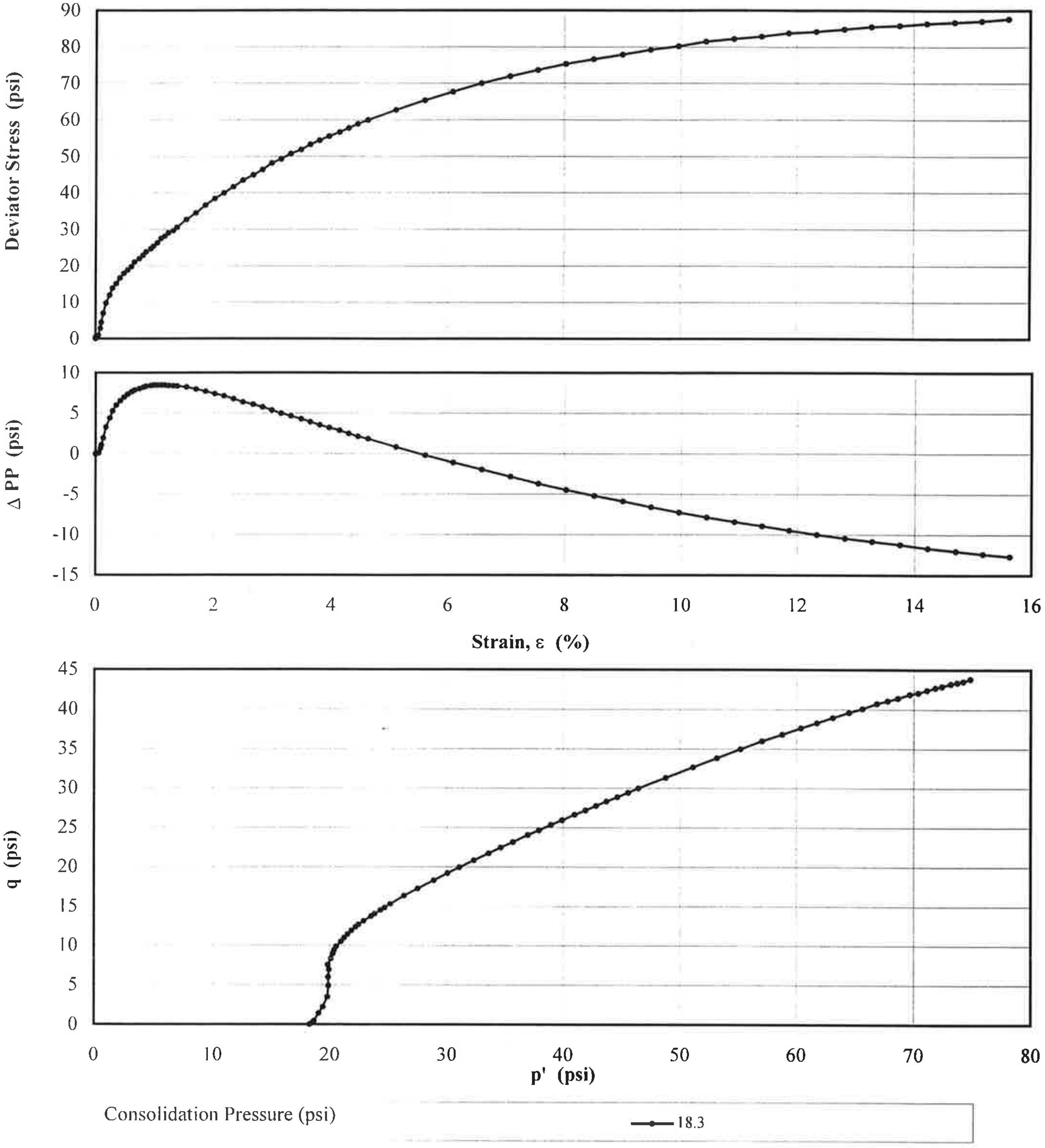
GEOSYNTEC CONSULTANTS
Geomechanics and Environmental
Laboratory

Sample ID: **G-1B (E) (20'-22')-Remolded**
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 5



Note(s):

1. The test specimen was formed/remolded by recycling the tested (sheared) undisturbed Shelby tube specimen. The test material was passed through a U.S. Standard No. 3/8" sieve. The passing portion was remolded at a moisture content of 16.9% and at a dry unit weight of 102.8 pcf.
2. The test specimen was initially consolidated at 23.8 psi. and then was over-consolidated and sheared at 18.3 psi.

TABLE 5

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i (psi)	σ'_c (psi)	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)			$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-1B (E) (20'-22') Remolded	98J67-Remolded.1	6.26	2.85	16.9	102.8	78.6	18.3					87.7	118.6	15.6	65.9	5	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1. The test specimen was formed/remolded by recycling the tested (sheared) undisturbed Shelby tube specimen. The test material was passed through a U.S. Standard No. 3/8" sieve. The passing portion was remolded at a moisture content of 16.9% and at a dry unit weight of 102.8 pcf.
2. The test specimen was initially consolidated at 23.8 psi, and then was over-consolidated and sheared at 18.3 psi.

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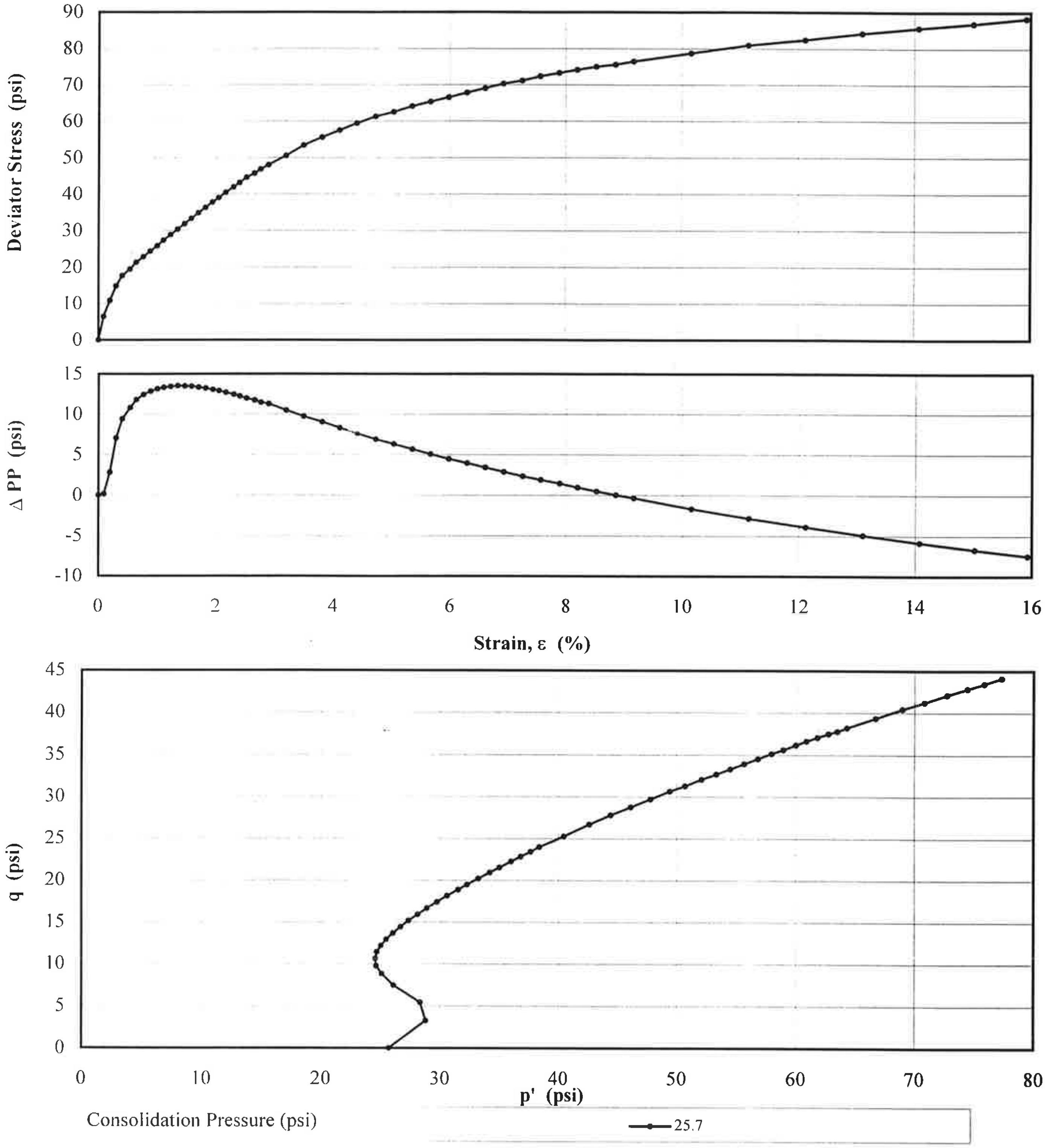
Sample ID: G-1B (H) (38'-40')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 6



Note:

98J68

TABLE 6

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-1B (H) (38'-40')	98J68.1	6.69	2.87	19.8	104.8	60.1	25.7					88.3	121.4	15.9	52.6	6	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

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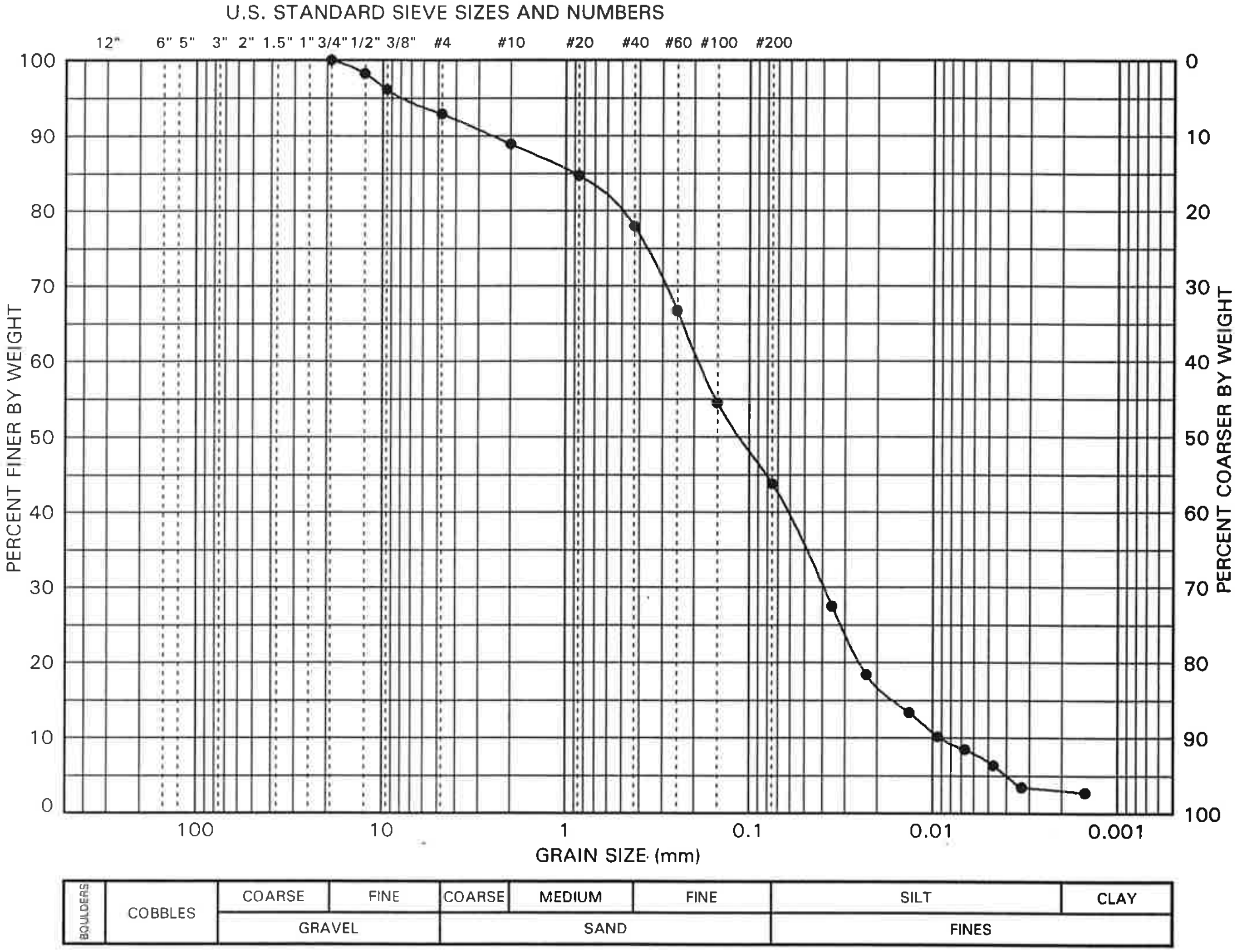
FIGURE
PROJECT: Lake Petit Dam
PROJECT NO.: GL0625
DOCUMENT NO.:

GS FORM:
4PS2 10/26/98

PARTICLE SIZE DISTRIBUTION AND PHYSICAL PROPERTIES

ASTM C 136, D 422, D 2487
D 3042 AND D 4318



SITE SAMPLE ID	*	LIQUID LIMIT (%)	33
LAB. SAMPLE NO.	98J68	PLASTIC LIMIT (%)	30
SAMPLE DEPTH (ft)		PLASTICITY INDEX	3

SOIL CLASSIFICATION:
SM - Silty Sand

SOIL FRACTIONS	GRAVEL (%)	7.1
	SAND (%)	49.1
	FINES (%)	43.8
	SILT (%)	40.7
	CLAY(%)	3.1
COEFF. UNIFORMITY (Cu)		
COEFF. CURVATURE (Cc)		

PERCENT PASSING U.S. STANDARD SIEVE SIZES AND NUMBERS														PERCENT FINER THAN HYDROMETER PARTICLE DIAMETER (mm)				
3"	2"	1.5"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200					
PERCENT PASSING SIEVE SIZES (mm)																		
75	50	37.5	25	19	12.5	9.5	4.75	2.00	0.850	0.425	0.250	0.150	0.075	0.050	0.020	0.005	0.002	0.001
100	100	100	100	100	98	96	93	89	85	78	67	55	44	35	17	7	3	

NOTES: * G-1B(H) (38-40)

