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8 December 1998

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Environmental Engineer
Safe Dams Program
Environmental Protection Division
Georgia Department of Natural Resources
Atlanta Tradeport, Suite 110
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Atlanta, Georgia 30354

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Subject: Transmittal of Report Titled "Evaluation of Stability and Rehabilitation Measures"
Lake Petit Dam

Dear Tom:

On behalf of the Big Canoe Property Owner's Association, GeoSyntec Consultants, Atlanta (GeoSyntec) is pleased to submit the enclosed report for your review and comment.

The report presents the results of our re-evaluation of steady-state seepage and slope stability conditions at the Lake Petit Dam, located within the Big Canoe development. The report also presents revised recommendations for various rehabilitation measures designed to improve existing conditions and increase calculated factors of safety against embankment stability failure under both static steady-state seepage and seismic loading conditions.

Since it is Big Canoe POA's intention to commence development of the detailed design for the recommended rehabilitation measures concurrently with the Safe Dams Program's review of this report, we would greatly appreciate any early feedback on comments that you can provide on this report. It is our intent to develop the design and secure all necessary approvals to allow construction activities to commence in April 1999 as previously agreed.

We appreciate your cooperation in this matter and look forward to hearing from you soon. If you have any questions in the matter, please call Neil Davies at (404) 705-9500.

Sincerely,

R. Neil Davies

R. Neil Davies, C.Eng., MICE, P.E.
Project Manager

Robert C. Bachus

for Rudolph Bonaparte, Ph.D., P.E.
Principal

GL00625-15/GA981181

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Prepared for:

Big Canoe Property Owners Association
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Big Canoe, Georgia 30143

EVALUATION OF STABILITY AND REHABILITATION MEASURES LAKE PETIT DAM

BIG CANOE, GEORGIA

Prepared by:

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Project Number GL0625-15

December 1998

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EXECUTIVE SUMMARY

Overview

This report presents the results of geotechnical engineering evaluations performed by GeoSyntec Consultants (GeoSyntec) for the existing Lake Petit Dam located within the Big Canoe development, Pickens County, Georgia. In this report, an assessment of existing conditions is presented. Based on this assessment, rehabilitation measures are recommended which are intended to enhance the stability of the existing dam under both static and seismic conditions.

Lake Petit Dam is a 115-ft (35-m) high zoned earth embankment constructed in 1972. The Category I dam includes 2.5H:1V (horizontal:vertical) downstream slopes with 10-ft (3m) wide benches at approximately 20-ft (6-m) vertical intervals and 3.5H:1V upstream slopes. During a routine inspection of the dam conducted in March 1996 by the Georgia Environmental Protection Division (GaEPD) Safe Dams Program several potential deficiencies in the condition of the dam were identified. Subsequent geotechnical investigation and engineering evaluations were performed (by others) and rehabilitation measures were proposed involving construction of a substantial earthen downstream slope buttress.

GeoSyntec was retained by the Big Canoe Property Owners Association (POA) to perform an independent review of the proposed rehabilitation measures and to assess the feasibility of incorporating alternative measures which would be less intrusive. As part of its scope of work, GeoSyntec conducted field investigations, installed field instrumentation, and completed a laboratory testing program on samples of the embankment material. Using the results of the field and laboratory investigation activities, a site physical conditions model was developed. A steady-state seepage evaluation and an assessment of static and seismic slope stability were conducted.

Analysis results indicate that a high phreatic surface may develop near the downstream toe of the dam at the time of seasonal high water (i.e., in the spring). These results are consistent with site observations. Analysis results also indicate that calculated slope stability factors of safety meet the requirements of the newly adopted GaEPD Safe Dams Program rules, with the exception that surficial stability is inadequate in the area where the high phreatic surface may develop. To remedy this situation, rehabilitation alternatives are recommended that lower the phreatic surface in the lower portion of the dam. Implementation of the recommended rehabilitation measures, which involves the use of trench drains, paved benches, and bench drains will result in calculated factors of safety for all potential failure surfaces which meet the requirements identified in the new adopted GaEPD Safe Dams Program rules.

The results of GeoSyntec's investigations, engineering evaluations, and recommendations are presented in this report. The report was developed in essentially three phases. The first phase included an assessment of the existing physical conditions of the dam and involved the execution of a field investigation program, a laboratory testing program, a field instrumentation program, and the development of a site physical conditions model. The second phase included analyses of the seepage and slope stability conditions of the dam. The third and final phase included an evaluation and selection of rehabilitation alternatives to enhance stability of the dam. This report has been developed to present to the GaEPD the results of these three phases of work. The next step in the process is development of the detailed engineering design for the proposed rehabilitation measures.

Phase I - Assessment of Site Conditions

Field Investigation Program - Six geotechnical borings were advanced through the dam along the centerline and on both sides of the centerline of the downstream face of the dam to provide additional geotechnical data on the soils used to construct the dam. The locations of these borings were selected to provide aerial distribution across the site. Representative Shelby tube samples were obtained at selected locations in each boring using both conventional methods and a Pitcher barrel sampler. Seismic downhole testing was conducted in two of the borings to obtain the shear wave velocities of the compacted soils within the dam. The investigation confirmed that core and shell materials consist of micaceous silty soils. The core had a higher silt and clay content than the shell.

During the field investigation program, in-situ standard penetration tests (SPTs) were conducted in each borehole. The measured SPT blowcount values were used to obtain approximate values of friction angle using industry-accepted correlations. The results of these tasks indicated that the friction angle of the dam core and shell soils range from 32 to 48 degrees. The shear wave velocity tests results, which ranged from 457 to 1501 ft/s (139 to 458 m/s), were consistent with other results obtained for compacted earth fills.

Laboratory Testing Program - Using the samples obtained during the field investigation program, a laboratory testing program was conducted that included six index tests (i.e., Atterberg limits and particle size distribution) and 16 triaxial compression tests. The index test results indicated that the shell specimens all had a Unified Soil Classification System (USCS) classification of SM, silty sand, and had a clay content of less than 6 percent. In contrast, the core specimens all had a USCS classification of ML, low plasticity silt, and had clay contents between 10 and 20 percent. Isotropically consolidated undrained triaxial compression tests were conducted to assess both effective-stress (drained) and total-stress (undrained) shear strength parameters. The effective-stress and total-stress shear strength parameters obtained from the test results were typical of values for the type of compacted earth fill materials identified at the site.

Field Instrumentation and Water Level Monitoring - Using historical information provided by four piezometers installed in 1974 and specific results from twelve vibrating wire piezometers installed between 1997 and 1998, GeoSyntec evaluated the measured piezometric water levels in Lake Petit Dam. In addition, GeoSyntec installed seven additional standpipe piezometers during the field investigation program and monitored the water levels to provide additional piezometer pressure readings at strategic points within the dam cross-section. The water level monitoring results confirm that the dam flow regime is consistent with that of a zoned earth embankment. These results also indicate a significant seasonal effect in which water levels respond to precipitation events, suggesting recharge infiltration through the downstream slope face. The results were used in the development of the site physical conditions model and in assessing the benefits of various rehabilitation alternatives for the dam.

Site Physical Conditions Model - Field and laboratory data from the GeoSyntec investigation and results from previous investigations at the site were used by GeoSyntec to develop a physical conditions model of the Lake Petit Dam site. This model was used in subsequent static and seismic slope stability analyses and in rehabilitation scenario evaluations. The following provides a summary of the site physical conditions:

- Dam construction: compacted and non-stratified zoned earth fill consisting of a silty sand shell and a low plasticity sandy silt core;
- Flow regime through dam: consistent with conventional flow through a zoned earth embankment;

- Material properties for analysis: unit weight, shear strength, and hydraulic conductivity characteristics were developed for the materials from the compacted shell and core and the native saprolite soils and bedrock; and

- Water levels within dam: an estimated maximum water level (EML) scenario was developed using the water-level data to represent seasonal high piezometric levels in the dam.

Phase II: Seepage and Slope Stability Analyses

Seepage Analyses - Finite element seepage analyses were performed to establish grid of porewater pressure values within the dam to be used in subsequent slope stability analyses. Soil properties, specifically for hydraulic conductivity, were used as inputs to the analysis and were adjusted to obtain a "calibrated" porewater pressure distribution which closely corresponds to a recent set of ~~piezometer measurements~~. Soil properties used to develop the calibrated porewater pressure distribution and actual water level data corresponding to seasonal high water levels were then used to establish a best-fit porewater pressure distribution for the EML scenario. This distribution is a conservative representation of seasonal high water levels and was subsequently used in the slope stability analyses.

Slope Stability Analyses - Using the dam geometry, site physical conditions model, and the EML porewater pressure distribution described in previous sections of the report, static and seismic slope stability analyses were conducted. Analyses were conducted to assess embankment stability under the existing conditions and the post-rehabilitation scenario (subsequently discussed). These analyses considered a full range of potential slip surfaces except for surficial (shallow) surfaces. Surficial surfaces are discussed subsequently. For seismic analyses, a seismic acceleration of 0.183g was used in a conventional psuedo-static analysis. Calculated factors of safety from these analyses are summarized as follows:

Analysis	GaEPD Target [1]	Existing Conditions @ EML	Post-Rehabilitation Conditions
Static Condition	1.50	1.52	1.74
Seismic Condition	1.10	1.46	>1.46

Note [1]: Minimum calculated factor of safety from the newly adopted GaEPD Safe Dams Program rules.

Analysis results indicate that the calculated factors of safety for embankment stability exceed the minimum values identified in the newly adopted GaEPD Safe Dams Program rules. Limited analyses were also conducted to assess the surficial stability of the downstream slope face. It is noted that the calculated factors of safety will be less than 1.5 with the EML porewater pressure scenario for potential surficial slip surfaces near the toe of the dam. This is due to the location of the phreatic surface for the EML scenario which is at or near the downstream slope face. Surficial stability issues are typically addressed during routine maintenance activities at a facility. Nevertheless, rehabilitation measures were considered to improve surficial stability. These measures are expected to also improve the stability of downstream embankment. In fact, analysis results indicate that the calculated slope stability factor of safety for both embankment stability and surficial stability under post-rehabilitation condition exceeds the minimum values presented in the GaEPD Safe Dams Program rules.

Phase III: Rehabilitation Alternatives Assessment and Recommendation

Rehabilitation Alternatives - An evaluation was made of several potential rehabilitation alternatives to enhance stability of Lake Petit Dam. Specific rehabilitation was shown to be necessary near the downstream toe due to the location of the seasonal high phreatic surface. Alternatives were evaluated with respect to the following six criteria: (i) improvement of calculated factor of safety; (ii) constructability; (iii) implementation schedule; (iv) ability to monitor effectiveness; (v) capital cost; and (vi) operation and maintenance cost. A total of 14 rehabilitation alternatives were identified and evaluated. A combination of three alternatives (i.e., the use of trench drains, paved benches, and bench drains) was selected and steady-state slope stability analyses were conducted to evaluate the effects of the rehabilitation measures. As shown previously, the use of these rehabilitation techniques significantly increase the calculated factors of safety and are recommended for implementation at Lake Petit Dam. A schedule is presented for implementing the recommendations made in this report and for monitoring the performance of the dam.