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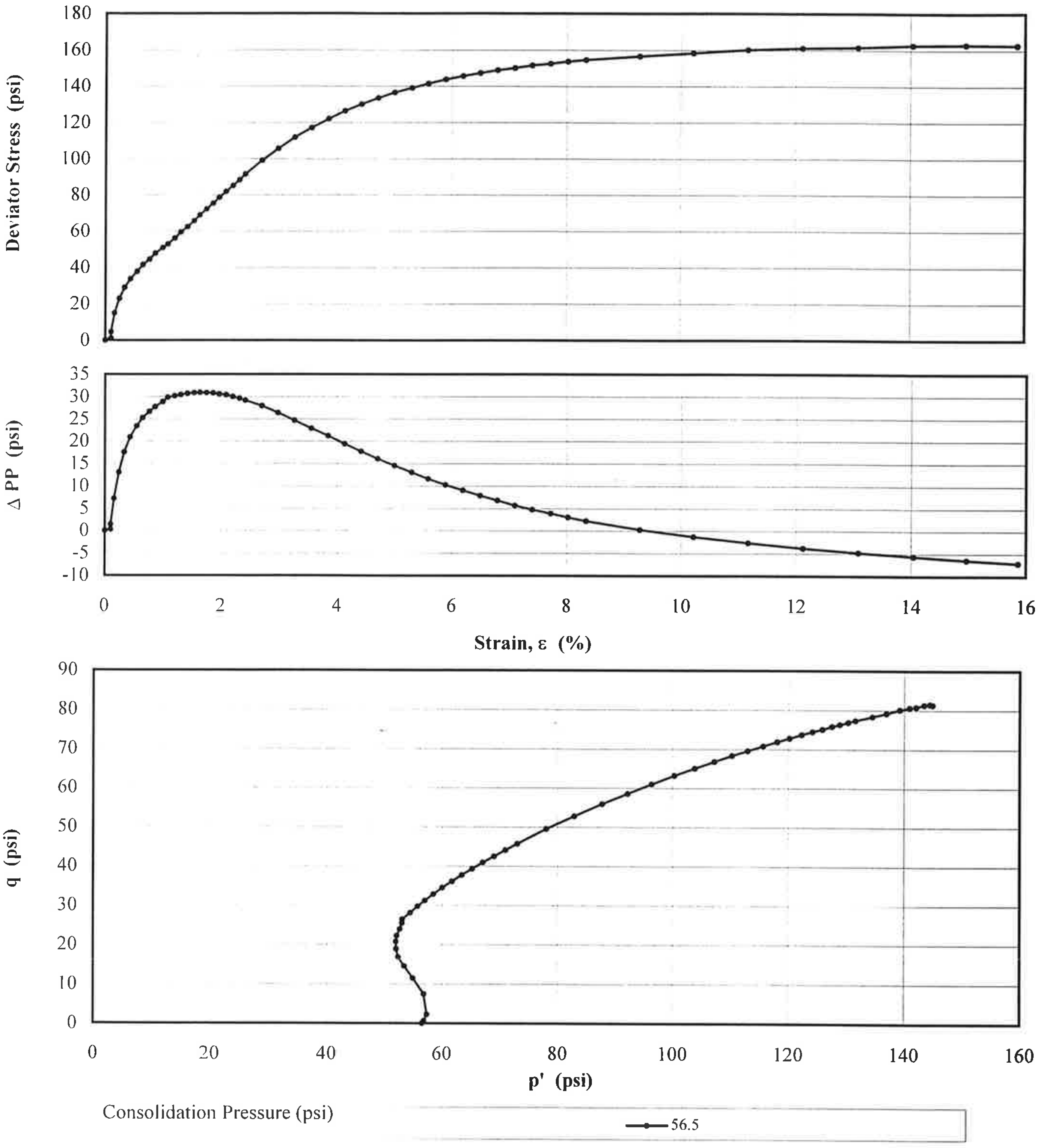
Sample ID: G-1B (P) (80'-81.5')
Project Name: LAKE PETIT DAM
Project No.: GLG0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 7



Note:

98J75

GLG0625/46198800.XLS

TABLE 7

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions						Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	u_i (psi)	σ'_c (psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-1B (P) (80'-81.5')	98J75.1	6.93	2.89	16.5	108.1	48.2	56.5					162.6	226.2	15.9	41.1	7	

Notes:

u_i = Initial pore pressure,(psi)

u = Pore pressure,(psi)

σ'_c= Consolidation pressure, (psi)

σ'_1 = Effective axial stress, (psi)

σ'_3 = Effective radial stress (confining pressure), (psi)

ε_a = Axial strain, (%)

1.

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Atlanta, Georgia

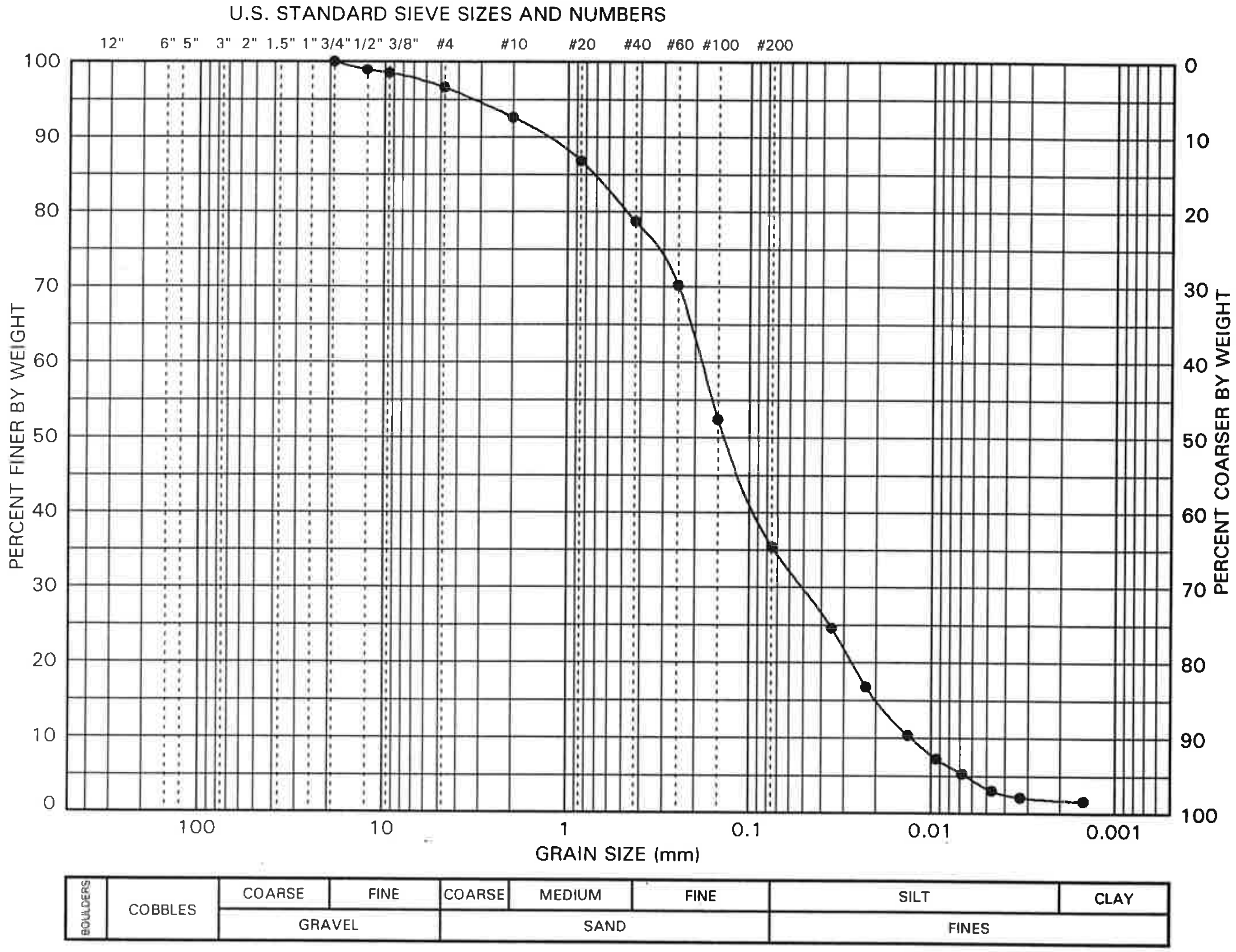
FIGURE

PROJECT: Lake Petit Dam
PROJECT NO.: GL0625
DOCUMENT NO.:

GS FORM:
4PS2 10/26/98

PARTICLE SIZE DISTRIBUTION AND PHYSICAL PROPERTIES

ASTM C 136, D 422, D 2487
D 3042 AND D 4318



SITE SAMPLE ID	*	LIQUID LIMIT (%)	NP	GRAVEL (%)	3.4
LAB. SAMPLE NO.	98J75	PLASTIC LIMIT (%)	NP	SAND (%)	61.2
SAMPLE DEPTH (ft)		PLASTICITY INDEX	NP	FINES (%)	35.4
				SILT (%)	33.5
				CLAY(%)	1.9

SOIL FRACTIONS

SOIL CLASSIFICATION:
SM - Silty Sand

COEFF. UNIFORMITY (Cu)
COEFF. CURVATURE (Cc)

PERCENT PASSING U.S. STANDARD SIEVE SIZES AND NUMBERS														PERCENT FINER THAN HYDROMETER PARTICLE DIAMETER (mm)				
3"	2"	1.5"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200					
PERCENT PASSING SIEVE SIZES (mm)																		
75	50	37.5	25	19	12.5	9.5	4.75	2.00	0.850	0.425	0.250	0.150	0.075	0.050	0.020	0.005	0.002	0.001
100	100	100	100	100	99	99	97	93	87	79	70	52	35	30	15	3	2	

NOTES: * G-1B(P) (80-81.5)

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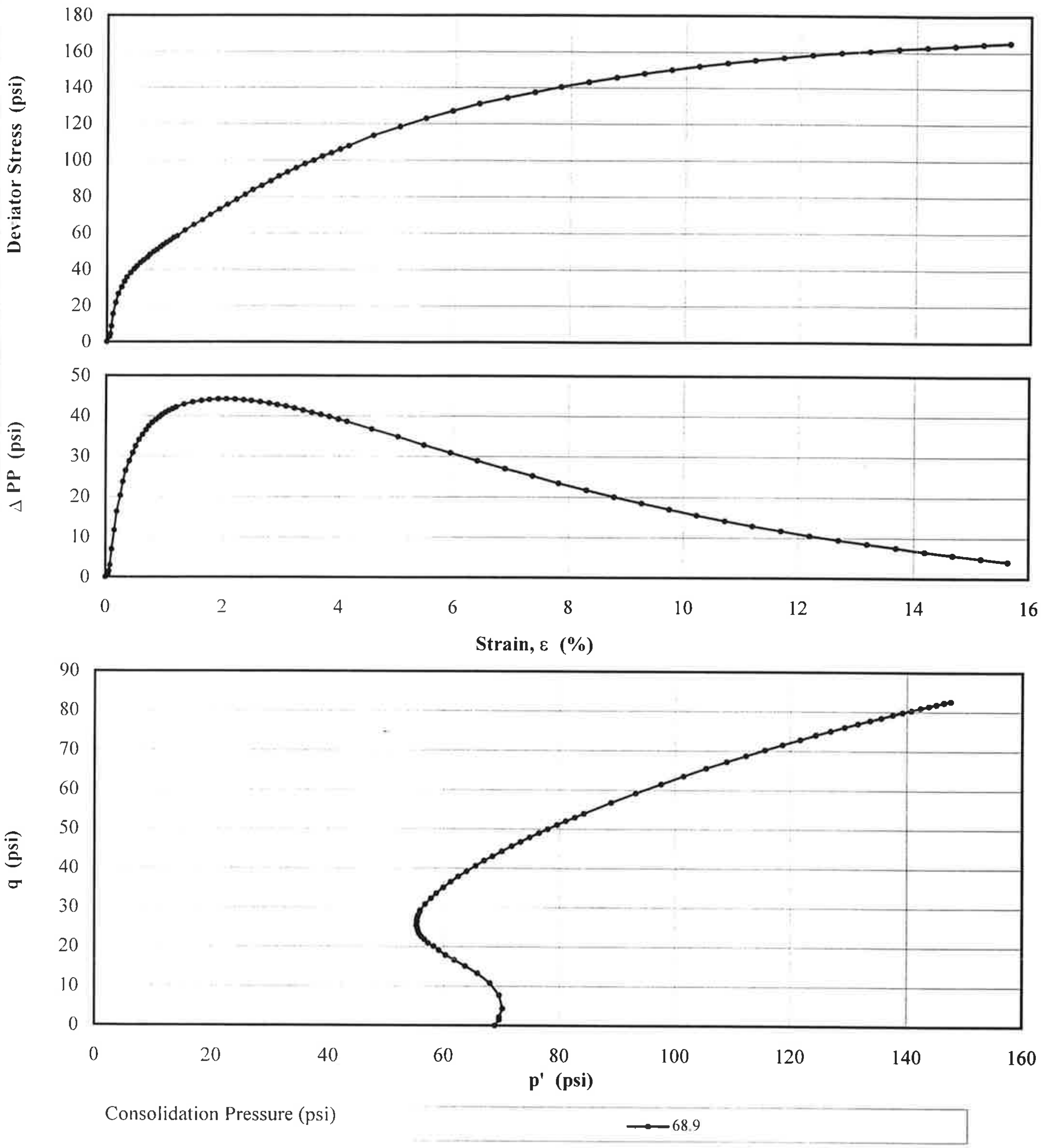
Sample ID: G-1B (U) (105'-107')
Project Name: LAKE PETIT DAM
Project No.: GLG0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 8



Note:

TABLE 8

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-1B (U) (105'-107')	98J76.1	6.65	2.88	20.7	109.8	32.2	68.9					165.3	230.1	15.6	36.2	8	

Notes:

u_i = Initial pore pressure,(psi)

u = Pore pressure,(psi)

σ'_c= Consolidation pressure, (psi)

σ'_1 = Effective axial stress, (psi)

σ'_3 = Effective radial stress (confining pressure), (psi)

ε_a = Axial strain, (%)

1.