Conclusion

Based on the results presented in this report, the existing conditions at Lake Petit Dam meet or exceed the slope stability requirements of the newly adopted GaEPD Safe Dams Program rules in all areas, except for surficial stability near the downstream toe of the dam where the location of the phreatic surface is at or near the ground surface. Rehabilitation alternatives are recommended to reduce infiltration of precipitation into the downstream face and to lower the phreatic surface in this area of the dam. Implementation of these measures will result in an increase of the calculated slope stability factors of safety under both static steady-state seepage conditions and seismic conditions. A schedule is presented for implementing the rehabilitation alternatives and monitoring performance of the dam. When these rehabilitation measures are implemented, the Lake Petit Dam will meet or exceed the slope stability requirements identified in the newly adopted GaEPD Safe Dams Program rules.

1. INTRODUCTION

1.1 Terms of Reference

This report was prepared by GeoSyntec Consultants (GeoSyntec), Atlanta, Georgia on behalf of the Big Canoe Property Owners Association (POA), Big Canoe, Georgia. The report presents the results of geotechnical engineering evaluations performed by GeoSyntec for the existing Lake Petit Dam, located within the Big Canoe development, Pickens County, Georgia. This report was prepared under the direction and peer review of Dr. Rudy Bonaparte, P.E. The report was prepared by Mr. R. Neil Davies, P.E., Dr. Gary R. Schmertmann, P.E., Dr. Paul J. Sabatini, P.E., Mr. Dennis Vander Linde, P.E., and Mr. Daniel G. Pass.

1.2 Purpose

The purpose of this report is to present the results of the geotechnical engineering evaluations performed for Lake Petit Dam. The report also addresses recommended rehabilitation measures intended to enhance the stability of the existing structure under both static and seismic conditions. The geotechnical and engineering evaluations presented herein were performed in general accordance with the Scope of Work presented in a letter from GeoSyntec addressed to Mr. Dallon Thomas Woosley, P.E. of the Georgia Environmental Protection Division Safe Dams Program (GaEPD Safe Dams Program) dated 18 September 1998.

1.3 Background

The Lake Petit Dam is located within the Big Canoe development on Petit Creek, approximately 5.8 miles (9.3 km) upstream of Marblehill, Georgia. The reservoir formed by the dam has a surface area of approximately 104 acres (42 ha) at a normal pool elevation of 1635 ft mean sea level (MSL). Total storage for the reservoir is approximately 3,000 acre-ft (3.7 Mm^3).

Lake Petit Dam was constructed in 1972 as a zoned earth embankment consisting of a central clayey silt core and predominantly silty sand embankment shells. The dam has a maximum height of 115 ft (35 m) measured vertically from the downstream toe, a crest length of approximately 880 ft (268 m), and a crest width of approximately 35 ft

(11 m). The upstream slope of the dam is inclined at 3.5H:1V (horizontal:vertical). The downstream shell is inclined at 2.5H:1V with 10 ft (30 m) benches at approximate 20 ft (6.1 m) vertical intervals. Design drawings for the dam were prepared by Baldwin and Cranston Associates [1971]. The dam is permitted as a Category I Dam under Chapter 391-3-8 of the state code "Rules for Dam Safety".

Based on a routine periodic inspection, several potential deficiencies in the condition of the dam were reported by representatives of the GaEPD Safe Dams Program in a report dated 15 April 1996. Specifically, items 5 through 8 of that report addressed seepage-related concerns, and possible deficiencies in the condition of the principal spillway. The April 1996 report recommended that the owner, Big Canoe POA, secure the services of an engineer to evaluate and report on the potential deficiencies. Accordingly, Big Canoe POA retained Jordan Jones and Goulding, Inc. (JJ&G) to perform a reconnaissance of the dam. Subsequently, JJ&G retained Piedmont Geotechnical Consultants, Inc. (Piedmont) under subcontract, to assist with a more detailed geotechnical investigation and engineering evaluation of the dam.

JJ&G and Piedmont conducted a phased investigation of the dam that included the following key elements:

- review of historical information;
- site observations and inspections;
- geotechnical investigations focused on establishing the phreatic surface within the downstream slope of the dam; and
- evaluation of the stability of the dam under both static conditions with steady-state seepage and seismic conditions.

The results of these investigations and evaluations are described in reports by Piedmont dated 29 May 1997 and 1 April 1998. In summary, the evaluations performed by Piedmont utilized conservatively estimated (assumed) soil shear strength values and measured piezometric levels to calculate minimum factors of safety against a downstream embankment slope stability failure under both static steady-state seepage and seismic conditions. The reported minimum factors of safety were 1.28 for static steady-state seepage conditions and 0.75 for seismic conditions. These factors of safety are less than the minimum values acceptable under newly adopted GaEPD Safe Dams Program rules (i.e., 1.5 and 1.1, respectively). Based on these results, Piedmont evaluated various

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rehabilitation measures for the dam and recommended the construction of a blanket drain and earthen toe buttress over a significant portion of the existing downstream slope in order to increase slope stability factors of safety to the required minimum values.

Subsequent to Piedmont's initial recommendation for rehabilitation of the dam, additional investigations were conducted by Piedmont, including the installation of two piezometers close to the toe of the existing slope. Piedmont utilized the additional data to refine the stability evaluations. Also, JJ&G developed a preliminary design and cost estimate for the recommended blanket drain and toe buttress.

While the proposed rehabilitation measures provide an adequate solution to the potential deficiencies, the importation and placement of the large quantities of fill required to construct the toe buttress would adversely impact the Big Canoe community. Community concerns include: (i) heavy dump truck traffic; (ii) loss of community amenities including a ball field, jogging trail, picnic area, and road at the toe of the dam; (iii) encroachment on property adjacent to the dam; (iv) environmental impacts in the vicinity of the dam and at borrow location(s); and (v) the high costs involved. Therefore, prior to finalization of the rehabilitation design, Big Canoe POA retained GeoSyntec to perform an independent review of the proposed rehabilitation measures, with the goal of identifying and evaluating potentially less intrusive rehabilitation measures.

Based on a careful review of project background information and the previous engineering evaluations, GeoSyntec developed an initial Scope of Work to perform the reviews and evaluations requested by Big Canoe POA. Representatives from GeoSyntec and Big Canoe POA presented the initial Scope of Work to Mr. Woosley and Mr. Feigle of the GaEPD Safe Dams Program during a meeting on 19 August 1998. Subsequent to that meeting, GeoSyntec developed a more formal Scope of Work which was submitted to the GaEPD Safe Dams Program in a letter dated 18 September 1998. This Scope of Work focused on a re-evaluation of existing conditions using site-specific soil strength data and additional piezometric data, combined with the use of more sophisticated engineering analysis techniques. The portions of the Scope of Work relating to gathering site-specific soil strength data and additional piezometric data were developed with assistance from Piedmont. How The re-evaluation was designed to expand upon the initial evaluations performed by Piedmont. Since the re-evaluation incorporates site-specific soil strength data and piezometric data, the use of more sophisticated engineering analysis methods is justified to more accurately estimate the minimum factors of safety against slope stability failures under both static steady-state seepage and seismic conditions.

The remainder of this report and the enclosed supporting appendices present the findings of the re-evaluation. The main topics addressed in this report are:

- additional field investigation;
- embankment fill shear strength evaluation;
- steady-state seepage evaluation;
- static and seismic slope stability evaluation;
- evaluation of dam rehabilitation alternatives; and
- schedule for implementing dam rehabilitation measures.

2. FIELD INVESTIGATION

2.1 Design of Field Investigation Program

The field investigation program was designed in accordance with the 18 September 1998 Scope of Work to gather site-specific information on subsurface stratigraphy, soil shear strength, soil shear wave velocity, and piezometric levels within the downstream shell. This information was generally intended to supplement that previously collected by Piedmont. In addition, a key element of the program was the recovery of intact soil samples for laboratory shear strength testing. The results of these laboratory tests provide site-specific information on soil shear strength for the project. During design of the field investigation program, GeoSyntec consulted with Piedmont regarding a number of issues, including: the optimal numbers of new borings, drilling methods, sampling intervals, and techniques for recovery of intact soil samples.

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The field investigation program involved the drilling of six geotechnical borings in the downstream face of the dam. The borings are identified as G-1A, G-1B, G-2, G-3, G-4, and G-5 and are located as shown in Figure 2-1. The rationale for the geotechnical boring program is presented below.

- The borings were performed by the same drilling company used for previous borings into the dam, i.e., Atlanta Testing and Engineering (AT&E). The same driller and drill rig was engaged. These measures were used to achieve a high degree of consistency between the previous and current boring programs.

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- Several of the borings were located along the centerline of the dam, the same alignment used for the previous borings. Others were located on both sides of the centerline to provide an areal distribution of subsurface information and soil samples.

- Sampling intervals were selected such that intact (tube) soil samples were obtained over a broad range of elevations within the dam fill. Sampling intervals for tube samples were fixed at a regular spacing to reduce possible bias in sample collection (i.e., sample depths were not altered in the field if difficult sampling conditions were present). These measures promote collection of a representative group of tube samples for laboratory shear strength testing.

- Boring G-2 was located on the bench approximately 25 ft (7.6 m) upstream of the transverse dam foundation drain shown on the dam design drawings. Comparison of piezometric levels in boring G-2 with those in borings G-3 and P6 (approximately 40 ft (12.2 m) downstream of the drain) provide information on the effect of the drain on piezometric levels within the dam. In addition, comparison of piezometric levels in borings G-3 and P6, which are located on the same bench, provide information on variation of piezometric levels in the dam in the transverse direction.

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- Borings G-1B and G-2 were advanced down to competent bedrock to provide information on the thickness of dam fill, underlying native soils, and partially-weathered rock along the dam centerline.

2.2 Work Performed

2.2.1 Scope and Schedule

The field investigation program for Lake Petit Dam was performed between 2 and 16 October 1998. The investigation activities included: (i) geotechnical borings; (ii) soil sampling; (iii) piezometer installation; (iv) downhole seismic testing; and (v) surveying.

2.2.2 Geotechnical Borings

Subsurface conditions of the Lake Petit Dam were investigated by drilling six geotechnical borings in the downstream face of the dam. These borings are designated G-1A, G-1B, G-2, G-3, G-4, and G-5 and are located as shown in Figure 2-1. These borings were located along the approximate centerline of the dam, shown by the section line A1, on benches where no prior borings were drilled. One boring, G-3, was located approximately 115 ft (35 m) west of the approximate centerline, above the original valley bottom. Two other borings were located above the dam abutments. Drilling activities were supervised on a full-time basis by an engineer or geologist from GeoSyntec. The GeoSyntec representative classified soils samples and prepared written logs of the subsurface conditions encountered.

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geotechnical engineer, w/o Piedmont

The geotechnical borings were drilled by AT&E using a CME 750 truck-mounted drilling rig. The following two drilling techniques were used: (i) hollow stem auger

(HSA); and (ii) bentonite mud rotary drilling. The geotechnical borings ranged in depth from 47 to 114 ft (14.3 to 34.7 m) and were terminated in dam fill material, native soil, or bedrock. A summary of the depth and drilling technique for each boring is provided in Table 2-1. Geotechnical boring logs are presented in Appendix A.

2.2.3 Soil Sampling

Samples were recovered by split spoon sampling using Standard Penetration Test (SPT) procedures (in accordance with ASTM D 1586, except that an automatic hammer was used), pushed thin wall (Shelby) tube sampling (ASTM D 1587), and tube sampling procedures using a Pitcher barrel sampler. The Pitcher barrel sampler provided for higher intact sample recoveries in portions of the dam fill with relatively high gravel content and was used for lower samples in Borings G-1A, G-2, G-4, and G-5. Twenty-seven tube samples were obtained out of 35 attempts at advancing tubes. Sample depths are shown in Figure 2-2 and details are provided in the geotechnical boring logs (Appendix A). Note

2.2.4 Piezometer Installation

open well.

Seven standpipe piezometers were installed at locations within the shell, core, and underlying native soil to obtain water pressure information and to define the seepage paths within the dam. These seven piezometers were installed within the G-1A, G-1B, G-2, and G-3 boreholes. Depths and screened intervals for these piezometers are shown on Figure 2-3. Similar information for previously installed (by others) piezometers is also shown on Figure 2-3. Piezometer depths, screened intervals, and other construction information are provided on construction summary logs presented in Appendix A.

2.2.5 Seismic Downhole Testing

Seismic downhole testing was performed on 15 October 1998 to obtain site specific shear wave velocities of the dam shell and core materials. Tests were performed in grouted 4 in. (102 mm) diameter PVC casing installed in boreholes G-1B and G-5. Do not agree. Tests were performed to depths of 103 ft (31 m) and 66 ft (20 m) in boreholes G-1A and G-5, respectively.

The downhole seismic testing was performed using the true interval method. For each test, measurements were made using three triaxial geophones separated vertically by 5 ft (1.5 m) and temporarily fixed in the borehole. Tests were performed and repeated at normally 10-ft (3-m) intervals. Shear and compression wave velocities were calculated using 5 and 10 ft (1.5 and 3 m) intervals between geophones. Further information regarding this testing is provided in a letter report prepared by Law Engineering and Environmental Services, Inc., entitled "*Report of Seismic Downhole Testing*" and dated 21 October 1998. The Law report is contained in Appendix B of this document.

2.2.6 Surveying

A global positioning system (GPS) survey was performed on 15 October 1998 to locate borings and piezometer casings and to verify the dam surface geometry used in stability and seepage analyses. The GPS survey was performed by GeoSyntec personnel. Elevations for borings and piezometer casings are provided on the logs included in Appendix A.

– type/accura

surveyed slopes.

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2.3 Results

2.3.1 Description of Subsurface Conditions

General descriptions of dam fill materials and underlying soil and rock were developed based on: (i) logs for borings drilled as part of the October 1998 GeoSyntec investigation program; and (ii) material specifications in engineering plans prepared for construction of the dam. The locations of borings within a cross section view of the dam are shown in Figure 2-2. The development of the cross section geometry in this figure showing shell, core, saprolite and bedrock is described in Section 5.1. Specifications for dam fill materials found in engineering drawings allowed for use of clayey silts and silty clays for core construction and silty sands and weathered rock for shell construction. Native soil and weathered bedrock material was identified in boring logs generally as saprolite, based on a well defined structure characteristic of less weathered residual soil.

Note Shell material was encountered in all borings. Boring logs generally describe this material as follows:

"Silt, micaceous, with trace to some fine sand, and many weathered gneiss fragments up to 2 in. in diameter."

Core material was encountered in borings G-1B and G-5. Boring G-1B encountered only about 25 ft (7.6 m) of the core. Boring G-1B was drilled through approximately 50 feet (15.2 m) of core material near the crest of the dam. Boring logs generally describe this material as follows: conflicting

"Silt, micaceous, with trace to some clay, trace sand and occasional wood and root fragments."

No significant fill stratification was identified within either the core or the downstream shell.

The native soil underlying the dam was encountered in borings G-1B, G-2, and G-4. The elevation of the top of this material was found to be at or below the original ground surface existing prior to construction of the dam, based on a comparison to the pre-construction site topography.

Bedrock was encountered in borings G-1B and G-2 at 4 to 6 ft (1.2 to 1.8 m) below the surface of the native soil material.

2.3.2 SPT Correlations to Soil Friction Angle

As part of the evaluation of the shear strength of the embankment fill and underlying native soil, two empirical correlations were used to estimate the effective-stress shear strength (in terms of the parameter effective-stress peak friction angle) of the fill from SPT blow count (N) values. These correlations were developed for sandy to silty sandy soils, such as comprise the embankment. The correlations become less reliable as the gravel content of the sandy soil increases. The gravel content of the embankment material was measured to range from 2 to 12 percent, as indicated in Table 3-1. This amount of gravel is considered not insignificant as it relates to the correlations. Nonetheless, GeoSyntec considers use of the correlations appropriate as a secondary basis (and complement to laboratory testing) for evaluating the shear strength of the embankment material. The primary basis for selecting fill shear strengths is the results of the laboratory testing program.

The first correlation (Kulhaway and Mayne [1990], Schmertmann [1975]), was developed using uncorrected SPT blow count (N) values and is given by the equation:

$$\phi' = \tan^{-1}[N/(12.2 + 20.3\,\sigma'_{vo} / p_a]^{0.34} \qquad \text{(Eq. 2-1)}$$

where ϕ' is the effective-stress friction angle of the soil (degrees); N is the measured SPT blow count value corrected by a factor of 1.5 [Kovacs, 1994] to account for the use of a nonstandard automatic hammer (dimensionless); σ'_{vo} is the vertical effective stress in the surrounding ground (psf or kPa); and p_a is atmospheric pressure (psf or kPa). This correlation was not applied to N values from depths less than 6.6 ft (2 m), based on guidance from the stated references. Note that σ'_{vo} and P_a must have the same units.

The second correlation [Hatanaka and Uchida, 1996], was developed using corrected SPT N values for clean to silty sands and is represented by the equation:

$$\phi' = \sqrt{15.4\,(N_1)_{60}\cdot} + 20^\circ \qquad \text{(Eq. 2-2)}$$

where $(N_1)_{60}$ is the SPT blow count value (dimensionless) corrected to a vertical effective stress of 1 ton/ft^2 (96 kPa) and to a standard recommended in Kavazanjian et. al. [1997]. This correlation was not applied to $(N_1)_{60}$ values larger than 39 based on guidance from the stated reference.

A summary of the results of these correlations are presented in Table 2-2.

2.3.3 Shear and Compression Wave Velocities

The results of seismic downhole testing is provided in Figures 2-4 for borings B-1A and B-5. Shear wave velocities range from 457 to 1501 ft/s (139 to 458 m/s) and generally increase with depth. This range of shear wave velocity is consistent with published values for compacted fills. Compression wave velocities are consistent with published values for unsaturated to nearly saturated soils.

3. LABORATORY TESTING

3.1 Overview

Laboratory testing was conducted on soil samples obtained by GeoSyntec during the October 1998 field investigation activities described in Section 2 of this report. The testing was performed at GeoSyntec's Geomechanics and Environmental Laboratory (GEL) located in Alpharetta, Georgia. Both triaxial compression tests (TX tests) and index property tests were performed as outlined below.

- TX tests were performed on samples of embankment dam material in order to evaluate material shear strength parameters for use in stability analyses. In all but one case, these tests were performed on intact (i.e., relatively undisturbed) samples obtained from thin-walled sample tubes in order to closely simulate in-situ conditions.

- Index property tests included Atterberg Limits to evaluate sample plasticity and grain size tests to evaluate sample gravel, sand, silt, and clay content. Results of these index tests allow each sample to be classified using the Universal Soil Classification System (USCS) (ASTM D 2487).

The total numbers of tests performed for the project are as follows: (i) Atterberg Limits tests - six; (ii) grain size distribution tests - six; and (iii) TX tests - 16. The results of the laboratory testing program are presented below.

3.2 Triaxial Compression Tests

3.2.1 Test Procedure

Isotropically consolidated-undrained triaxial compression tests with porewater pressure measurements (ICU TX tests) were performed, in accordance with ASTM D 4767: "*Standard Test Method for Consolidated-Undrained Triaxial Compression Test on Cohesive Soils.*"

The ICU TX test procedure involved two stages, a consolidation stage followed by a shearing stage. For the consolidation stage, each test specimen was placed in the test cell and subjected to an isotropic effective stress slightly greater (i.e., 10 percent greater) than the effective vertical stress estimated to be acting on the specimen in the dam at the time of sampling. The specimen was then allowed to consolidate (i.e., come to an equilibrium condition) under the applied stress in order to closely simulate in-situ conditions. The shearing phase then commenced. In the shearing phase, the vertical stress on the specimen was gradually increased until failure occurred (i.e., the shear stresses within the specimen exceeded the specimen shear strength). During the shearing phase, the specimen was not allowed to drain, and the water pressure that developed within the specimen, referred to as porewater pressure, was measured. The use of the ICU TX procedure allows determination of strength parameters appropriate for steady-state stability analysis (i.e., effective stress strength parameters). The test also allows determination of strength parameters appropriate for seismic stability analysis (i.e., total stress strength parameters) if the soil is not prone to cyclic porewater pressure buildup during an earthquake. The compacted embankment fill for Lake Petit Dam meets this criterion.

3.2.2 Samples Tested

The 16 soil specimens tested were obtained from Shelby tube and Pitcher barrel samples recovered from various elevations at the October 1998 boring locations. The samples were selected in order to produce test results from a full range of depths and locations in both the shell and core material of the dam, as indicated in Figure 2-2. In the laboratory, each selected sample was extruded from the sampling tube and examined to assess the degree of disturbance caused by the sampling process. In three or four cases, the sampling tube did not contain a sufficient length (i.e., approximately 6 in. (150 mm)) of undisturbed material from which to obtain a test specimen. In these cases, a test specimen was obtained from a tube from an adjacent sampling interval. gravel

The material for one of the 16 test specimens was obtained by reusing the material from a previously tested specimen. The material was completely broken down from an intact condition and then recompacted to a density consistent with in-situ conditions to form a test specimen. The recompacted specimen was consolidated to the same effective stress used for the previous test. The purpose of this test was compare the response of an intact specimen and a recompacted specimen as a means to identify potential sample disturbance effects in the intact specimens.

3.2.3 Test Results

Test conditions and test results for the 16 ICU TX tests are summarized in Table 3-1. A detailed report presenting the complete results of the laboratory testing program is presented in Appendix C. The 16 test specimens exhibited consistent responses, generating similarly shaped compression and porewater pressure response curves.

The 16 test specimens typically generated negative, to slightly positive, porewater pressures at the end of the tests. This type of porewater pressure response is common for compacted fill materials and is largely a result of the high stresses imposed by compaction operations during dam construction. It is noted that the response of the recompacted test specimen and corresponding intact specimen were similar, indicating that the tendency for negative porewater pressure generation is characteristic of the dam fill material and is not an artifact of the sampling procedures used in the field investigation.

The test results have been plotted to assess both effective-stress shear strength parameters and total-stress shear strength parameters. Effective-stress parameters are assessed by the plots shown in Figure 3-1. These plots indicate results for peak strength conditions, which are mobilized prior to overall failure of the test specimen, and for ultimate strength conditions, which are mobilized at the end of the test. Total stress parameters are assessed by the plot shown in Figure 3-2. The lines on Figures 3-1 and 3-2 represent the shear strength parameters used in the stability analyses. The selection of parameters is discussed in Section 5 of this report.