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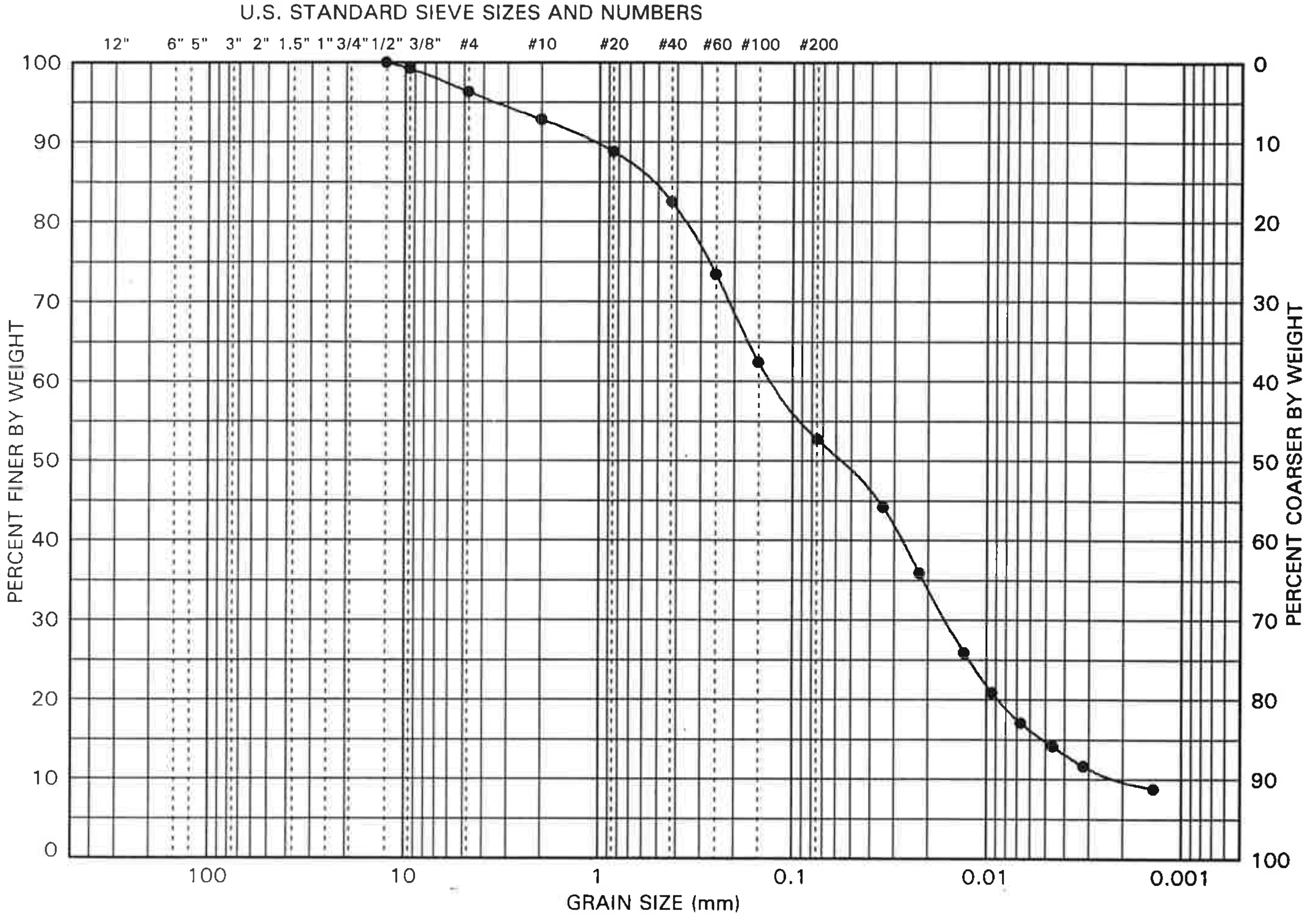
FIGURE
PROJECT: Lake Petit Dam
PROJECT NO.: GL0625
DOCUMENT NO.:

GS FORM:
4PS2 10/26/98

PARTICLE SIZE DISTRIBUTION AND PHYSICAL PROPERTIES

ASTM C 136, D 422, D 2487
D 3042 AND D 4318



BOULDERS	COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		GRAVEL		SAND			FINES	

SITE SAMPLE ID	*	LIQUID LIMIT (%)	41	SOIL FRACTIONS	GRAVEL (%)	3.7
LAB. SAMPLE NO.	98J76	PLASTIC LIMIT (%)	32		SAND (%)	43.6
SAMPLE DEPTH (ft)		PLASTICITY INDEX	9		FINES (%)	52.7
SOIL CLASSIFICATION: ML - Sandy Silt					SILT (%)	42.6
					CLAY(%)	10.1
					COEFF. UNIFORMITY (Cu)	
					COEFF. CURVATURE (Cc)	

PERCENT PASSING U.S. STANDARD SIEVE SIZES AND NUMBERS														PERCENT FINER THAN HYDROMETER PARTICLE DIAMETER (mm)				
3"	2"	1.5"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200					
PERCENT PASSING SIEVE SIZES (mm)																		
75	50	37.5	25	19	12.5	9.5	4.75	2.00	0.850	0.425	0.250	0.150	0.075	0.050	0.020	0.005	0.002	0.001
100	100	100	100	100	100	99	96	93	89	83	73	62	53	48	34	15	10	

NOTES: * G-1B(U) (105-107)

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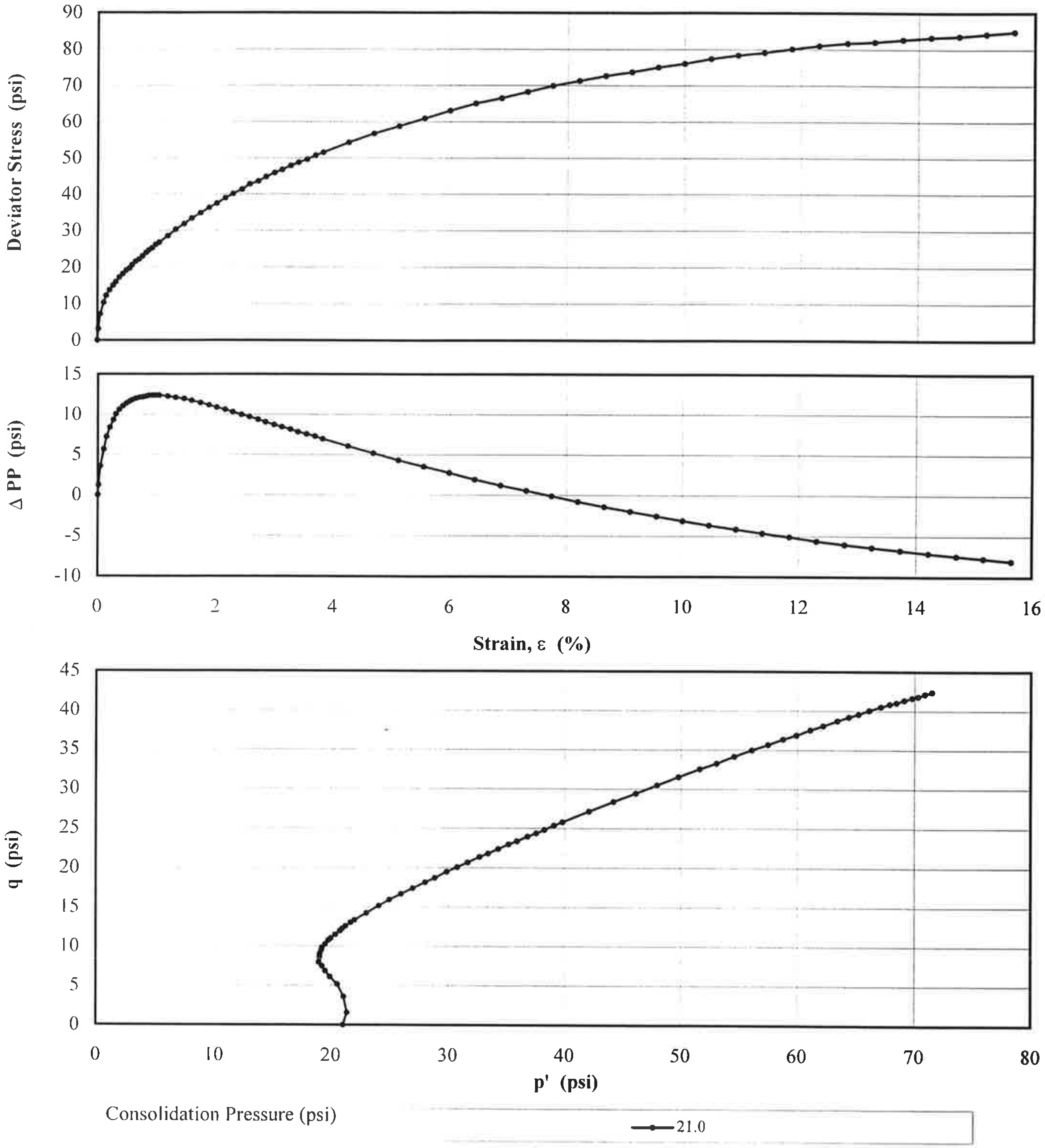
Laboratory

Sample ID:	**G-5 (G) (27'-30')**
Project Name:	LAKE PETIT DAM
Project No.:	GL0625

ASTM D 4767 | **TRIAXIAL COMPRESSION TESTING** | **Figure 9**



Note:

98J111

GL0625/46215997.XLS

TABLE 9

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-5 (G) (27'-30')	98J111.1	6.87	2.86	17.5	114.4	52.4	21.0					84.8	113.9	15.6	44.3	9	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

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Atlanta, Georgia

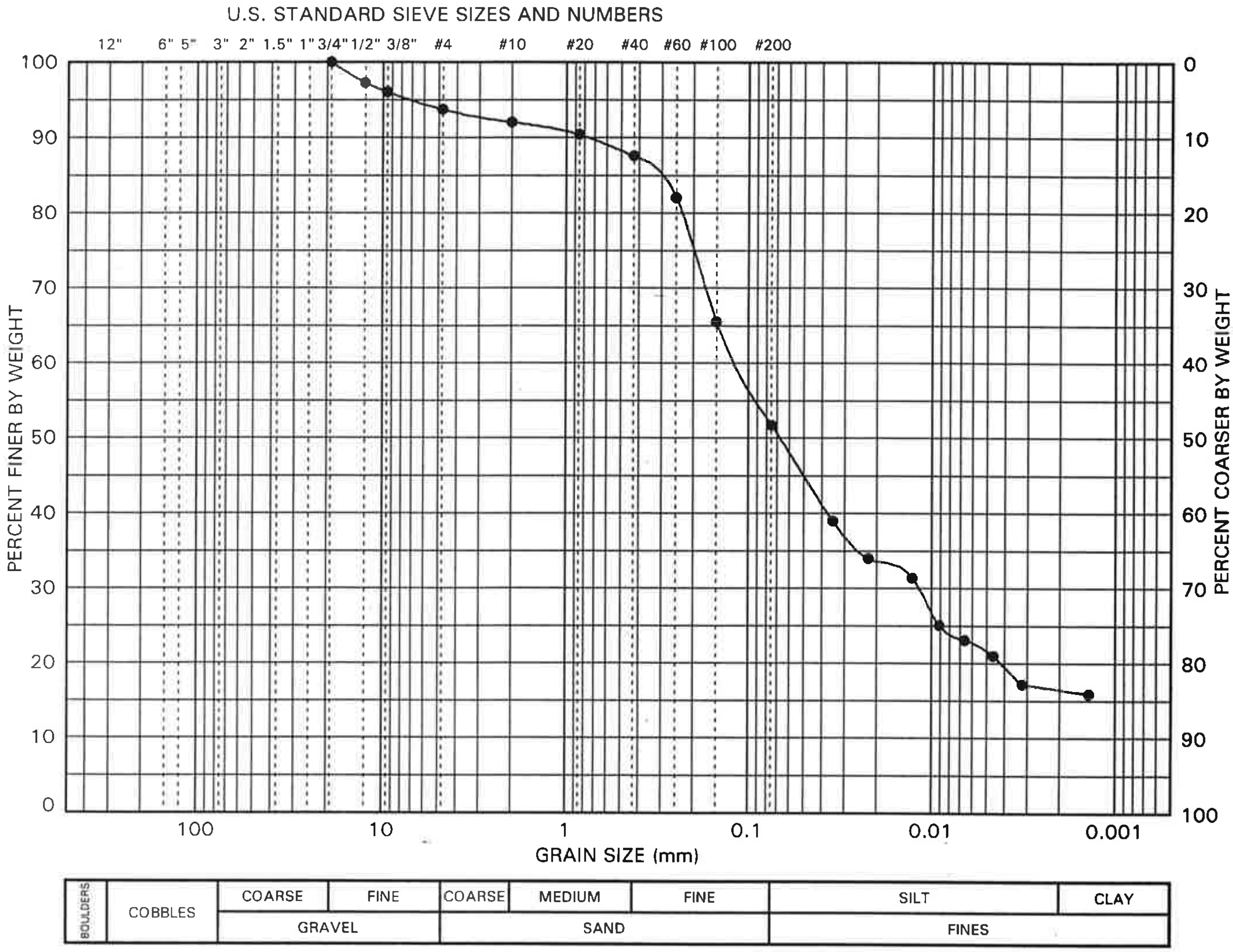
FIGURE

PROJECT: Lake Petit Dam
PROJECT NO.: GL0625
DOCUMENT NO.:

GS FORM:
4PS2 10/26/98

PARTICLE SIZE DISTRIBUTION AND PHYSICAL PROPERTIES

ASTM C 136, D 422, D 2487
D 3042 AND D 4318



SITE SAMPLE ID	*	LIQUID LIMIT (%)	33
LAB. SAMPLE NO.	98J111	PLASTIC LIMIT (%)	24
SAMPLE DEPTH (ft)		PLASTICITY INDEX	9

SOIL CLASSIFICATION:

ML - Sandy Silt

SOIL FRACTIONS		
	GRAVEL (%)	6.3
	SAND (%)	42.0
	FINES (%)	51.7
	SILT (%)	35.2
	CLAY(%)	16.5
	COEFF. UNIFORMITY (Cu)	
	COEFF. CURVATURE (Cc)	

PERCENT PASSING U.S. STANDARD SIEVE SIZES AND NUMBERS														PERCENT FINER THAN HYDROMETER PARTICLE DIAMETER (mm)				
3"	2"	1.5"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200					
PERCENT PASSING SIEVE SIZES (mm)																		
75	50	37.5	25	19	12.5	9.5	4.75	2.00	0.850	0.425	0.250	0.150	0.075	0.050	0.020	0.005	0.002	0.001
100	100	100	100	100	97	96	94	92	90	88	82	66	52	45	33	21	16	

NOTES: * G-5(G) (27-30)

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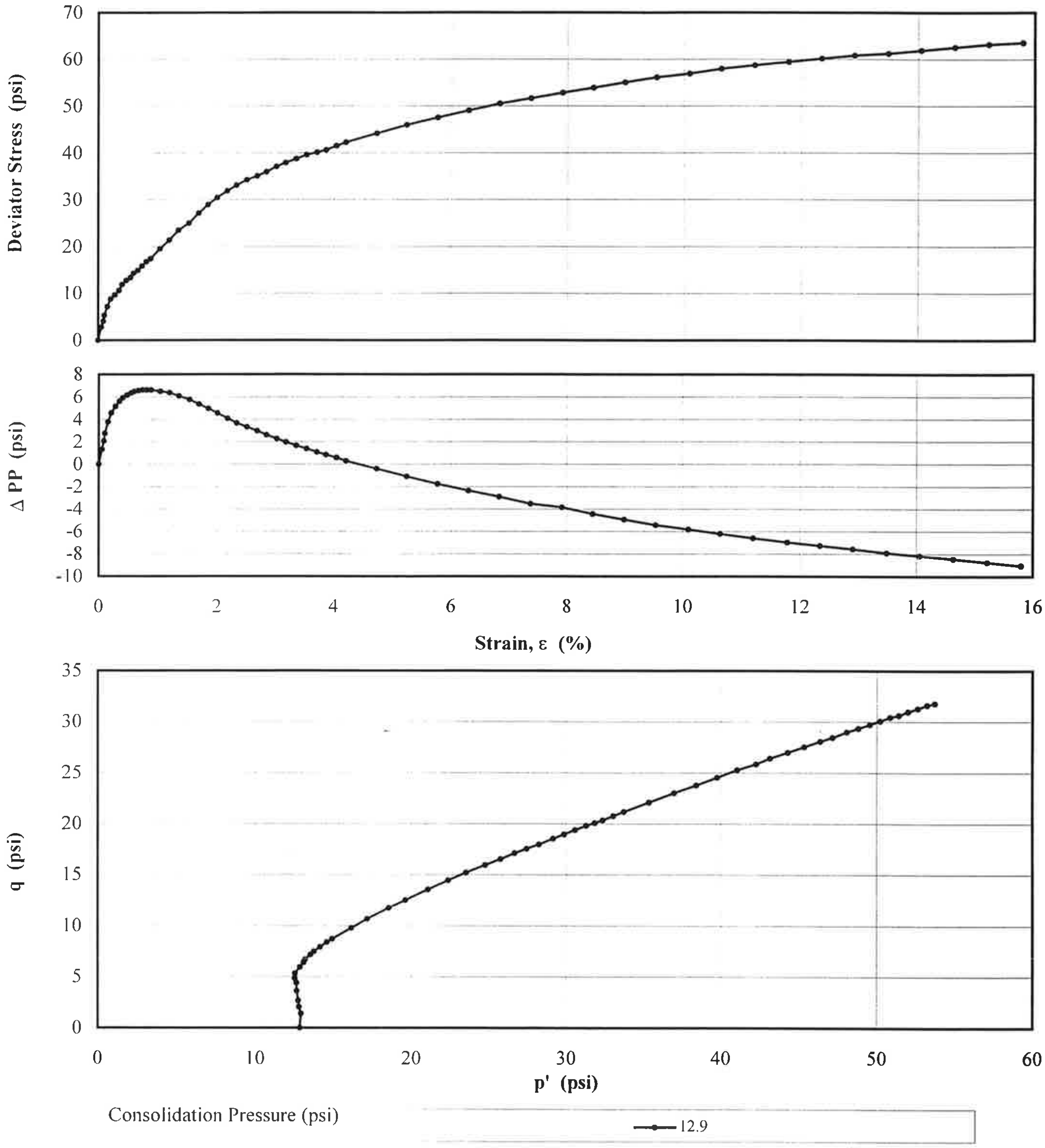
Sample ID: G-5 (C) (13'-15')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767 | **TRIAXIAL COMPRESSION TESTING** | **Figure 10**



Note:

98J112

TABLE 10

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-5 (C) (13'-15')	98J112.1	5.69	2.86	24.2	105.1	50.6	12.9					63.6	85.5	15.8	41.6	10	

Notes:

u_i = Initial pore pressure,(psi)

u = Pore pressure,(psi)

σ'_c= Consolidation pressure, (psi)

σ'_1 = Effective axial stress, (psi)

σ'_3 = Effective radial stress (confining pressure), (psi)

ε_a = Axial strain, (%)

1.

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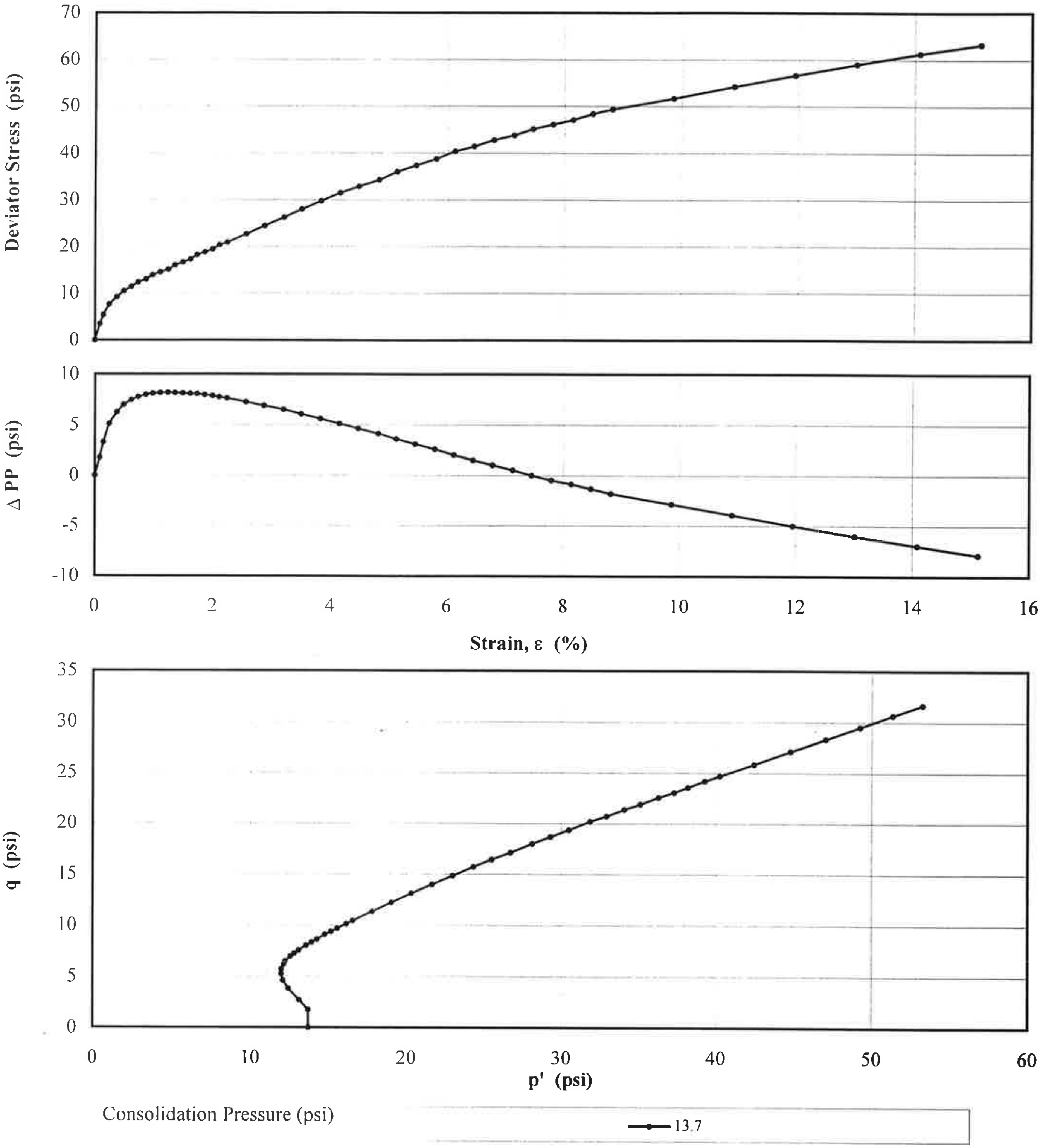
Sample ID: G-3 (D) (15'-17')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 11



Note:

TABLE 11

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	$\sigma'_1-\sigma'_3$ (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-3 (D) (15'-17')	98J141.1	6.14	2.84	22.5	107.4	51.1	13.7					63.3	84.9	15.1	43.2	11	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

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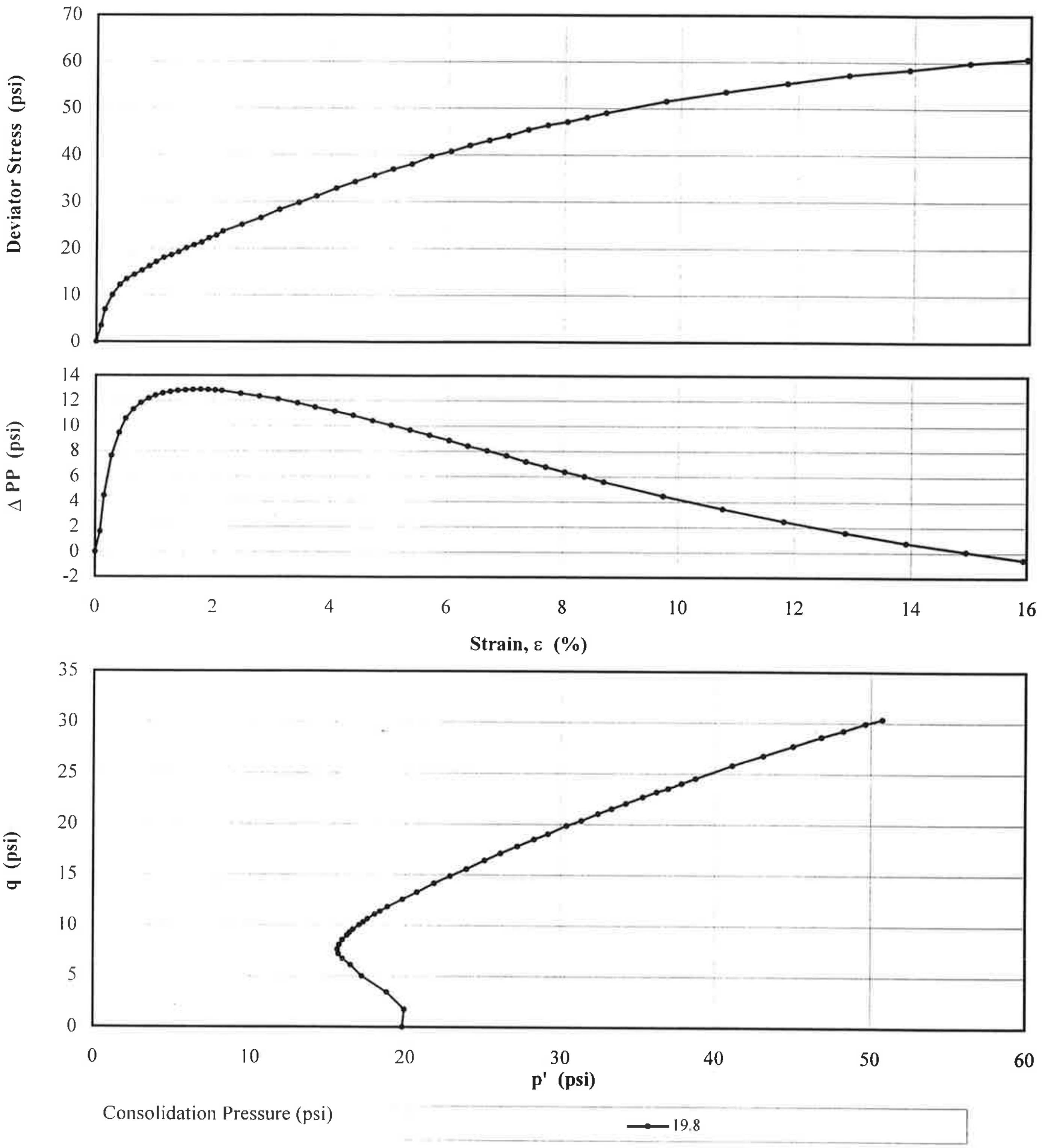
Sample ID: G-3 (G) (28'-30')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 12



Note:

TABLE 12

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-3 (G) (28'-30')	98J142.1	6.26	2.86	24.1	98.5	51.3	19.8					60.7	81.1	15.9	50.7	12	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

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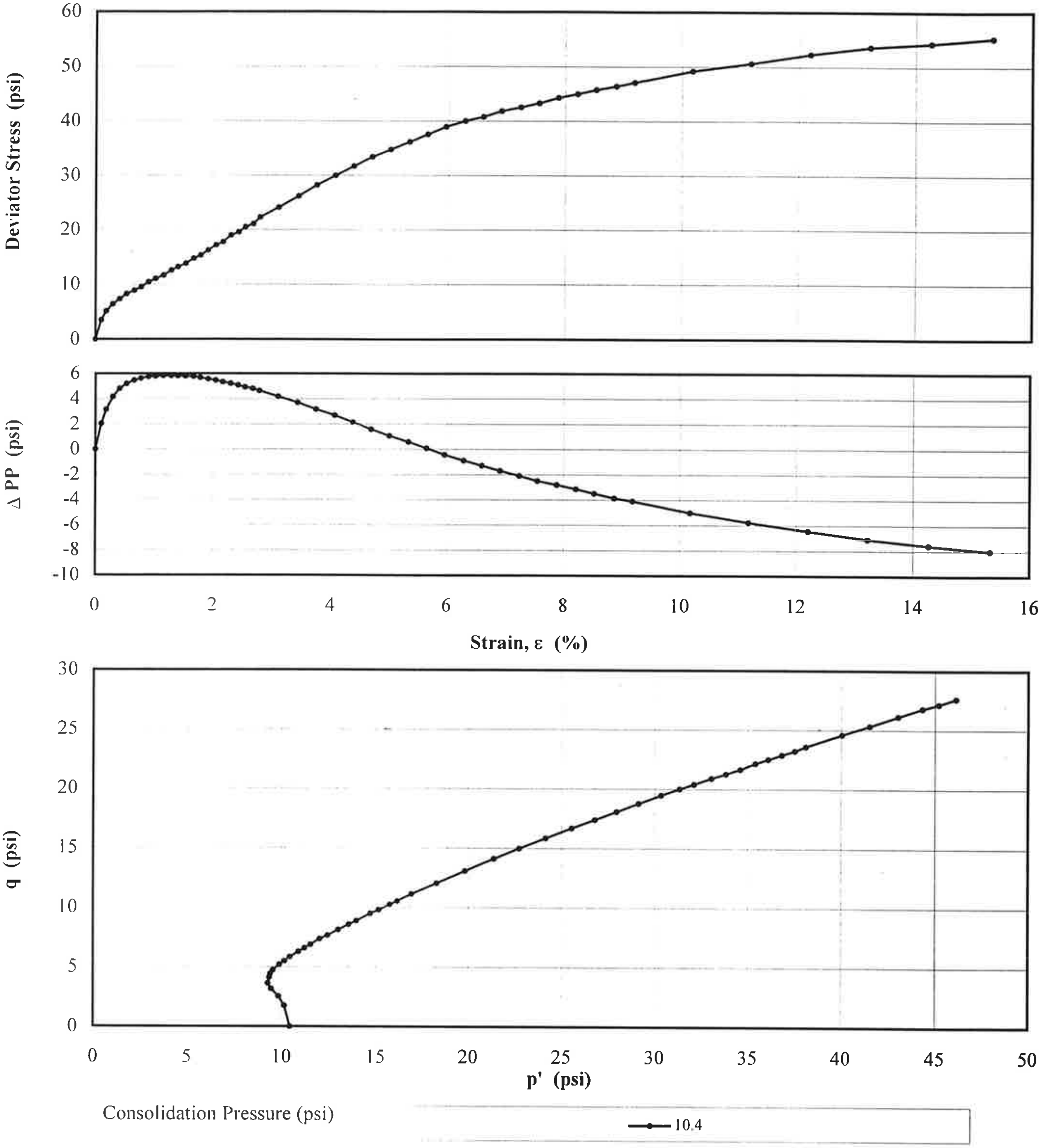
Sample ID: G-2 (B) (18'-20')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 13



Note:

TABLE 13

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-2 (B) (18'-20')	98J156.1	6.06	2.84	23.8	98.3	49.2	10.4					55.3	73.8	15.3	41.1	13	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

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Sample ID: G-2 (E) (38'-40')