3.3 Index Property Tests

3.3.1 Test Procedures

Index property tests were performed in accordance with the three ASTM test methods given below. Each of the six index property tests conducted for this project involved all of these test methods.

- Atterberg Limits: ASTM D 4318, "*Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.*"

- Grain Size Analysis: ASTM D 422, "*Standard Test Method for Particle-Size Analysis of Soils.*"

- USCS Soil Classification: ASTM D *2487, "Standard Test Method for Classification of Soils for Engineering Purposes (Unified Soil Classification System)."*

3.3.2 Samples Tested

The six index property tests were conducted on material from previously tested ICU TX specimens. The samples were selected in order to produce test results for both the shell and core material of the dam.

3.3.3 Test Results

Test results for the six index property tests are summarized in Table 3-1. Detailed test results are provided in the laboratory testing report presented in Appendix C. The three test specimens obtained from the dam core exhibited slightly different soil index properties than the specimens obtained from the dam shell. Specifically, the dam shell specimens all had a USCS classification of SM, silty sand, and had a clay content of less than 6 percent. In contrast, the dam core specimens all had a USCS classification of ML, low-plasticity silt, and had clay contents between 10 and 20 percent. The results confirm a difference in physical properties for the shell and core materials, and are consistent with the construction specifications for each zone of the dam.

4. PIEZOMETER MEASUREMENTS

4.1 Overview

GeoSyntec conducted a detailed evaluation of available piezometric data for the purpose of assembling an analytical model to simulate steady-state seepage through the dam. This section of the report addresses both historical and contemporary data obtained from the three series of piezometers installed in the dam. Based on a detailed evaluation of these data, two piezometric data sets have been developed as being representative of seasonal low and seasonal high piezometric pressure grids. The use of these grids in terms of seepage analysis is addressed in detail in Section 5 and 6 of this report.

4.2 Law Piezometers (Installed 1974)

Law Engineering Testing Company (Law) performed a subsurface investigation and engineering evaluation of seepage in Lake Petit Dam in 1974. Results of this investigation and evaluation are presented in a report titled "Report of Engineering Evaluation, Lake Petit Dam, Dawson County, Georgia" dated 18 March 1974.

The Law investigation program involved the installation of five piezometers (referenced to as P-1 through P-5 in the Law report) located on each bench in an approximate North-South line, just west of the centerline of the dam. Three of these piezometers were located by Piedmont during their site reconnaissance. These locations are shown on Figure 2-1 and are designated as L3, L4, and L5. Based on discussions with Mr. Craig Robinson (Piedmont), the serviceability of these piezometers was questionable at the time Piedmont measured water elevations, due to inadequate surface covers. The wells were bailed down by Piedmont, and re-measured one week later. Data are presented in Table 4-1. GeoSyntec measured water elevations in these piezometers during October 1998. These data are also presented in Table 4-1. Due to the age of these piezometers and condition of the surface covers, Piedmont did not utilize these data in their analyses. Similarly, GeoSyntec did not specifically use these data, but checked the readings for general consistency with other data sets subsequently described. Law not used.

It is interesting to note that the Law report represents the first reporting of a high phreatic surface in the dam. Based on the Law observations, it appears that a high phreatic surface was established in the downstream shell of he dam since at least March 1974, which is not very long after the dam was constructed. The Law report contained recommendations to install supplemental surficial drains, abutment drains, and gutter drains on the existing benches The supplemental drainage measures were recommended as a means of controlling downstream shell seepage by collecting and channeling the flow in a manner that would prevent erosion and piping. Based on GeoSyntec's field reconnaissance, it appears that at least some of Law's recommendations were implemented. The Law report also recommended the installation of paved concrete gutters on the upper three berms of the dam, to eliminate ponding and subsequent infiltration of surface water in these areas. Based on GeoSyntec's field reconnaissance, this recommendation was apparently not implemented.

Look @ Law report

To summarize, Law recognized the high phreatic surface as a feature that could result in uncontrolled seepage and possibly piping. Control measures were implemented to address seepage both in the face of the dam and in the abutments. These measures appear to have been effective in the case of the abutments, and moderately effective in the case of surface seepage from the downstream face of the dam. Mr. Neil Davies and Dr. Gary Schmertmann (GeoSyntec) recently discussed the project with Mr. David Pauls (a co-author of the Law report). Mr. Pauls confirmed that the emphasis of Law's work was to investigate and control seepage. At the time of Law's evaluation, the effect of the high phreatic surface on the embankment stability of the dam was considered but not formerly evaluated. Mr. Pauls also stated that the recommended rehabilitation measures were reviewed by the late Professor George Sowers of the Georgia Institute of Technology, a noted authority on embankment dams.

Talk to him

4.3 Vibrating Wire Piezometers (Installed 1997-1998)

Two nested sets of vibrating wire piezometers were installed by Piedmont at locations P-2 and P-4 in October 1997. These piezometers were installed as part of Piedmont's phased investigation of the dam. Each location included three vibrating wire piezometers placed at isolated depth intervals within each of the borings. The intent was to evaluate the possible variation of piezometric levels with depth through the embankment.

Based on measured piezometric pressures from these two sets of piezometers, Piedmont developed an analytical model of the dam that incorporated three generally horizontal soil layers, each having the same strength properties but different piezometric levels as indicated by the piezometric data for that level. According to Piedmont, *"the piezometric levels were originating at the normal pool elevation at the upstream shoreline, diverging through the piezometric data in the two borings that were performed, and then essentially converging to a point where seepage has been observed on the second slope section up from the base of the dam."*

In constructing the piezometric profiles, Piedmont used the highest recorded water elevations, and used this model to estimate the minimum factors of safety for downstream embankment slope stability failure under both static steady-state seepage conditions and seismic conditions. As previously reported, the resulting calculated minimum factors of safety were 1.28 and 0.75, respectively. Subsequent to reporting these values, two additional locations were drilled and three sets of piezometers were installed at each location (locations P-6 and P-7). These piezometers were installed in May 1998 for the purpose of obtaining piezometric data in the lower portion of the dam. Subsequent analyses by Piedmont utilized the piezometric data obtained from these additional piezometers to refine their analytical model. This refined model was then used to evaluate various rehabilitation measures. where is this?

Piezometric pressures have been recorded for each vibrating wire piezometer (12 total) on a regular basis since their installation. These data are presented in Table 4-2. who

GeoSyntec performed an initial evaluation of the vibrating wire piezometer data at an early stage of its re-evaluation. GeoSyntec interpreted the variation in piezometric elevation with depth as a series of pressure points indicative of steady-state seepage through a porous homogeneous, but anisotropic, medium. GeoSyntec was able to construct a conceptually reasonable flow net using the vibrating wire piezometric data set, that satisfied Darcy's law and the equations of continuity. Since the overall shape of this flow net corresponded to published examples of flow nets for similar structures [e.g., Cedergren, 1989], GeoSyntec concluded that this interpretation of piezometric pressure variations is a better representation of the steady-state seepage flow regime within the dam than the earlier interpretation by Piedmont. To confirm this interpretation of the vibrating wire piezometer data, GeoSyntec focused a significant part of its 1998 investigation program on the collection of additional piezometric data. The preliminary conceptual flow net described above was used as a planning aid to select locations and monitoring intervals for the additional piezometers. what. ok.

See water surface

4.4 Standpipe Piezometers (Installed 1998)

Seven standpipe piezometers were installed as part of GeoSyntec's 1998 investigation of the dam, as described in Section 2 and indicated on Figure 2-3. These piezometers were installed at strategic locations within both the downstream shell and the core in order to better define the interpreted flow regime within the dam. The rationale for placement of piezometers is described in Section 2. These piezometers were installed in October 1998, and recorded piezometric elevations are presented in Table 4-3.

4.5 Interpretation of Piezometric Data

4.5.1 Combined Data Set

GeoSyntec has used the combined data set from the vibrating wire piezometers and the standpipe piezometers to assemble the physical conditions model for the dam described in Section 5 of this report. As previously stated, data from the Law piezometers was not specifically used since the integrity of the surface seals is questionable and their response may be influenced by surface water infiltration.

Based on a detailed review of the assembled data set, two key findings were apparent: (i) contemporary data from both the vibrating wire piezometers and the standpipe piezometers supports the interpretation of a flow regime through a porous medium; and (ii) significant seasonal variations were observed in the vibrating wire piezometer data set. The use of the data in developing the physical conditions model is described in detail in Section 5 and 6.

4.5.2 Seasonal Variations

A review of the vibrating wire piezometer data indicates considerable seasonal variation following initial stabilization of the instruments. Table 4-4 presents a summary of the measured piezometric variations, expressed in feet of water, following initial stabilization. As can be seen, most piezometers exhibit seasonal variations in the range 1 to 3 ft (0.3 to 0.9 m). However, P-4A, P-4B, and P-4C exhibit higher variations with P-4A having a maximum variation of 6.9 ft (2.1 m). Figure 4-1 presents a plot of piezometric head variation versus time superimposed over monthly precipitation as reported for Jasper, Georgia for the time period of October 1997 to October 1998

not local

(plotted on the same time scale). Figure 4-1 clearly supports the following observations:

- each piezometer indicates temporal variations exhibiting similar trends (i.e., peaks typically occur at similar times);
- peak piezometric readings generally occurred in the period March/April 1998, which corresponds to a period of relatively high and generally increasing total monthly rainfall (i.e., January through April); and
- the general trend in the piezometric readings is consistent with the general trend in rainfall quantity (i.e., high readings correspond to high rainfall and low readings correspond to low readings).

In summary, there appears to be a correlation between seasonal rainfall and piezometric readings. Given that Lake Petit is maintained at a close to constant elevation, this correlation to rainfall is attributed by GeoSyntec to percolation of surface water into the downstream shell of the dam. Discrepancies between observed rainfall peaks and piezometric readings are likely attributable to the following:

- rainfall measurements used in this comparison were taken approximately 20 miles (32 km) from location of the dam; Note 2
- the type of rainfall event (e.g., light prolonged rain versus heavy intense rainfall) will likely influence infiltration; and
- seasonal temperature variations will influence evapotranspiration rates which are not accounted for in the foregoing analysis.

Further evaluation of rainfall data from Jasper indicates that April 1998 was a period of abnormally high rainfall. Figure 4-2 presents a histogram of monthly rainfall for Jasper, Georgia and indicates the comparison of the 30-year monthly average rainfall to monthly average rainfall for the period October 1977 through October 1998. This figure clearly indicates that total rainfall in April 1998 (12.23 in) was considerably higher than the 30-year average of 4.99 in for that month. Therefore, the use of piezometric data from April 1998 in developing a site physical conditions model is considered both conservative and appropriate for the evaluation of rehabilitation measures.

5. SITE PHYSICAL CONDITIONS MODEL

5.1 Geometry

A representative cross section of the Lake Petit dam was developed to be used for seepage and slope stability analyses. The cross section is shown in Figure 5-1 and is located approximately along the centerline of the dam (see Figure 2-1). This cross section corresponds to the maximum height of the dam and is located in an area where surface seepage has been observed.