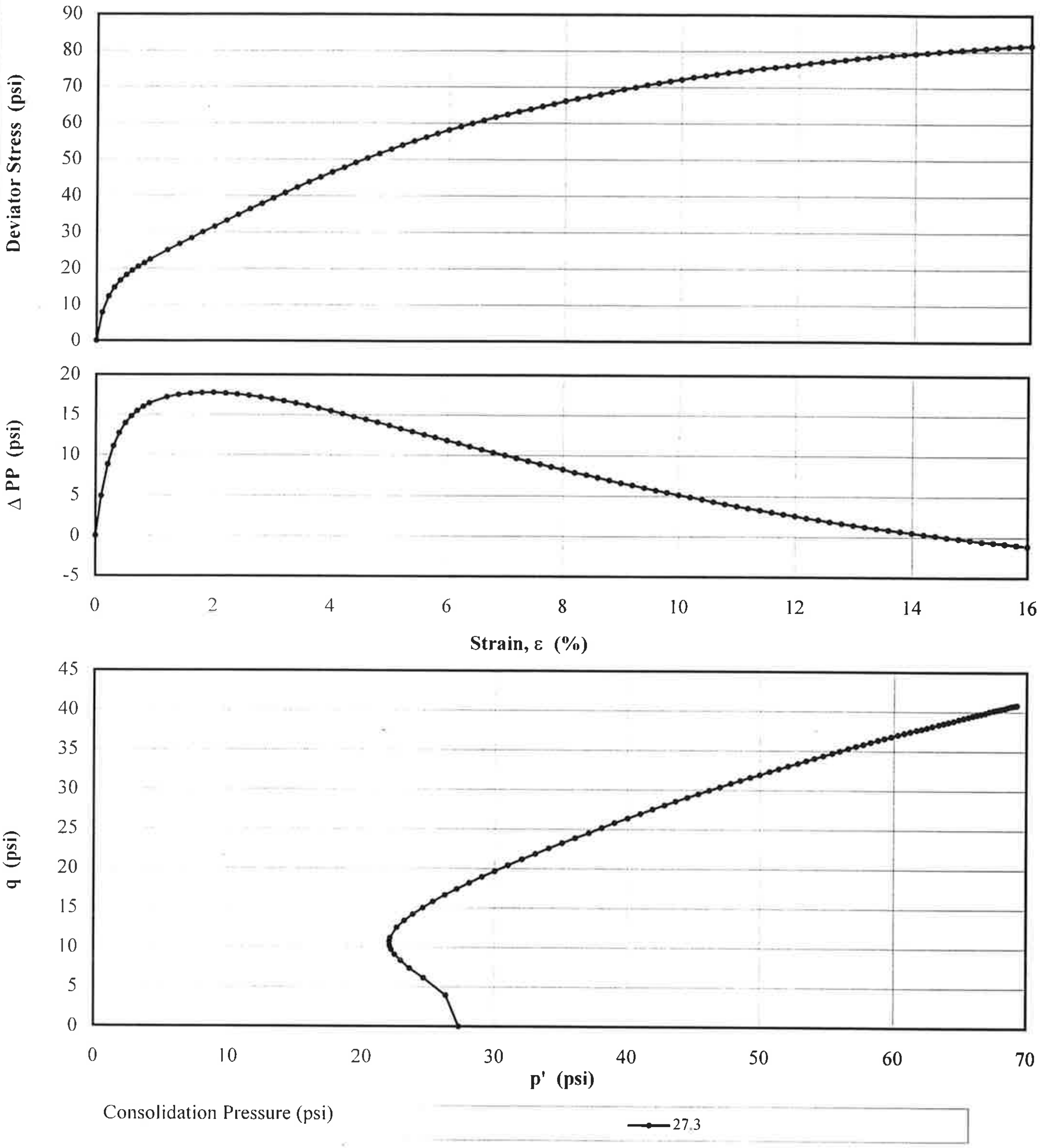
Project Name: LAKE PETIT DAM

Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 14



Note:

98J157

GL0625/4628796B.XLS

TABLE 14

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-2 (E) (38'-40')	98J157.1	5.83	2.87	18.7	106.5	49.7	27.3					81.7	110.1	16.0	48.6	14	

Notes:

u_i = Initial pore pressure,(psi)

u = Pore pressure,(psi)

σ'_c= Consolidation pressure, (psi)

σ'_1 = Effective axial stress, (psi)

σ'_3 = Effective radial stress (confining pressure), (psi)

ε_a = Axial strain, (%)

1.

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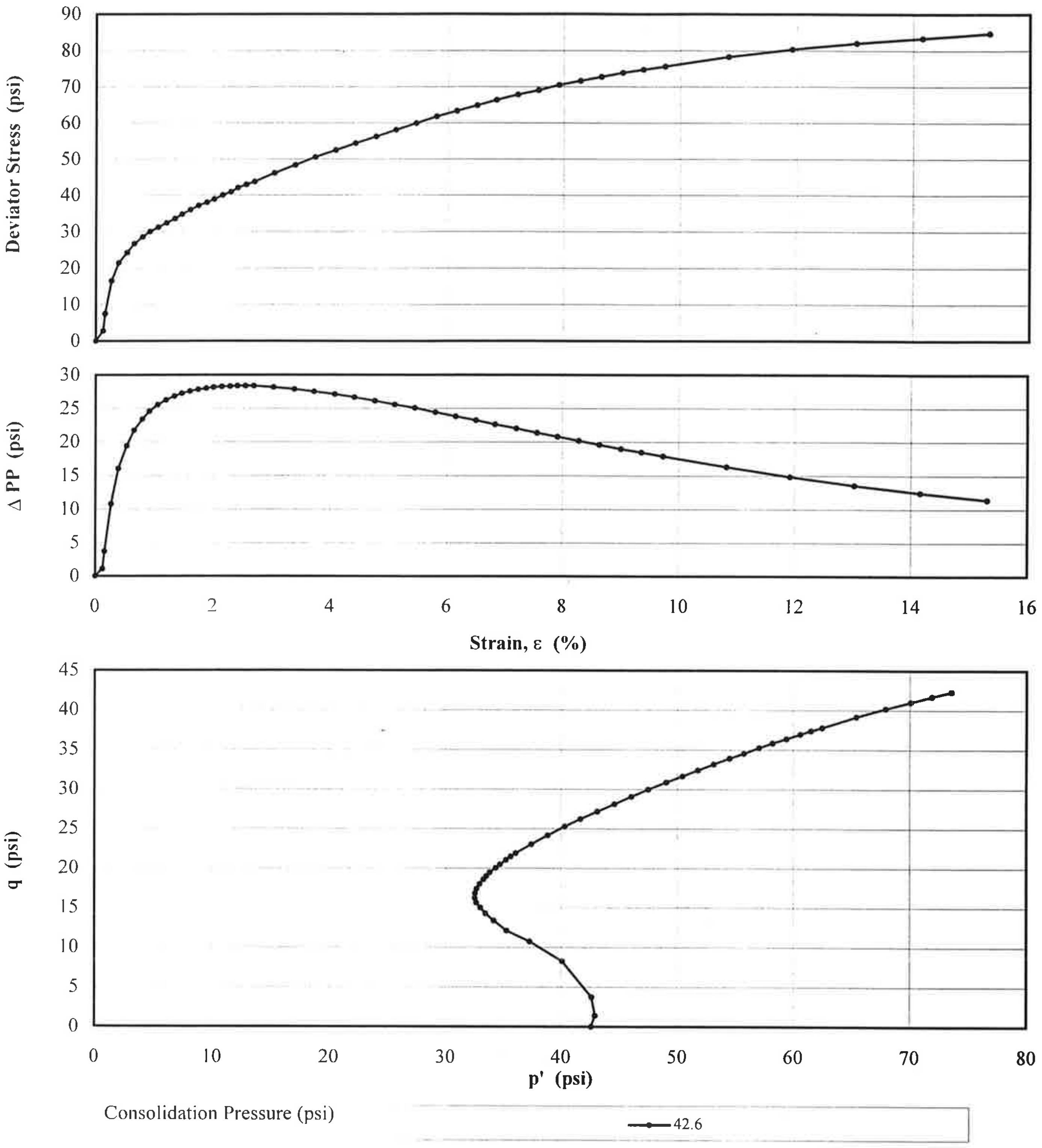
Geomechanics and Environmental
Laboratory

Sample ID: G-2 (H) (58'-60')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 15



Note:

98J159

TABLE 15

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) (1)

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i	σ'_c	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	(psi)	(psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-2 (H) (58'-60')	98J159.1	5.67	2.87	21.6	106.0	50.5	42.6					84.7	115.9	15.3	61.9	15	

Notes:

u_i = Initial pore pressure,(psi)
u = Pore pressure,(psi)
σ'_c= Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)

1.

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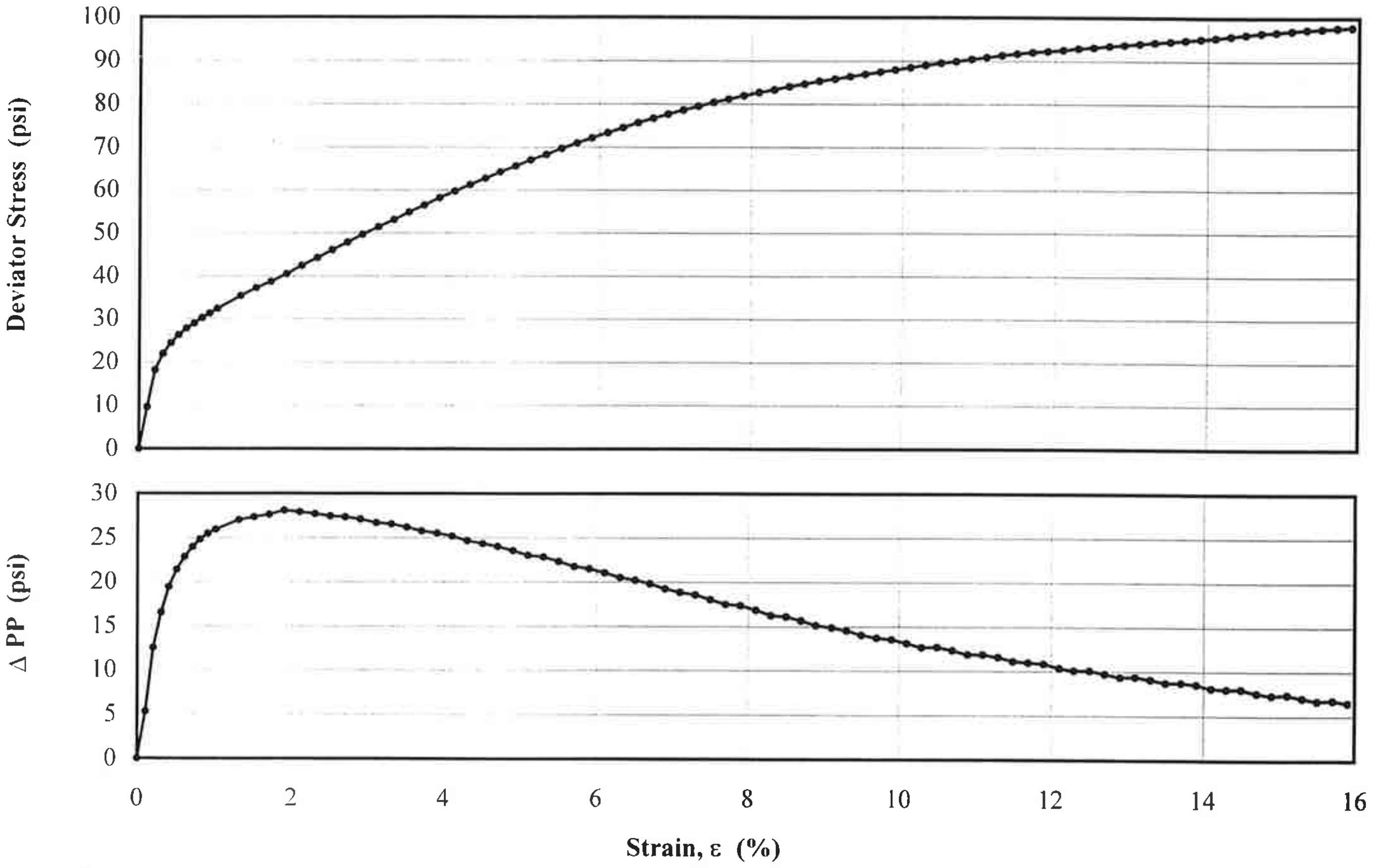
Laboratory

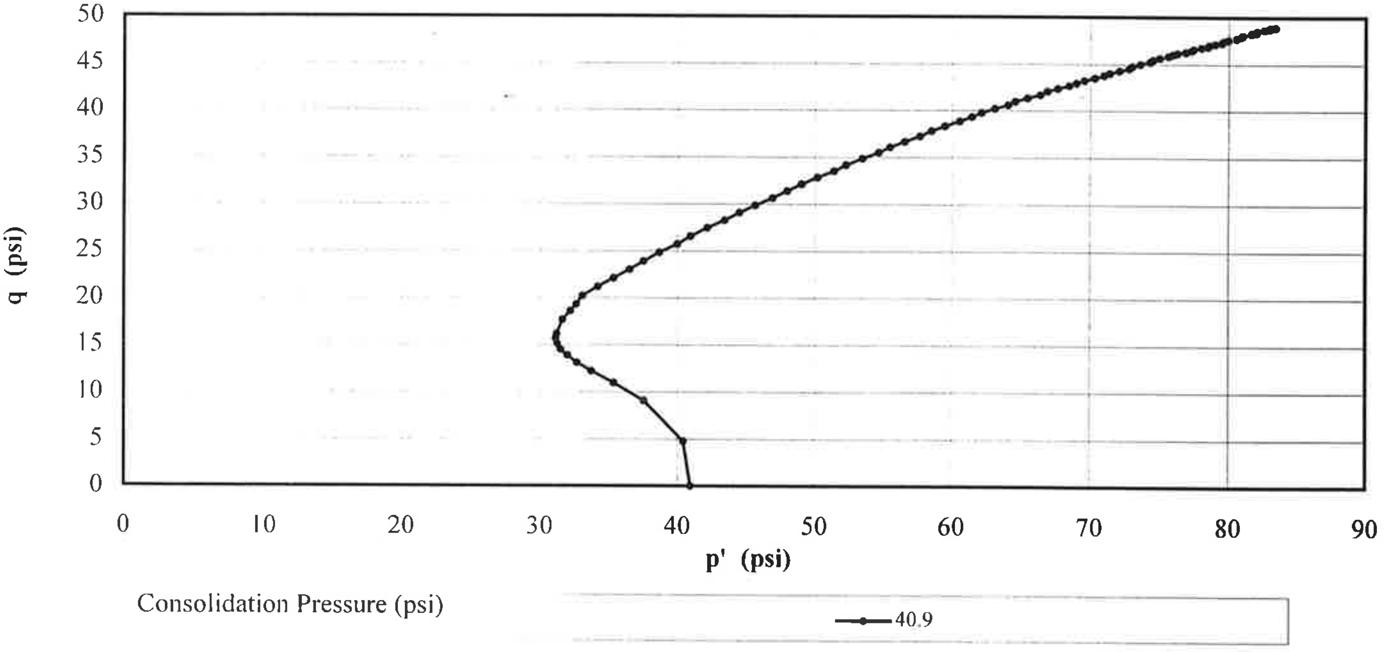
Sample ID: G-5 (P) (60'-62')
Project Name: LAKE PETIT DAM
Project No.: GL0625