The surface elevations of the downstream slope of the dam were developed based on the current (1998) topographic map. These elevations were verified based on a GPS survey conducted by GeoSyntec on 15 October 1998. Based on the design drawings, the surface of the upstream slope of the dam was assumed to be inclined at 3.5H:1V. when

The materials comprising this cross section include the dam shell and core fills and the underlying saprolite and bedrock. The subsurface boundaries between these materials were established using information from the following sources: (i) boring logs from recent field investigations by GeoSyntec and Piedmont as well as from field investigations performed prior to dam construction [Coleman, 1971a, 1971b]; where (ii) topographic map of the area prepared prior to construction of the dam (preconstruction topographic map); and (iii) design drawings for the dam [Baldwin and Cranston, 1971]. Use of the available information in establishing the subsurface boundaries is described in the following paragraphs. supply

Information from the current subsurface investigation and laboratory testing program indicates that different materials were used to construct the dam shell and dam core. Field descriptions of samples and laboratory test results indicate that the shell material is predominately a nonplastic silty sand (USCS classification of SM) containing weathered rock fragments. The core material is predominately a low plasticity silt (USCS classification of ML) with fewer weathered rock fragments. Correlations relating SPT blowcount values obtained during the field investigation to effective-stress friction angles also indicate that shell and core materials differ (Section 2.3.2) in that the core material generally has a smaller average effective-stress friction angle. The lack of significant stratification observed during the subsurface investigation (Section 2.3.1) indicates that both the shell and core materials are relatively consistent and homogenous within their identified zones of placement.

The geometry of the core was developed based on information obtained from borings and the geometry of the core as shown in the design drawings. The top of the core was assumed to be at elevation 1637 ft MSL and to be centered under the crest of the dam. Based on the depth to the core/shell interface indicated in boring logs G-1B and G-5, the slope of the downstream face of the core was assumed to be 1H:1.5V. The slope of the upstream face of the core was assumed to be equal to that of the downstream face. The design drawings indicate that a cutoff trench was to be constructed to extend the core down through the saprolite layer to bedrock. The cutoff trench was not included in the analysis cross section but its effect on the seepage analyses was considered as discussed subsequently in Sections 5.2.4 and 6.4. Borings should show this

The thickness of the saprolite layer at the site varies from 5 to 10 ft (1.5 to 3.0 m) based on current and previous boring records. The location of the top of the saprolite was developed using boring information and the preconstruction topographic map. In the area below the ball field and at the upstream toe of the dam, the top of the saprolite was assumed to correspond to the ground surface existing prior to construction of the dam. From the downstream toe of the dam inwards towards the core, the top of the saprolite was identified based on GeoSyntec borings G-1B and G-2 and Piedmont borings P-4 and P-7. At the centerline of the dam, the top of saprolite was assumed to be 5 ft (1.5 m) thick above bedrock. .

The interpreted bedrock surface was defined based on GeoSyntec borings G-1B and G-2, and Piedmont boring P-4, and three of the Coleman [1971a,1971b] borings. In the area below the ballfield, the bedrock surface was assumed to be located 5 ft (1.5 m) below the top of the saprolite layer. why?

5.2 Material Properties

5.2.1 Dam Shell

Unit Weight

Laboratory dry unit weight and water content measurements on intact specimens of dam shell material are given in Table 3-1. Based on these measurements, calculated total (i.e., moist) unit weights varied from 115 to 132 lb/ft^3 (18.1 to 20.7 kN/m^3), with an average value of 125 lb/ft^3 (19.6 kN/m^3). Based on these results, a moist unit weight of 125 lb/ft^3 (19.6 kN/m^3) was used for the slope stability analyses.

Shear Strength

Shear strength parameters for the dam shell material were evaluated using the results of the laboratory ICU TX tests performed on intact specimens. The laboratory test results are presented in Section 3.2. Based on the sampling strategy used during the field investigation, as discussed in Section 2.1, and the range of locations and elevations from which the samples were obtained, as discussed in Section 3.2.2 and shown on Figure 2-2, the tested specimens are believed to be representative of the dam shell material. The testing results therefore form an appropriate basis for evaluation of shear strength parameters for use in slope stability analyses.

Effective-stress shear strength parameters, also referred to as drained shear strength parameters, are evaluated from the plots of q versus p' given on Figure 3-1. Lines representing different values of effective stress friction angle, ϕ', with cohesion, c', equal to zero, are shown on Figure 3-1. Based on the plot for the peak strength condition, all the measured ϕ' values, excluding Test B (see Table 3-1), fall within a relatively narrow range between 38° and 43°. A conservative characterization of the measured values for the peak strength condition is given by ϕ'=38° and c'=0. It is noted that the empirical correlations between ϕ' and SPT blow counts, as discussed in Section 2.3.2 and summarized in Table 2-2, indicate a minimum friction angle of 37° and an average value of 40°. These values are consistent with the values from the triaxial tests and support the observation that the fill material used in the shell of the dam is relatively homogeneous.

Effective-stress shear strength parameters for the ultimate strength condition are addressed in the plot in the lower portion of Figure 3-1. In this plot all the measured ϕ' values, excluding Test B, fall within a narrow range between 34° and 37°. A conservative characterization of the measured values for the ultimate strength condition is given by ϕ'=34° and c'=0.

The general standard of practice for selecting effective-stress shear strength parameters for compacted soil materials for use in static, steady-state seepage slope stability analyses is to use parameters for the peak strength condition. GeoSyntec, however, conservatively elected to use parameters for the ultimate strength condition. The following effective-stress shear strength parameters were thus selected for use in static, steady-state seepage slope stability analyses, ϕ'=34° and c'=0. ? is this a fact, Z

Total-stress shear strength parameters, also referred to as undrained shear strength parameters, are evaluated from the plots given on Figure 3-2. The plots are constructed with the effective consolidation stress used in each ICU TX test on the horizontal axis and the measured undrained shear strength, q, on the vertical axis. The plot in the lower portion of Figure 3-2 is identical to that in the upper portion except that undrained strength values have been reduced by 25 percent, as discussed in the following paragraph.

As indicated in section 3, the ICU TX tests were conducted with an isotropic stress condition during the consolidation stage of the test. Soils in the dam may not have consolidated under isotropic stress conditions and it is possible that undrained strengths measured under consolidation stress conditions more consistent with those in the field would be smaller than measured in the ICU TX tests. This possibility was accounted for by reducing measured undrained strengths by 25 percent, a conservative reduction based on data provided by Mayne [1985]. It is noted that it was not necessary to consider this possibility when assessing effective-stress (drained) shear strength parameters because effective-stress parameters are not significantly affected by differing consolidation stress conditions.

The plot in the lower portion of Figure 3-2 was used to establish conservative undrained shear strengths within the dam shell. A line with a slope of 0.58 and a vertical axis intercept of 7 psi (48 kPa), as shown in the Figure, provides a lower bound characterization of undrained shear strength. These parameters, slope of 0.58 and vertical axis intercept of 7 psi (48 kPa), were used to conservatively characterize the undrained shear strength of the dam shell material for the seismic slope stability analyses. These parameters were applied in the seismic slope stability analysis by equating vertical effective stress at each point in the cross section with effective consolidation stress.

It is noted that use of these parameters to characterize undrained shear strength is potentially unconservative for points within the dam shell where effective consolidation stresses are less than the smallest value used in the ICU TX tests (i.e., 10 psi (69 kPa)). Therefore, these parameters were not used in seismic slope stability analyses performed for shallow potential slip surfaces. One approach for characterization of undrained shear strengths for such analyses is to use drained shear strength parameters. Although this approach is considered very conservative for the dam shell material because of its observed tendency to generate negative pore pressures during undrained shear (Section 3.2.3), it was selected for use. Therefore, the effective-stress shear strength parameters previously described (i.e., ϕ'=34° and c'=0) were conservatively used for seismic slope stability analyses performed for shallow potential slip surfaces.

Hydraulic Conductivity

The dam shell material classifies as a non-plastic to low plasticity silty sand (USCS classification of SM). The percent of material passing the U.S. Standard No. 200 sieve (i.e., percent fines) ranges from 30 to 44 percent and the maximum plasticity index (PI) is 3. A representative range of hydraulic conductivities for a silty sand is 3.3×10^{-7} ft/s (1×10^{-5} cm/s) to 3.3×10^{-5} ft/s (1×10^{-3} cm/s) [Terzaghi and Peck, 1967]. An intermediate hydraulic conductivity value of 3.3×10^{-6} ft/s (1×10^{-4} cm/s) was used for the dam shell for each seepage analysis.

Since a compacted embankment dam is constructed in horizontal lifts, it is likely that the hydraulic conductivity in the horizontal direction, K_h, is greater than the hydraulic conductivity in the vertical direction, K_v. The hydraulic conductivity ratio, K_h/K_V, can be estimated by comparing measured porewater pressure distributions within the dam with porewater pressure distributions developed for various hydraulic conductivity ratios. This approach is used for the seepage analyses described in Section 6.

According to Sherard et al. [1963], for dams constructed with uniform, fine-grained soils that are placed using appropriate moisture-density control, it is likely that K_h/K_v will not exceed four. However for coarser soils or where borrow source materials are variable, K_h/K_v may be significantly higher. A range of one to ten for K_h/K_v was used for the dam shell soils in the seepage analyses. It is noted that the selected hydraulic conductivity value of 3.3×10^{-6} ft/s (1×10^{-4} cm/s), as mentioned above, was used as the vertical hydraulic conductivity of the dam shell for each seepage analysis.

pretty high — remolded sample?

5.2.2 Dam Core

Unit Weight

Laboratory dry unit weight and water content measurements on intact specimens of dam core material are given in Table 3-1. Based on these measurements, calculated total (i.e., moist) unit weights varied from 128 to 134 lb/ft^3 (20.1 to 21.2 kN/m^3), with an average value of 131 lb/ft^3 (20.6 kN/m^3). Based on these results, a moist unit weight of 130 lb/ft^3 (20.4 kN/m^3) was used for the slope stability analyses.

Shear Strength

Shear strength parameters for the dam core material were evaluated in a similar manner as for the dam shell material. In considering effective-stress shear strength parameters for peak strength conditions, the measured values fall within a narrow range between ϕ' values of 40° to 41°. A conservative characterization of the measured values for the peak strength condition is given by ϕ'=40° and c'=0. It is noted that the empirical correlations between ϕ' and SPT blow counts, as discussed in Section 2.3.2 and summarized in Table 2-2, indicate a minimum friction angle of 30° and an average value of 34°. These values are lower than the values from the ICU TX tests.

In considering effective-stress shear strength parameters for ultimate strength conditions, the measured values fall within a narrow range between ϕ' values of 34° to 36°. A conservative characterization of the measured values for the ultimate strength condition is given by ϕ'=34° and c'=0. high silt.

The general standard of practice for selecting effective-stress shear strength parameters for compacted soil materials for use in static, steady-state seepage slope stability analyses is to use parameters for the peak strength condition. GeoSyntec, however, conservatively elected to use parameters for the ultimate strength condition. In addition, because the values of effective stress friction angle obtained for the dam core material from the SPT blow count correlations were lower than those for the dam shell material, GeoSyntec conservatively elected to use a lower friction angle for the dam core material than that obtained from the ICU TX test results. Specifically, effective-stress shear strength parameters of ϕ'=32° and c'=0 were selected for the dam core material for use in the static, steady-state seepage slope stability analyses.