ASTM D 4767

TRIAXIAL COMPRESSION TESTING

Figure 16





Note:

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Atlanta, Georgia

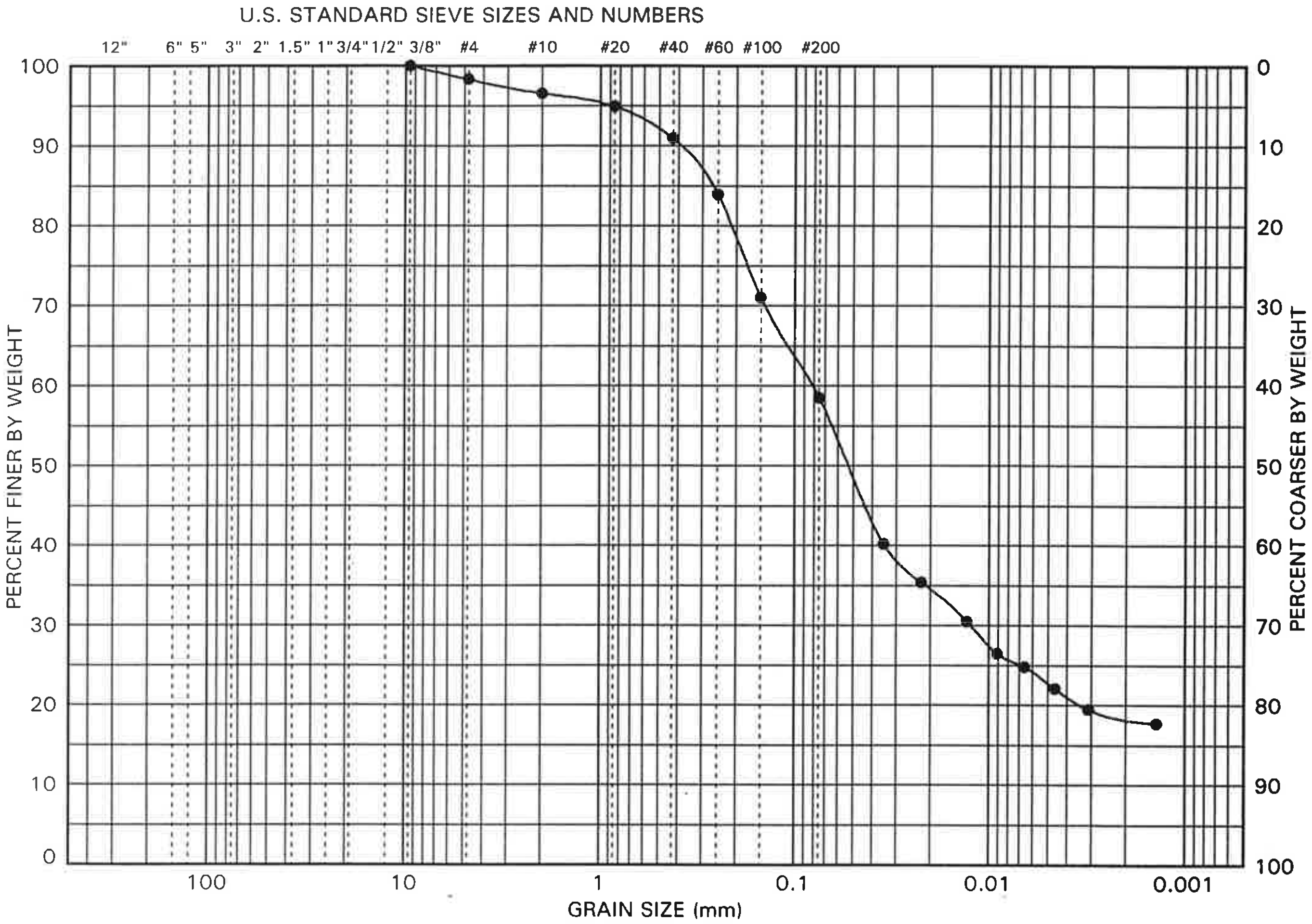
FIGURE

PROJECT:	Lake Petit Dam
PROJECT NO.:	GL0625
DOCUMENT NO.:	

GS FORM:
4PS2 10/26/98

PARTICLE SIZE DISTRIBUTION AND PHYSICAL PROPERTIES

ASTM C 136, D 422, D 2487
D 3042 AND D 4318



BOULDERS	COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		GRAVEL		SAND			FINES	

SITE SAMPLE ID	*	LIQUID LIMIT (%)	45
LAB. SAMPLE NO.	98J162	PLASTIC LIMIT (%)	30
SAMPLE DEPTH (ft)		PLASTICITY INDEX	15

SOIL CLASSIFICATION:

ML - Sandy Silt

SOIL FRACTIONS	
GRAVEL (%)	1.7
SAND (%)	39.8
FINES (%)	58.5
SILT (%)	40.0
CLAY(%)	18.5
COEFF. UNIFORMITY (Cu)	
COEFF. CURVATURE (Cc)	

PERCENT PASSING U.S. STANDARD SIEVE SIZES AND NUMBERS														PERCENT FINER THAN HYDROMETER PARTICLE DIAMETER (mm)				
3"	2"	1.5"	1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200					
PERCENT PASSING SIEVE SIZES (mm)																		
75	50	37.5	25	19	12.5	9.5	4.75	2.00	0.850	0.425	0.250	0.150	0.075	0.050	0.020	0.005	0.002	0.001
100	100	100	100	100	100	100	98	97	95	91	84	71	59	49	34	23	19	

NOTES: * G-5(P) (60-62)

TABLE 16

CONSOLIDATED UNDRAINED (ICU) TRIAXIAL COMPRESSION TESTS

SUMMARY OF TEST RESULTS (ASTM D 4767) $^{(1)}$

Site Sample ID	Lab Sample No.	Specimen Initial Conditions				u_i (psi)	σ'_c (psi)	Peak				Ultimate				Figure No.	Remarks
		Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)			σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)	σ'_1-σ'_3 (psi)	σ'_1 (psi)	ε_a (%)	u (psi)		
G-5 (P) (60'-62')	98J162.1	6.10	2.85	22.0	104.8	50.0	40.9					97.8	132.3	15.9	56.5	16	

Notes:

u_i = Initial pore pressure,(psi)

u = Pore pressure,(psi)

σ'_c= Consolidation pressure, (psi)

σ'_1 = Effective axial stress, (psi)

σ'_3 = Effective radial stress (confining pressure), (psi)

ε_a = Axial strain, (%)

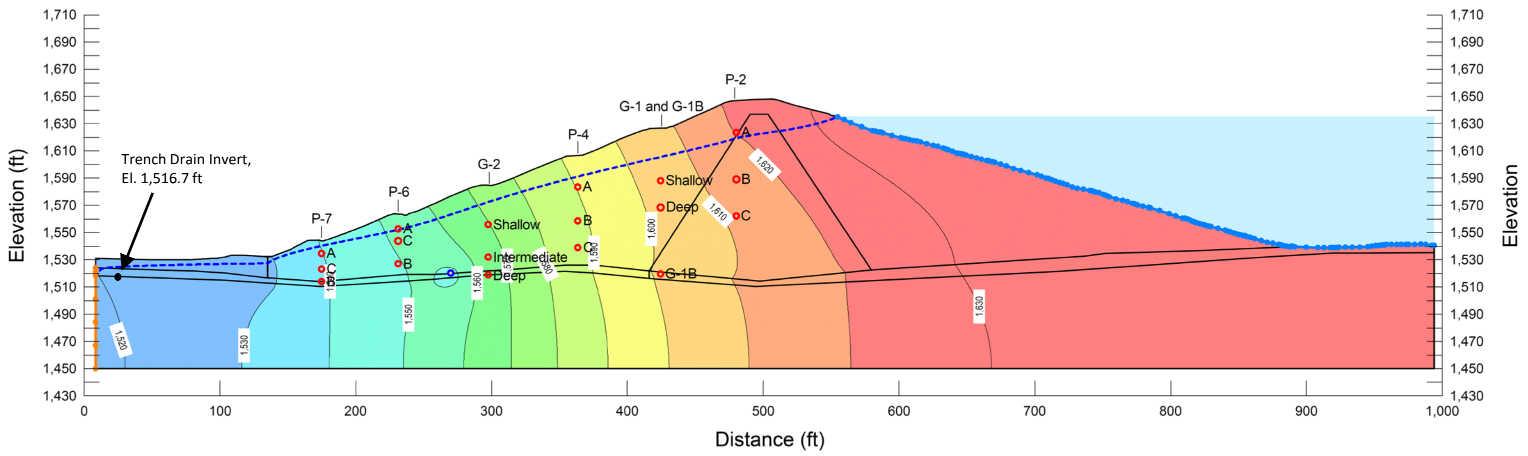
1.

ATTACHMENT 3
Seepage Analysis Results

Color	Name	K-Function	Sat Kx (ft/sec)
🟥	Bedrock		3.3e-09
🟩	Dam Core	Kx = 3.3e-5 ft/s (Dam Core)	
🟪	Dam Shell	Kx = 3.3e-5 ft/s (Dam Shell)	
🟦	Saprolite - D/S	Kx = 1.6e-6 ft/s (D/S Saprolite)	
🟩	Saprolite - U/S		3.3e-09
🟥	Soil below ball field	Kx = 1.6e-3 ft/s (Ball Field Soils)	

Color	Name
🟦	Normal Reservoir, EL 1635.5
🟧	Trench Drain Exit, 1516.7 ft
🟧	Trench Drain, 1,535 ft

Water Total Head
🟦 ≤ 1,520 - 1,530 ft
🟦 1,530 - 1,540 ft
🟦 1,540 - 1,550 ft
🟩 1,550 - 1,560 ft
🟩 1,560 - 1,570 ft
🟩 1,570 - 1,580 ft
🟩 1,580 - 1,590 ft
🟨 1,590 - 1,600 ft
🟧 1,600 - 1,610 ft
🟧 1,610 - 1,620 ft
🟥 ≥ 1,620 ft



Legend:
- Approximate location of piezometric instruments.
- Approximate location of trench drain, El. 1,520 ft.

Notes:
Cross-Section A-A
Steady-State Seepage Analysis
Normal Pool Reservoir
Headwater El. = 1,635.5 ft

STEADY-STATE SEEPAGE ANALYSIS
LAKE PETIT DAM

Geosyntec consultants

Figure 2-1

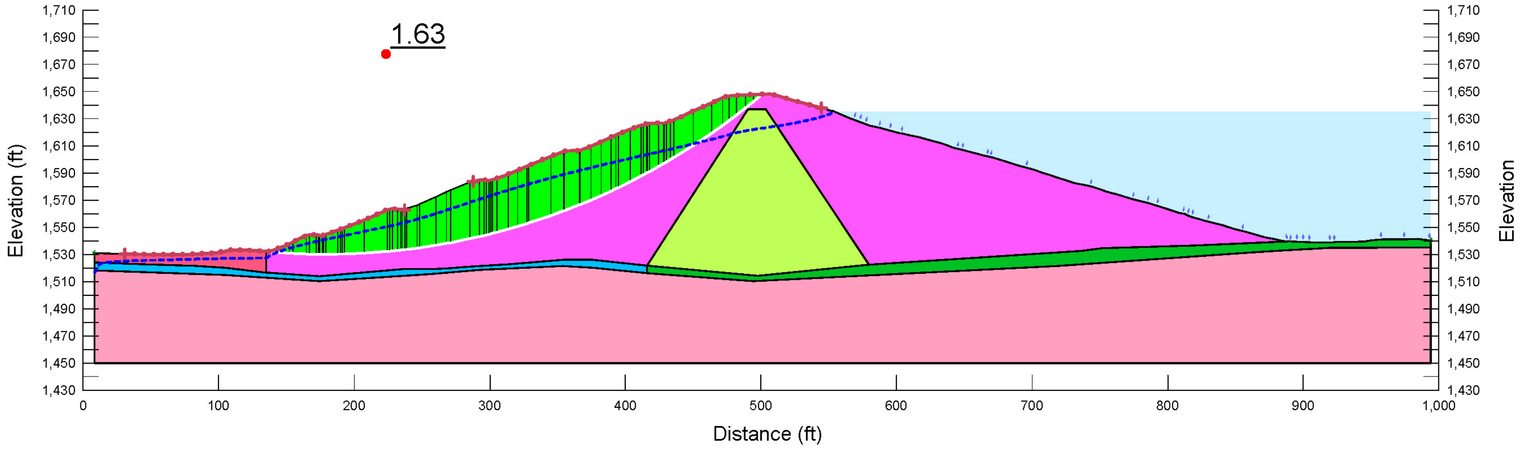
PROJECT NO. TCG10217

DATE: MAY 2024

ATTACHMENT 4
Slope Stability Analysis Results

Steady-State Seepage Stability Results

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core	Mohr-Coulomb	130	0	32
🟪	Dam Shell	Mohr-Coulomb	125	0	34
🟦	Saprolite - D/S	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S	Bedrock (Impenetrable)			
🟥	Soil below ball field	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Steady-State Seepage Stability Analysis of Downstream Slope
Normal Pool Elevation
Headwater Elev. = 1,635.5 ft