Undrained strength measurements for the dam core materials were consistent with those for the dam shell material. Therefore, undrained strength parameters for the core material were selected to be equal to those selected for the dam shell material (Section 5.2.1).

Hydraulic Conductivity

The dam core material classifies as a low plasticity sandy silt (USCS classification of ML). The percent fines for this material ranges from 52 to 58 percent and the PI ranges from 9 to 15. A representative range of hydraulic conductivities for a sandy silt is 3.3 x 10^{-8} ft/s (1 x 10^{-6} cm/s) to 3.3 x 10^{-6} ft/s (1x10^{-4} cm/s) [Terzaghi and Peck, 1967]. On average, the ML material used for the dam core has approximately 20 percent more fines than the SM material used for the dam shell. Because of this difference in fines content, it was assumed that the hydraulic conductivity of the dam core material is no greater than that of the dam shell material. For the seepage analyses, the dam core was modeled with a vertical hydraulic conductivity that ranged from 3.3 x 10^{-6} ft/s (1x10^{-4} cm/s), the same hydraulic conductivity used for the dam shell, to 3.3 x 10^{-8} ft/s (1x10^{-6} cm/s). Similarly to the dam shell, the ratio of horizontal to vertical hydraulic conductivity (K_h/K_v) for the dam core was varied from one to ten.

5.2.3 Saprolite

Unit Weight and Shear Strength

A moist unit weight of 125 lb/ft^3 (19.6 kN/m^3) was used for the slope stability analyses. This value was selected based on experience with similar materials at other sites. Conservatively, shear strength parameters of ϕ'=35° and c'=0 were selected for use in the slope stability analyses for both effective-stress and total-stress analyses. These parameters are considered conservative based on the high SPT blow counts measured in the material and on experience with similar materials at other sites.

reference ~~[illegible]~~ info

Hydraulic Conductivity

The saprolite at the Lake Petit Dam consists primarily of a sandy silt that has weathered from granitic bedrock. It is likely that the saprolite is more permeable than the bedrock but less permeable than the dam shell material. The saprolite on the downstream side of the core was modeled with a vertical and horizontal hydraulic conductivity of 1.6 x 10^{-6} ft/s (5x10^{-5} cm/s), a value between that of the bedrock and the dam shell.

For the seepage analyses, a distinction is made between the saprolite on the downstream side of the core and the saprolite on the upstream side of the core. The design drawings for the dam indicate that a cutoff trench was to be constructed to extend the core down through the saprolite layer to bedrock. The effect of the cutoff trench would be to minimize seepage through the saprolite layer. Although the cutoff trench was not explicitly modeled in the analysis cross section, its effect was represented by assigning a relatively low hydraulic conductivity for the saprolite layer on the upstream side of the core. This modeling assumption is discussed further in Section 6.4.

5.2.4 Bedrock

Unit Weight and Shear Strength

A bedrock moist unit weight of 150 lb/ft^3 (23.6 kN/m^3) was used in the slope stability analyses. This value was selected based on experience with similar materials at other sites. Shear strength parameters were assumed, based on experience, to be large enough such that any potential slip surfaces passing through this material will have a more than adequate slope stability factor of safety. This assumption is based on judgement that the bedrock material identified in the borings is competent and is much stronger than the overlying soil materials. Shear strength parameters of ϕ'=45° and c'=10,000 lb/ft^2 (479 kPa) were selected for use in the slope stability analyses for both effective-stress and total-stress analyses.

Hydraulic Conductivity

Bedrock was assumed to be relatively impermeable compared to the dam shell. This modeling assumption is supported by the observation that no boils or other indications of upward seepage were observed in the tailwater creek below the dam. If the bedrock was more permeable than the dam shell, then such indications might be present. This modeling assumption also reflects the belief that seepage through bedrock does not have a significant influence on porewater pressures within the cross section along the dam centerline. This belief is supported by the observation, discussed subsequently in Section 5.3, that piezometric levels in the materials beneath the downstream side of the dam do not indicate that excessive uplift pressures are present in the downstream dam foundation. For the seepage analyses, a relatively small vertical and horizontal hydraulic conductivity value of $3.3x10^{-9}$ ft/s ($1x10^{-7}$ cm/s) was used for bedrock.

5.3 Pore Pressures

Measured piezometric levels from the vibrating wire piezometers (Section 4.3) and the standpipe piezometers (Section 4.4) were used to develop combined data sets of piezometric levels for use in the seepage analyses. Combined data sets were developed for the following two scenarios: (i) 23 October 1998 measured piezometric levels; and (ii) estimated maximum (conservative) piezometric level reflecting high seasonal piezometric levels. Each of these combined data sets is discussed in the following paragraphs.

The 23 October 1998 scenario was developed using the measurements taken in the vibrating wire piezometers and standpipe piezometers on the indicated date. This was the first date after installation and development of the standpipe piezometers when stable readings were obtained in all the devices. Measurements of the vibrating wire piezometers were also obtained for the date. The measured values are indicated in Figure 2-3. The following comments describe significant aspects of the 23 October 1998 scenario given in Figure 2-3:

- the 23 October 1998 scenario provides an internally consistent data set, with no clearly irregular measurements, that reflects steady-state seepage through a zoned earth dam with a core and homogeneous downstream shell;
- the phreatic line appears to be more than 20 ft (6 m) below the ground surface in the upper two-thirds of the dam face but may be within 10 ft (3 m) in the lower third; ck?
- piezometric levels in the saprolite and bedrock underlying the downstream side of the dam appear to be consistent with levels within the fill, suggesting that excessive uplift pressures are not present in the downstream dam foundation;
- piezometric levels in the G-3 and P6 piezometers decrease with depth, possibly indicating that the transverse dam foundation drain between the G-2 and G-3 benches is at least partially effective; and what drain.
- piezometric levels in the standpipe piezometer in boring G3 and the lowest vibrating wire piezometer in boring P6, which are installed at approximately the same elevation but offset by 115 ft (35 m), agree closely; this agreement suggests that piezometric levels within the highest portions of the dam may be consistent in the transverse direction. ok

The 23 October 1998 scenario represents a combined data set for actual measured conditions and is therefore suitable for use in calibration of the seepage analyses described in Section 6.

The estimated maximum (conservative) piezometric level scenario (EML scenario) was developed to represent seasonal high water levels in the dam for use in slope stability calculations. The EML scenario was established using seasonal high measurements from the vibrating wire piezometers (recorded around April 1998). As discussed in Section 4.5.2, these measurements are considered conservative and appropriate for this purpose. Measurements from the standpipe piezometers were also used to establish the EML scenario. However, because the standpipe piezometers (installed in October 1998) have not been in place long enough to record seasonal high water levels, the measured levels were extrapolated upward for the EML scenario. The values were extrapolated in a manner consistent with the seasonal fluctuations measured in the vibrating wire piezometers. The piezometric levels for the EML scenario are indicated in Figure 2-3. ?

written info – detailing software

6. SEEPAGE ANALYSIS

6.1 Overview

The steady-state seepage regime and porewater pressure distribution for the Lake Petit Dam were evaluated using a finite element analysis seepage computer program called SEEP/W [GEO-SLOPE International, 1998]. This computer program was used to model water movement (seepage and porewater pressure distribution) through the soil and rock materials within the dam. SEEP/W is well-suited for modeling unconfined flow such as occurs through an embankment dam. Issues related to the analysis of Lake Petit Dam that were evaluated using SEEP/W included: (i) variations in horizontal and vertical permeability of the compacted materials of the dam core and dam shell; (ii) differing permeabilities of the core, shell, and foundation rock layers; (iii) effectiveness of the internal drain system; and (iv) position of the phreatic (zero porewater pressure) line.

Parametric analyses using SEEP/W were performed to evaluate a reasonable porewater pressure distribution to be used in the steady-state seepage static and seismic slope stability analyses. Measured porewater pressure values (Section 5) were compared to computed values from the SEEP/W analyses to identify a "best-fit" porewater pressure distribution. The slope stability analysis program used for the analyses described in Section 7 uses a grid of porewater pressure values and interpolates between the specified grid values to evaluate porewater pressures at any point along the failure surface being modeled in the slope stability analysis. Porewater pressures have been measured using piezometers at 18 locations within the dam. Direct use of a porewater pressure grid based on 18 values would not provide an adequate interpolation of porewater pressures for areas of the dam remote from the grid points. With the SEEP/W program, porewater pressure values can be computed at a sufficiently large number of points to enable a porewater pressure grid for stability analyses to be developed, that is representative of actual conditions within the dam and its foundation.

6.2 Finite Element Mesh

As previously described in Section 5, one cross section was developed for seepage and stability analyses of the dam. For the SEEP/W analyses, a finite element mesh was developed to represent the overall geometry and seepage boundary conditions for this cross section. The mesh used for the analyses consists of 840 nodal points and 793 elements and is shown in Figure 6-1. The materials modeled include the dam core, dam shell, saprolite layer, bedrock, and soils below the ball field. For the analyses performed, an approximately 10-ft (3-m) thick bedrock layer was modeled using the hydraulic conductivity for bedrock given in Section 5.2. The bottom of this bedrock layer was modeled as a no-flow boundary. Prescribed total head values corresponding to a lake level of 1635 ft MSL were modeled along the upstream boundary of the dam. A foundation drain located approximately 150 ft (45 m) in from the toe of the dam at elevation 1520 ft MSL was modeled as a single nodal point.

The location of the six lines of piezometers (i.e., P2, G1, P4, G2, P6, and P7) are also shown on Figure 6-1. Three piezometers were installed at different depths at the location of each piezometer line. Measured porewater pressures at the 18 piezometers were compared to computed values from the SEEP/W analyses to assess the reasonableness of the computed porewater pressure distribution. The locations of the 18 piezometers are shown on Figure 6-2. A nodal point was placed as close as possible to each of these 18 locations.

6.3 Analysis Procedures

To develop the best-fit porewater pressure distribution to be used for slope stability analyses, over 30 SEEP/W finite element analyses were performed. Prior to performing these analyses, parametric ranges for each input variable were established, as subsequently discussed. The range of parameter values used for the analyses were within the ranges given in the site physical conditions model described in Section 5. Parameters that were varied for the analyses include: (i) vertical hydraulic conductivity of the core material; (ii) ratio of horizontal to vertical hydraulic conductivity of the core and shell materials; and (iii) effectiveness of the foundation drain. For each analysis, a particular parameter was varied and the computed porewater pressure head values were compared to measured porewater pressure head values at the location of each of the 18 piezometers. This process was repeated until the best-fit porewater pressure distribution was obtained. As a result of these analyses, two porewater pressure distributions were developed based on: (i) measured porewater pressure readings on 23 October 1998; and (ii) estimated maximum porewater pressure levels (EMLs) from seasonal high water levels. The EMLs were established as described in Section 5.3.

The following procedures were used to develop best-fit porewater pressure distributions for the 23 October 1998 scenario and the EML scenario.

- A set of material parameters was selected for the various materials in the mesh.

- A SEEP/W analysis was performed using the finite element mesh and boundary conditions previously described.

- At each node corresponding to an actual piezometer location, the computed porewater pressure head was computed and compared to measured values. For each analysis, the difference between the measured and the computed porewater pressure head value at the nodal location corresponding to the 18 piezometers was calculated. The average difference was also calculated. A plot showing the computed phreatic surface and equipotential lines was developed for each analysis. This plot was reviewed to evaluate the reasonableness of the solution. Unreasonable solutions were typically indicated by a phreatic surface that was irregular in shape or that was significantly lower than that based on measured values.

- Based on the results of an analysis, each model parameter was adjusted and the previous procedure of performing a SEEP/W analysis and comparing computed to measured porewater pressure head values was repeated. By performing successive iterations, the best-fit, most reasonable (based on professional judgement) porewater pressure distributions were established for the two analysis conditions.

6.4 Model Parameters

The relevant parameters for the SEEP/W analyses of Lake Petit Dam include the vertical and horizontal hydraulic conductivities of each of the soil and rock materials and the effectiveness of the foundation drain. A discussion of the vertical and horizontal hydraulic conductivity of the soil and rock materials is provided in Section 5.2 of this report. The ranges of parameters used is shown in Table 6-1. Additional discussion relevant to assumptions used in developing the finite element model is described below.

- The saprolite on the upstream side of the core of the dam was modeled using a hydraulic conductivity value of 3.3×10^{-9} ft/s (1×10^{-7} cm/s), the same value as that which was used for the bedrock (see Section 5.2).

- Water flow below the ball field is believed to be controlled by a subsurface drainage system. A drain exits into the creek at approximately elevation 1517 ft MSL. A boundary condition was prescribed in the SEEP/W analyses to maintain the water level below the ball field at a constant elevation of 1517 ft MSL. The hydraulic conductivity of the soils below the ball field was selected as 1.6×10^{-6} ft/s (5×10^{-5} cm/s).

- The ratio of the horizontal to vertical hydraulic conductivity for the core and the shell was varied from one to ten for the seepage analyses. Reasonably accurate porewater pressure distributions were obtained using this range.

- It was assumed that the horizontal hydraulic conductivity was equal to the vertical hydraulic conductivity for the soils below the ballfield and the saprolite and bedrock layers. This assumption was found to be insignificant with respect to the predicted seepage response within the dam.

In addition to varying the permeability of selected soil and rock materials, the effectiveness of the foundation drain was also evaluated as part of the parametric study. To assess the impact of the foundation drain on the computed results, three total head values were used for the drain: (i) h=1525 ft; (ii) h=1530 ft; and (iii) h=1535 ft (0 MSL datum). The base of the foundation drain is at approximately elevation 1520 ft MSL. Therefore, to model a fully effective drain, a total head value equal to the elevation of the drain (i.e., 1520 ft MSL) would be used. Total head values used in the analysis are consistent with a partially effective drain. Preliminary analyses assuming that the drain is fully effective resulted in a computed phreatic surface and ~~nodal pressure head values~~ that were significantly less than those measured. Therefore, the focus of this element of the parametric study was to assess the likely degree of effectiveness of the drain under present conditions.

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6.5 Analysis Results

A best-fit porewater pressure distribution was developed based on the 23 October 1998 measured values. The parameters used to develop this porewater pressure distribution were then used to develop the best-fit porewater pressure distribution for the

EML scenario. For the EML analyses, several nodal total head values for the upper piezometers were fixed so that that the computed phreatic surface closely matched that based on the EMLs developed in Section 5. The resulting data set represents the estimated porewater pressure distribution corresponding to seasonal high water levels. This data set is considered to be appropriate for use as a design condition

Table 6-2 shows the parameters used for the best-fit models. In Table 6-3, a comparison of the computed and measured porewater pressure head values for the 23 October 1998 scenario are presented. The input data file for SEEP/W for this analysis case is provided in Appendix D. On average, computed pressure heads are 1.2 ft (0.4 m) higher than the measured values. The computed difference is considered small, yet, conservative since, on average, it represents an overprediction of porewater pressures. Figures 6-3 and 6-4 show the computed seepage regime (i.e., equipotential contour lines and selected flow paths) and porewater pressure distribution, respectively, for the 23 October 1998 scenario.

Table 6-4 presents the comparison of the porewater pressure head values for the EML analyses. For these analyses, the computed pressure heads are on average 2 ft (0.6 m) higher than the measured EMLs. Only three of 18 computed porewater pressure values are more than 3 ft (1 m) less than their corresponding measured (or extrapolated) EMLs. This indicates that computed values represent a reasonable upper bound envelope to the measured EMLs. Figures 6-5 and 6-6 show the computed seepage regime and porewater pressure distribution for the EML scenario. The estimated maximum porewater pressure levels shown in Figure 6-6 represent a conservative assessment of the porewater pressure distribution within the dam. The computed phreatic surface is, on average, higher than that based on the measured EMLs, and is therefore considered conservative and appropriate as a design condition.

Does this say what it says?

7. SLOPE STABILITY EVALUATION

7.1 Overview

The Rules of Department of Natural Resources, Environmental Protection Division, Chapter 391-3-8, *Rules for Dam Safety* [GaEPD, 1998], require that dams be stable under all conditions of construction and/or operation of the impoundment. To demonstrate the stability of an embankment dam, a stability evaluation must be performed, and the results of the evaluation must yield the following minimum factors of safety: (i) steady-state seepage with static loading (steady-state condition), 1.5; and (ii) steady-state seepage with seismic loading (seismic condition), 1.1. To assess the compliance of the Lake Petit Dam with the GaEPD Safe Dams Program rules, the stability of the downstream slope of the dam was evaluated under steady-state and seismic conditions.

The stability analyses focused on downstream embankment stability. The analyses primarily considered relatively deep potential slip surfaces that passed near the dam toe and extended up to the elevation of the G2 bench (Figure 6-6), or higher. Stability analyses for shallow (surficial) potential slip surfaces, which are generally considered a maintenance issue, were performed for some embankment conditions.

? think about this — is this really true?

The stability evaluations were performed using slope stability analysis methods that are widely accepted in geotechnical engineering practice. Primarily, the simplified Bishop [Bishop, 1955] and Spencer [Spencer, 1967] methods of analysis, as implemented in the computer program XSTABL (version 5.888), were used to perform two-dimensional limit equilibrium analyses to calculate the factor of safety of potential slip surfaces for the downstream slope. The XSTABL program [Sharma, 1991] is a widely-used version of the slope stability analysis computer program, STABL, which was originally developed at Purdue University. The result of each stability evaluation is expressed as a minimum calculated factor of safety with an associated potential slip surface. The minimum calculated factor of safety refers to the smallest factor of safety calculated for all potential slip surfaces considered.

7.2 Seismic Analysis Approach

The 18 September 1998 Scope of Work indicated that a detailed seismic displacement analysis consisting of the following elements would be performed: (i) site response analysis to establish peak acceleration levels induced within the dam by the design earthquake; (ii) seismic slope stability analysis to establish the yield acceleration for the downstream slope; and, if necessary, (iii) displacement analysis to estimate permanent displacements induced by the design earthquake. A seismic stability analysis procedure, as described in the following paragraphs, was employed for this report rather than the detailed seismic displacement analysis.

GeoSyntec originally proposed performing detailed seismic displacement analysis for Lake Petit Dam under the belief that the seismic performance of the dam would control the outcome of the slope stability evaluation. However, based on slope stability analyses performed using the material parameters presented in the dam physical conditions model (Section 5), it has become apparent that the seismic performance of the dam does not control the outcome of the slope stability evaluation. For this reason, and to expedite completion of this report and submittal to GaEPD Safe Dams Program, GeoSyntec used a simpler procedure for the seismic stability evaluation.

The simpler procedure is consistent with the requirements of the GaEPD Safe Dams Program rules and the current state of practice for embankment dam engineering in Georgia. The procedure is also conservative. GeoSyntec intends to complement the simpler seismic stability evaluation presented herein with a detailed seismic displacement analysis that will be submitted as part of the final rehabilitation design report submitted to GaEPD Safe Dams Program. The simplified seismic slope stability evaluation procedure is described in the following paragraph.

The simplified seismic slope stability evaluation procedure consists of establishing the design peak earthquake acceleration for the site and then performing slope stability analyses using a seismic coefficient (i.e., value of pseudo-static acceleration) equal to the design peak earthquake acceleration. If the calculated minimum factor of safety from these analyses exceeds the required value of 1.1, then it is concluded that the dam meets the requirements of the newly adopted GaEPD Safe Dams Program rules. This procedure will generally produce conservative results for the types of design earthquakes that are relevant for most of Georgia, including the Lake Petit Dam project area.

7.3 Input Parameters

7.3.1 Geometry and Material Properties

As previously described in Section 5, one analysis cross section was developed to represent the dam geometry for use in seepage and slope stability analyses (Figure 5-1). Material properties (i.e., soil unit weights and shear strength parameters) for the four materials underlying or comprising the dam, also presented in Section 5, are summarized in Table 7-1. Drained (i.e., effective stress) material properties were used in the steady-state slope stability analyses. Undrained (i.e., total stress) strength properties were used in the seismic slope stability analyses to account for excess porewater pressure changes that would occur within the dam during rapid (i.e., earthquake) loading.

7.3.2 Porewater Pressures

Two porewater pressure scenarios were used in the analyses. These scenarios are summarized below. These scenarios were developed from seepage analyses performed using the finite element computer program SEEP/W. The results from SEEP/W can be presented as pore pressure values at each of the 740 nodes in the finite element mesh. These pore pressure values were input into XSTABL using the porewater pressure grid option in the program. This option allows the user to input a porewater pressure distribution within the dam as a grid of porewater pressures located by X and Y coordinates.

- *Estimated Maximum Porewater Pressure Level (EML)* – The EML condition, as described in Section 6, represents porewater pressures associated with estimated seasonal high water levels within the dam. This scenario is presented in Figure 6-6.

- *Post-Rehabilitation Scenario* - The post-rehabilitation porewater pressure scenario reflects the effect of the trench drain rehabilitation alternative discussed in Section 8 on the EML condition. The details of the post-rehabilitation scenario are presented as part of this section.

7.3.3 Seismic Acceleration

The GaEPD Safe Dams Program rules require that the peak bedrock acceleration, to be used in seismic slope stability analyses, be determined for the dam site as follows:

"All dams and appurtenant structures shall be capable of withstanding seismic accelerations defined in the most current "Map for Peak Acceleration with a 2% exceedance in 50 years" for the contiguous United States published by the United States Geological Survey (a.k.a. NEHRP) maps. The minimum seismic acceleration shall be 0.05g. The seismic accelerations may be reduced or seismic evaluation eliminated if the applicant's engineer can successfully demonstrate to the Director by engineering analyses or judgement that smaller seismic accelerations are appropriate or no seismic evaluation is needed."

The peak bedrock acceleration for the Lake Petit Dam site was obtained from the most current version of the seismic hazard probability maps prepared by the United States Geological Survey for the National Earthquake Hazard Reduction Program (NEHRP) [Frankel et. al., 1997]. The peak bedrock acceleration was selected using the map for Peak Acceleration (% g) with 2% Probability of Exceedance in 50 Years (Figure 7-1), as required by the GaEPD Safe Dams Program rules. The peak bedrock acceleration for the dam site, located at 34° 27' 45" N latitude and 84° 17' 25" W longitude as shown on Figure 7-1, is 0.183 g.