STEADY-STATE SEEPAGE STABILITY ANALYSIS OF DOWNSTREAM SLOPE
LAKE PETIT DAM

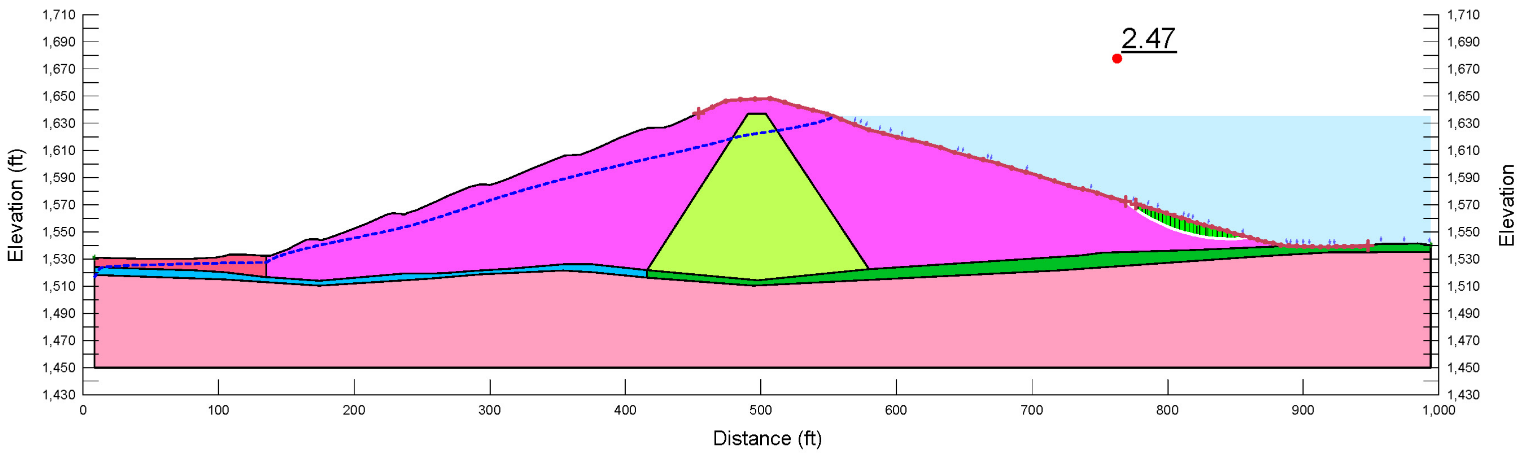
Geosyntec consultants

Figure 3-1

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core	Mohr-Coulomb	130	0	32
🟪	Dam Shell	Mohr-Coulomb	125	0	34
🟦	Saprolite - D/S	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S	Bedrock (Impenetrable)			
🟥	Soil below ball field	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Steady-State Seepage Stability Analysis of Upstream Slope
Normal Pool Elevation
Headwater Elev. = 1,635.5 ft

STEADY-STATE SEEPAGE STABILITY ANALYSIS OF UPSTREAM SLOPE
LAKE PETIT DAM

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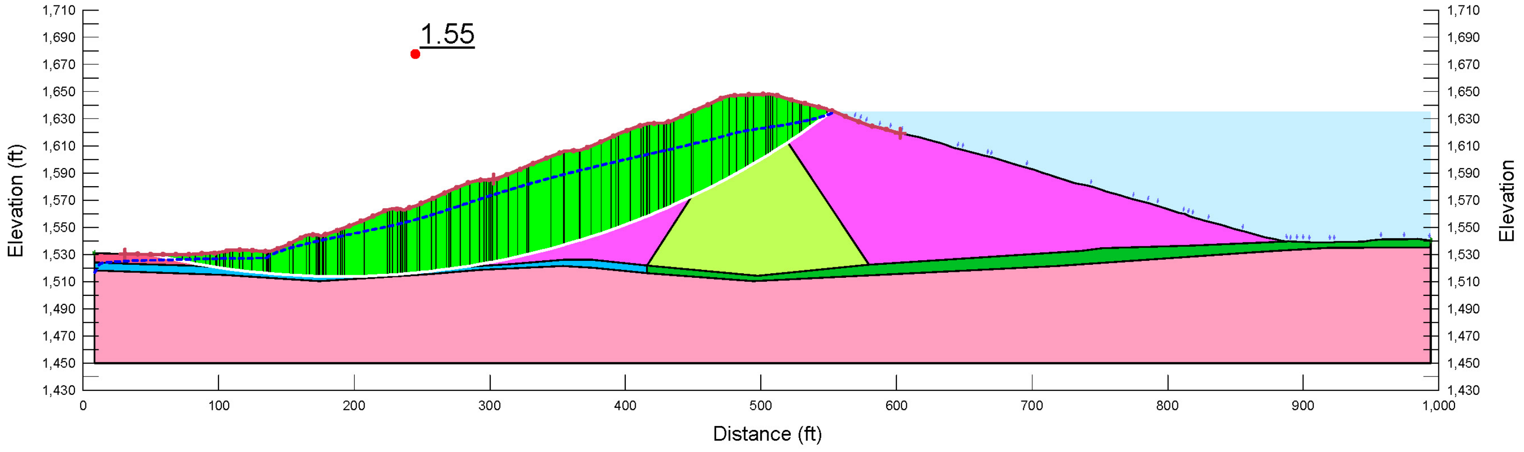
PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3-2

Steady-State Seepage Pseudostatic Stability Results

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.038 g for an allowable displacement of 100 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.038 g)

LAKE PETIT DAM

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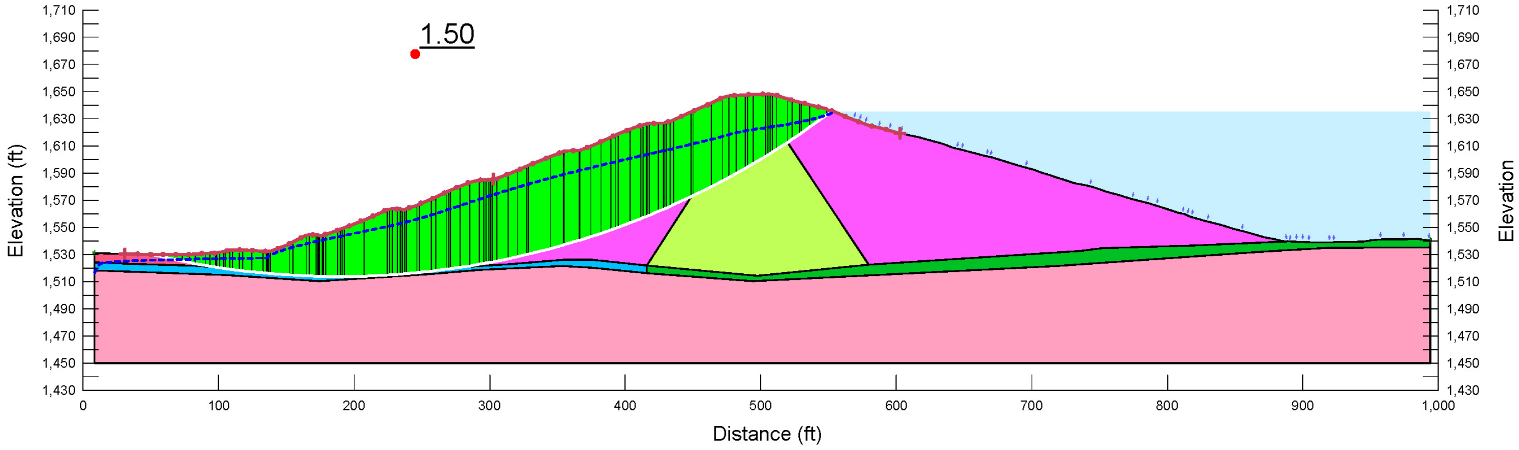
Figure 3-3

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.047 g for an allowable displacement of 70 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.047 g)

LAKE PETIT DAM

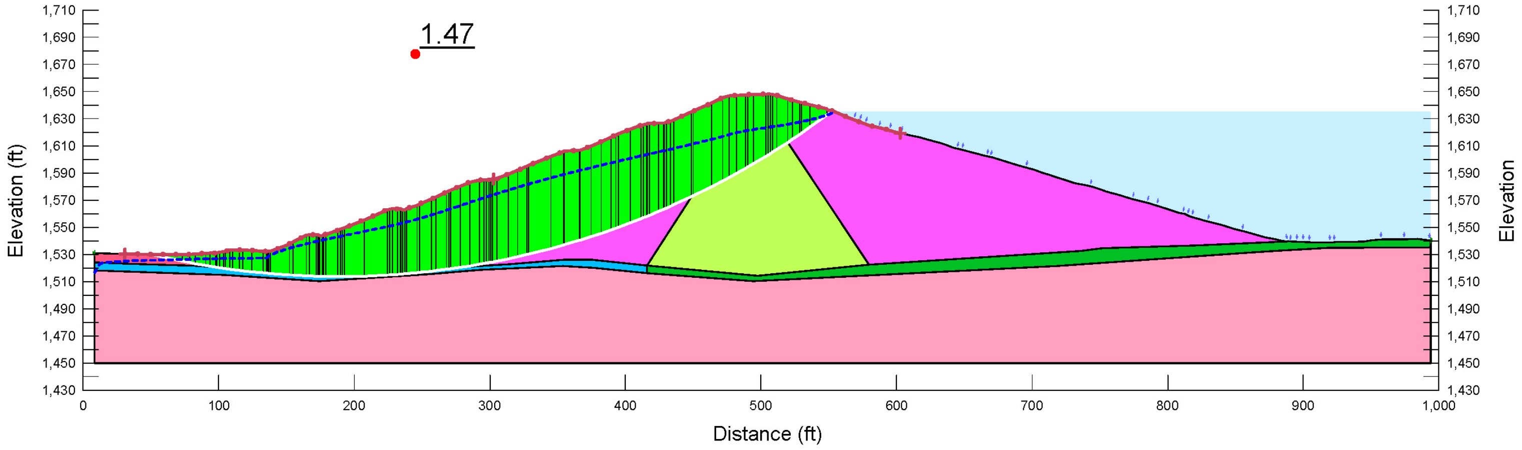
Geosyntec consultants

Figure 3-4

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.050 g, which is GS SDP minimum required seismic acceleration.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.050 g)
LAKE PETIT DAM

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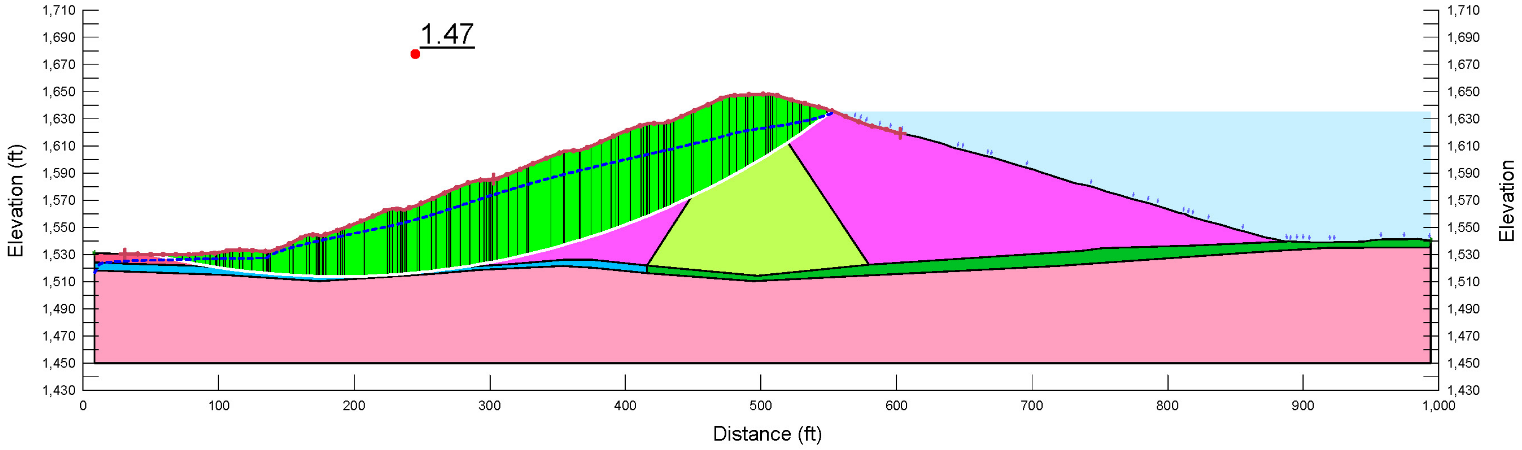
PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3-5

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.047 g for an allowable displacement of 60 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.054 g)
LAKE PETIT DAM

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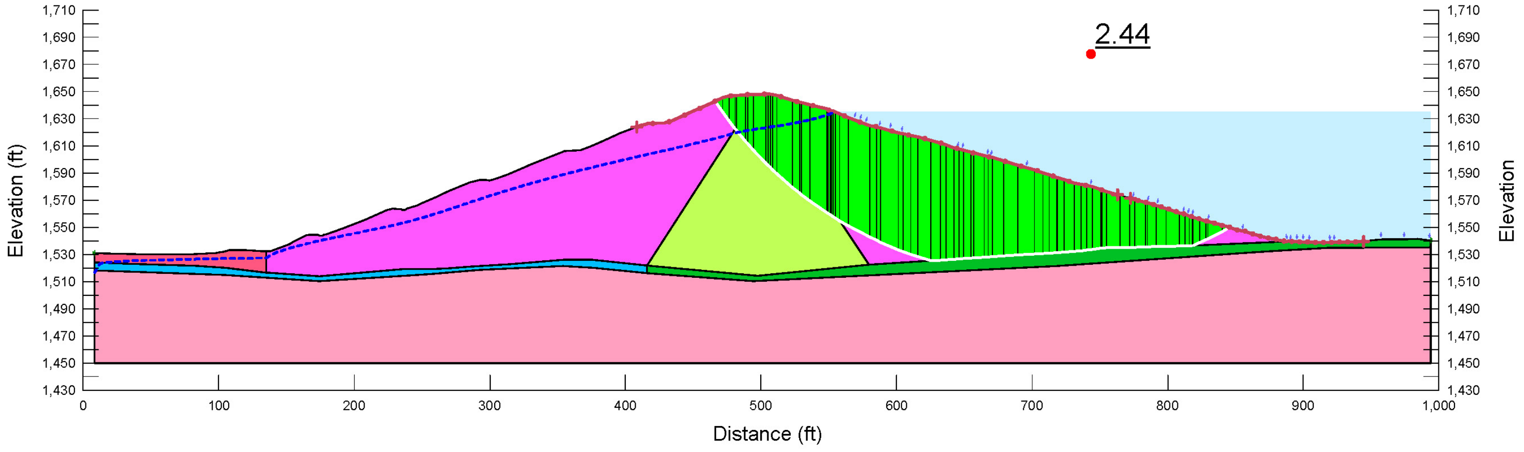
PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3-6

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.047 g for an allowable displacement of 60 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.054 g)

LAKE PETIT DAM

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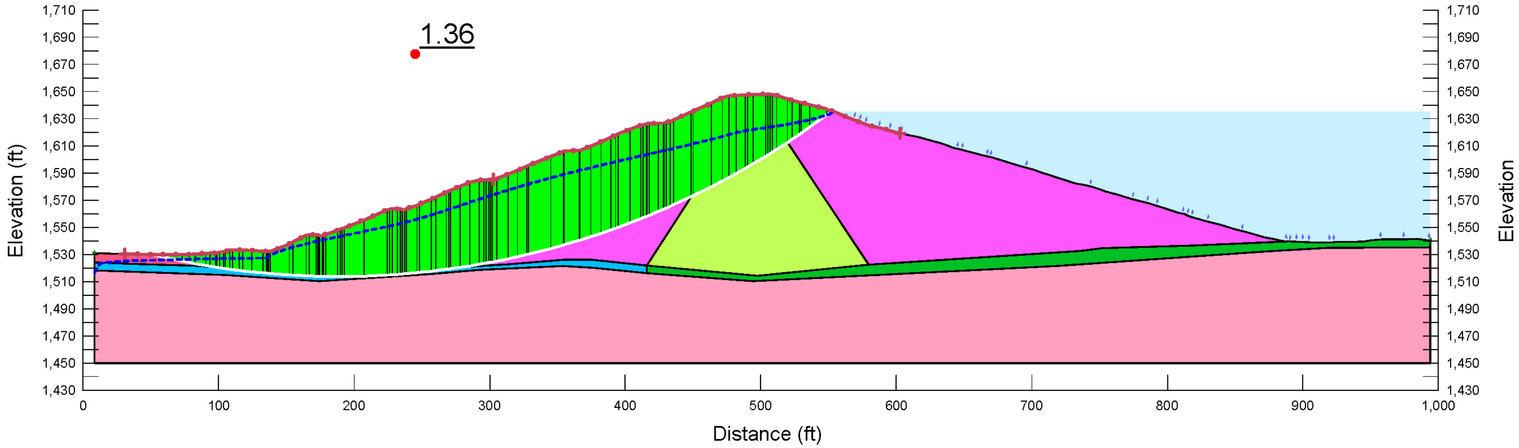
Figure 3-7

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.081 g for an allowable displacement of 30 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.081 g)

LAKE PETIT DAM

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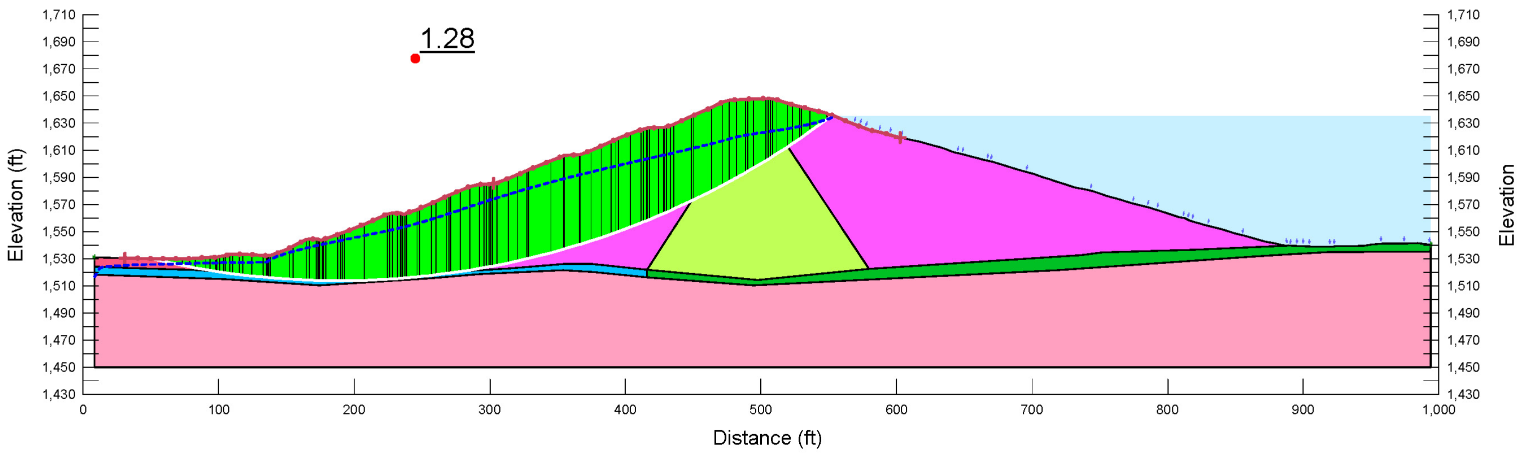
PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3-8

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.101 g for an allowable displacement of 20 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.101 g)

LAKE PETIT DAM

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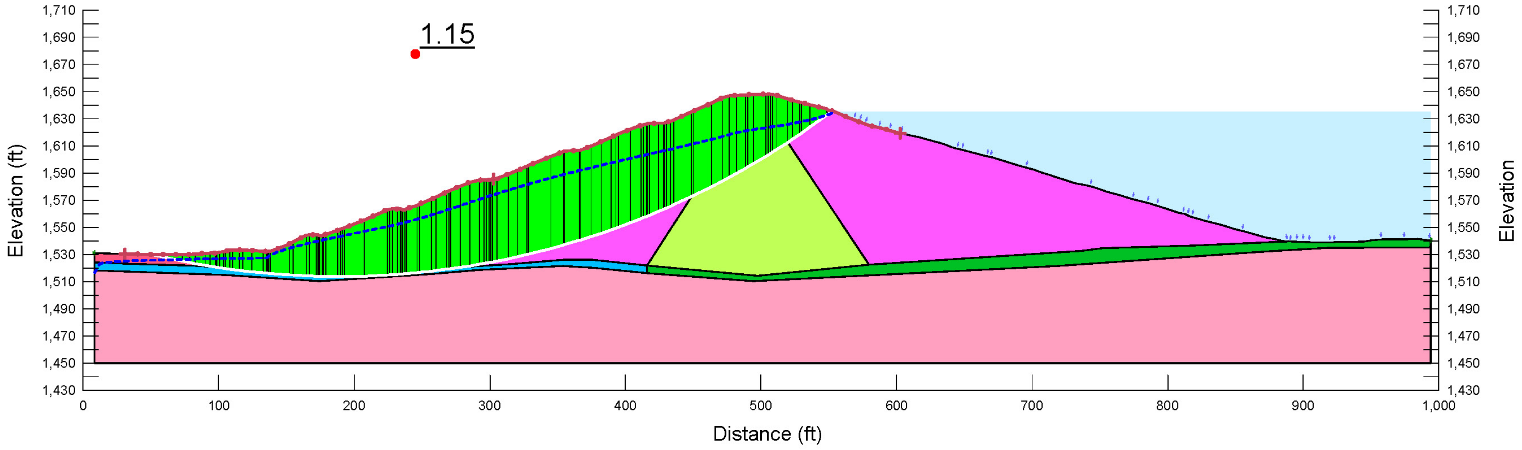
PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3-9

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.140 g for an allowable displacement of 10 cm.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.140 g)

LAKE PETIT DAM

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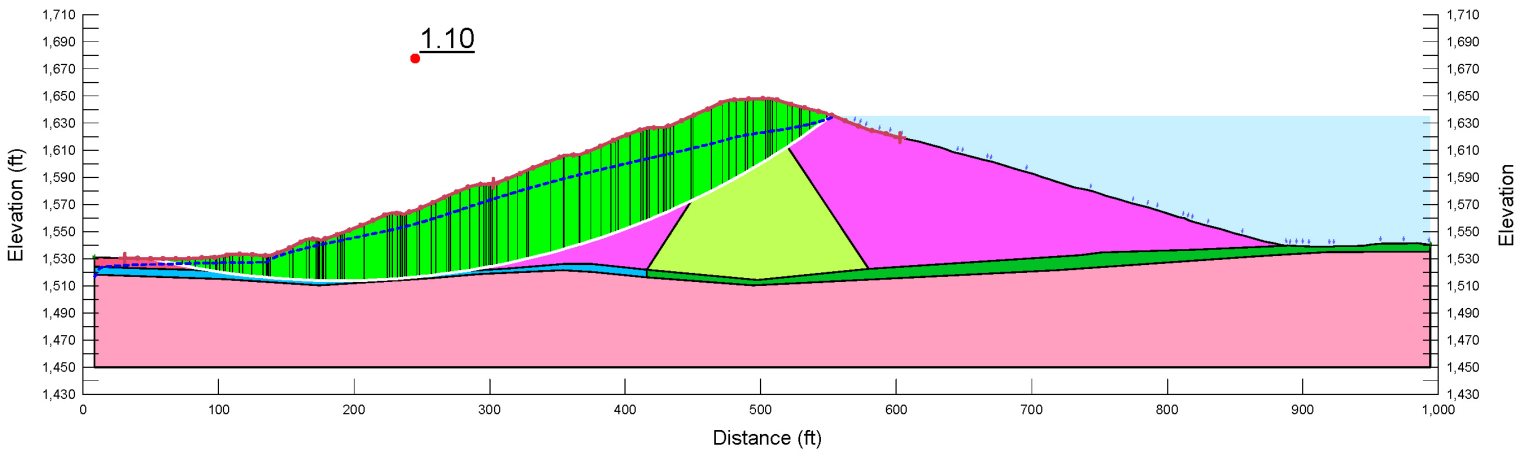
Figure 3-10

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.160 g, which is GS SDP minimum required factor of safety of 1.1.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.160 g)
LAKE PETIT DAM

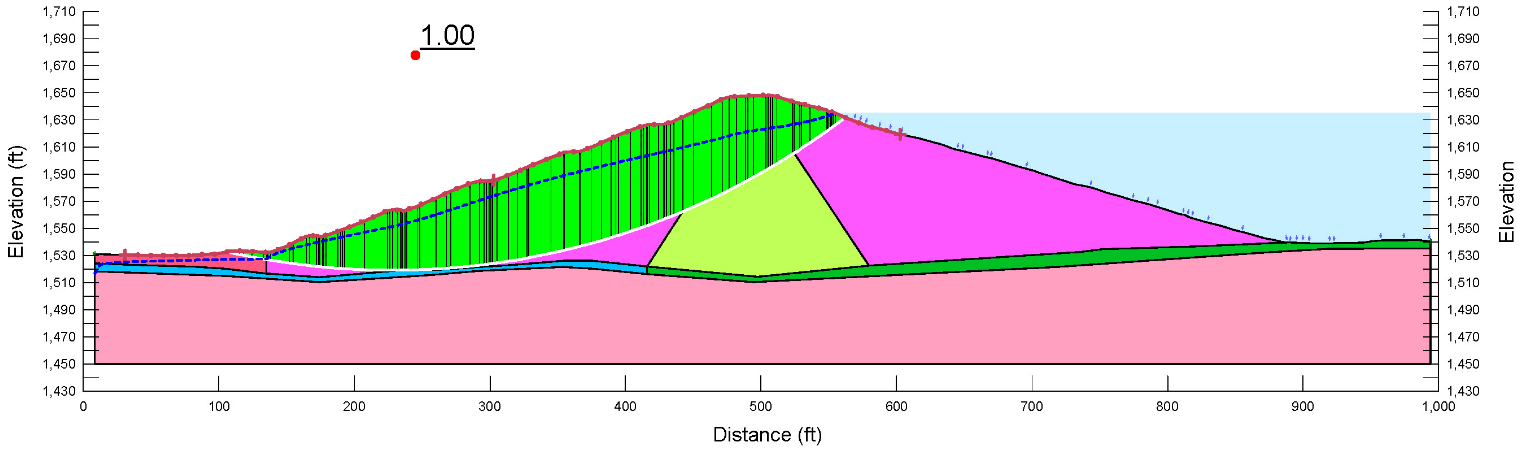
Geosyntec consultants

Figure 3-11

PROJECT NO. TN9418

DATE: FEBRUARY 2023

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟥	Bedrock	Bedrock (Impenetrable)			
🟩	Dam Core (Undrained)	Mohr-Coulomb	130	1,000	23
🟪	Dam Shell (Undrained)	Mohr-Coulomb	125	1,000	23
🟦	Saprolite - D/S (Undrained)	Mohr-Coulomb	125	0	35
🟩	Saprolite - U/S (Undrained)	Bedrock (Impenetrable)			
🟥	Soil below ball field (Undrained)	Mohr-Coulomb	125	0	32



Notes:
Cross-Section A-A
Pseudostatic Analysis of Downstream Slope
The pseudostatic analysis was performed with a seismic coefficient K_s of 0.200 g, which was performed to identify the yield coefficient K_y.

PSEUDOSTATIC SLOPE STABILITY ANALYSIS OF DOWNSTREAM SLOPE (K_s=0.200 g)
LAKE PETIT DAM

Geosyntec consultants

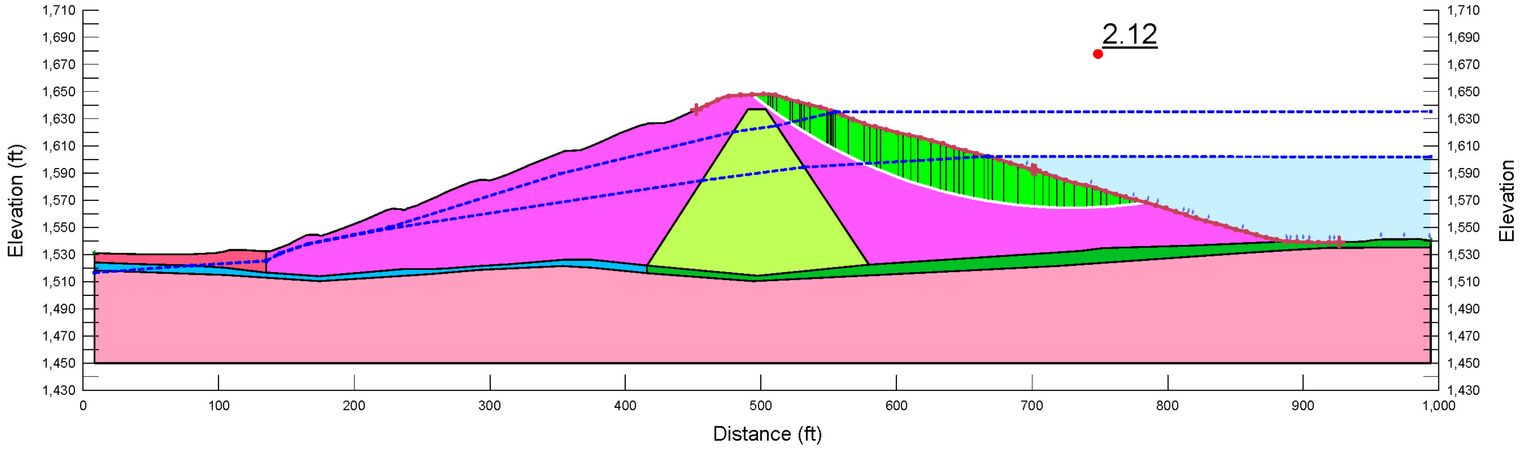
PROJECT NO. TN9418

DATE: FEBRUARY 2023

Figure 3-12

Rapid Drawdown Stability Results

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)	Piezometric Line	Piezometric Line After Drawdown
🟥	Bedrock (Duncan)	Bedrock (Impenetrable)						1	2
🟩	Dam Core (Duncan)	Mohr-Coulomb	130	0	32	1,000	23	1	2
🟪	Dam Shell (Duncan)	Mohr-Coulomb	125	0	34	1,000	23	1	2
🟦	Saprolite - D/S (Duncan)	Mohr-Coulomb	125	0	35	0	35	1	2
🟩	Saprolite - U/S (Duncan)	Bedrock (Impenetrable)						1	2
🟥	Soil below ball field (Duncan)	Mohr-Coulomb	125	0	32	0	32	1	2



Notes:
Cross-Section A-A
Rapid Drawdown Analysis of Upstream Slope
Analysis assumes a sudden release of two-thirds of the reservoir volume, from El. 1,635.5 to 1,602 ft.

RAPID DRAWDOWN SLOPE STABILITY ANALYSIS OF UPSTREAM SLOPE
LAKE PETIT DAM

Geosyntec consultants

Figure 3-13

PROJECT NO. TN9418

DATE: FEBRUARY 2023