7.4 Slope Stability Evaluation Results

7.4.1 Steady-State Condition

EML Scenario

The minimum calculated factor of safety for downstream embankment stability for steady-state conditions with the EML porewater pressure scenario is 1.52, as given in Table 7-2. This value of factor of safety exceeds the minimum value of 1.5 required by the newly adopted GaEPD Safe Dam rules. The associated potential slip surface is shown in Figure 7-2 and the output file from the XSTABL program is presented in Appendix E.

It is noted that factors of safety with the EML porewater pressure scenario for surficial potential slip surfaces below the G2 bench will be less than 1.5. This is due to the fact that the phreatic surface for the EML condition is at or near the downstream face in this location (Figure 6-6). To remedy this situation, rehabilitation alternatives that lower the phreatic surface in the lower face of the dam have been considered and are presented in detail in Section 8. To evaluate the effect of rehabilitation alternatives on dam stability, the SEEP/W program was used as described below.

Post-Rehabilitation Scenario

Section 8 presents an evaluation of the various rehabilitation measures considered for the Lake Petit Dam. SEEP/W was used to develop a preliminary model for several drainage-based rehabilitation alternatives. Based on the results of preliminary evaluations, the use of two trench drains located on each of the two lowest downstream benches was selected as the optimum arrangement of drainage measures that could be installed using conventional equipment and methods. The preliminary model was developed by setting the node in the finite element mesh at a depth of 12 ft (3.7 m) at each of the two benches to a zero porewater pressure condition (i.e., representative of a fully effective drain), and analyzing the impact of the drains on the EML scenario. A post-rehabilitation porewater pressure scenario from the preliminary model indicates that the phreatic surface would be lowered significantly below the G2 bench (Figure 8-1).

The minimum calculated factor of safety for downstream embankment stability for steady-state conditions with the post-rehabilitation porewater pressure scenario is 1.74, as given in Table 7-2. This factor of safety is greater than the minimum value of 1.5 required by the newly adopted GaEPD Safe Dam Rules and demonstrates the additional benefit that lowered water levels have on downstream embankment stability. The associated potential slip surface is shown in Figure 7-3 and the output file from the XSTABL program is presented in Appendix E.

A surficial stability analysis was performed for steady-state conditions with the post-rehabilitation porewater pressure scenario using the method presented by Koerner and Soong [1998]. The analysis considered a potential slip at a depth of 3 ft (0.9 m), extending between two adjacent benches on the downstream face of the dam. The calculated factor of safety for this potential slip surface is 1.90, which is greater than the minimum value of 1.5 required by the newly adopted GaEPD Safe Dam Rules. The calculation is presented in Appendix F.

7.4.2 Seismic Condition

EML Scenario

The minimum calculated factor of safety for downstream embankment stability for seismic conditions with the EML porewater pressure scenario is 1.46, as given in Table 7-2. This factor of safety is greater than the minimum value of 1.1 required by the newly adopted GaEPD Safe Dam Rules. The associated potential slip surface is shown in Figure 7-4 and the output file from the XSTABL program is presented in Appendix E.

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Post-Rehabilitation Scenario

The minimum factor of safety for downstream embankment stability for seismic conditions with the post-rehabilitation porewater pressure scenario was not calculated for this report but will be larger than the value of 1.46 calculated for the EML scenario.

A surficial stability analysis was performed for seismic conditions with the post-rehabilitation porewater pressure scenario using the method presented by Koerner and Soong [1998]. The analysis considered a potential slip at a depth of 3 ft (0.9 m), extending between two adjacent benches on the downstream face of the dam. The calculated factor of safety for this potential slip surface is 1.21, which is greater than the minimum value of 1.1 required by the newly adopted GaEPD Safe Dam Rules. The calculation is presented in Appendix F.

8. REHABILITATION ALTERNATIVES

8.1 Overview

This section of the report presents a summary of the findings of the evaluation of existing, pre-rehabilitation conditions. This section also addresses GeoSyntec's evaluation of rehabilitation alternatives recommended to enhance performance and slope stability to minimum factors of safety of the GaEPD Safe Dams Program. Consistent with the Scope of Work for this project, GeoSyntec has performed a detailed evaluation of potential rehabilitation measures for the purpose of recommending a preferred set of measures. This section also addresses the proposed schedule for development of detailed plans and specifications, securing GaEPD Safe Dams Program approval, and implementation of the recommended measures.

8.2 Evaluation of Existing Conditions

Previous sections of this report present the procedures and findings of GeoSyntec's evaluation of the stability of Lake Petit Dam.

Stability analyses conducted using existing, pre-rehabilitation conditions and the conservatively established EMLs indicate minimum factors of safety against a downstream embankment slope stability failure of 1.52 under static steady-state seepage conditions and 1.46 under seismic conditions. However, it was noted that lower value factors of safety for surficial potential slip surfaces could be identified due to the fact that the phreatic surface for the EML condition is at or near the downstream face of the dam in its lower portion. Although surficial stability issues are typically addressed during routine maintenance activities, rehabilitation measures are considered appropriate to improve surficial stability, and will have a corresponding beneficial impact on the factors of safety against a downstream embankment slope stability failure under both static steady-state conditions and seismic conditions.

GeoSyntec has conducted a detailed evaluation of potential rehabilitation measures. This evaluation consisted of a two-part process wherein potentially feasible alternatives were first subjectively evaluated and ranked. Then, a more detailed evaluation of the highest ranked measures was performed to assess the likely impact of each measure on the calculated factors of safety for stability of the dam. These potential rehabilitation measures have been evaluated using the existing geometry, measured soil strength properties, and the conservatively-established EMLs.

8.3 Evaluation of Alternatives

8.3.1 Methodology

Consistent with the Scope of Work, GeoSyntec performed an evaluation of a variety of conceptually-reasonable alternatives for dam rehabilitation. Table 8-1 provides a listing of alternatives considered and presents the results of an initial screening of each rehabilitation measure with respect to the primary criteria established in the Scope of Work (i.e., improvement of minimum factors of safety; constructibility, ability to implement within a reasonable schedule, ability to monitor effectiveness, capital cost, and operational and maintenance costs). Alternatives that appeared favorable with respect to the primary screening were then evaluated against the secondary criteria (i.e., road traffic safety during construction, loss of amenities, and impact on the environment). Based on the results of this evaluation process, the performance of the apparent optimum alternative was evaluated using the site physical conditions model as a method of estimating the post-rehabilitation slope stability factors of safety.

8.3.2 Primary Evaluation Criteria

This subsection of the report describes primary evaluation criteria for rehabilitation of Lake Petit Dam.:

Improvement of Minimum Factors of Safety – this criterion addresses the benefit of the rehabilitation measure in increasing minimum downstream slope stability factors of safety under both static steady-state seepage conditions and seismic conditions.

Constructability – this criterion addresses ease of construction considering the use of conventional construction equipment.

Schedule – this criterion addresses the likelihood of being able to design the rehabilitation measure and commence implementation by April 1999, together with the likelihood of completing implementation during one construction season.

Ability to Monitor Effectiveness – this criterion addresses the ability to monitor the effectiveness or benefits achieved through implementation of the rehabilitation measure using conventional instrumentation and monitoring techniques.

Capital Cost – this criterion considers both the magnitude of the capital cost together with the benefits achieved relative to the expended funds.

Operational and Maintenance Cost – this criterion addresses the relative annual operational and maintenance cost associated with implementing the rehabilitation measure.

8.3.3 Scoring System

The scoring system used in preparing Table 8-1 consists of a 1 through 5 rating, with a score of 1 being low and 5 being high. In the case of criteria involving cost, a low score is associated with high cost and conversely, a high score is associated with a rehabilitation measure that has a relatively low cost compared to the incremental benefit achieved. For the purpose of the primary evaluation of alternatives, a total score of 20 points was arbitrarily established as a threshold for carrying the measure forward to a more detailed evaluation.

8.3.4 Detailed Evaluation of Rehabilitation Measures

This section presents a brief summary of the evaluation process and scoring of each alternative considered.

Toe Drains – this measure would consist of constructing a relatively shallow (i.e. less than 15-ft deep) toe drain at the base of the downstream slope of the dam. This measure would result in an increase in the calculated factors of safety particularly with respect to simulated failure surfaces that have a large proportion of their length close to the toe of the dam. However, this measure would have a limited effect on deep failure surfaces or long shallow surfaces remote from the toe. This measure is considered easily constructable using conventional equipment within the preferred schedule. Its performance could be measured using conventional instrumentation (piezometers and flow meters or weirs). Capital costs would be low relative compared to the benefits achieved. If adequately designed, operational costs should be minimal.

Bench Drains – this measure would consist of undercutting the toe of the downstream slope at the inside edge of selected benches. This material would be replaced with free draining media. This measure would intercept the phreatic surface in areas where the phreatic surface comes close to the existing ground surface. As a result, a slight lowering of the phreatic surface would be achieved and seepage water would be conveyed to surface water drainage systems in a controlled manner. Although this measure alone may only result in a marginal improvement in calculated factors of safety, it is considered beneficial in terms of controlling the release of seepage water. This measure is considered easily constructable using conventional equipment within the preferred schedule. Its performance could be measured using conventional instrumentation (piezometers and flow meters or weirs). Capital costs would be low relative to the benefits achieved. If adequately designed, operational costs should be minimal.

Pressure Relief Wells – this measure would consist of drilling extraction wells close to the toe of the dam. These wells would be fitted with permanent electrical pumps. The overall effect would be similar to the use of a toe drain except the wells could be drilled deeper and thus have a greater zone of influence. This measure is considered easily constructable using conventional equipment within the preferred schedule. Its performance could be measured using conventional instrumentation (piezometers and flow meters or weirs). Compared to toe drains, the capital cost of this alternative would be relatively high compared to the benefits achieved, due to the relatively shallow depth to the rock. Operational and maintenance costs would be relatively high due to the need to operate and maintain electrical pumps and associated controls.

Trench Drains – this measure would be similar to the toe drain alternative, but could be applied to one or more benches on the downstream slope. Multiple drains could be used to lower the phreatic surface in strategic areas to achieve the desired incremental increase in calculated factors of safety. This measure is considered easily constructable using conventional equipment within the preferred schedule. Its performance could be measured using conventional instrumentation (piezometers and flow meters or weirs). Capital costs would be low relative to the benefits achieved. If adequately designed, operational costs should be minimal.

Horizontal Wells – this measure would consist of drilling a series, or multiple series, of horizontal wells into the downstream face of the dam. The location of the wells would be strategically located to provide porewater pressure relief and lower the phreatic surface. By drilling near horizontal from the existing face of the dam, specific zones deep in the shell could be targeted. Thus, locations that could not be reached by conventional trench drains could be targeted. When compared to trench drains, disadvantages of this approach include: (i) the installed drains would be difficult to construct in a manner that would ensure operation without clogging; (ii) routine cleaning and maintenance would be difficult to implement; and (iii) heterogeneities within the shell material may result in unpredictable effectiveness. This measure is considered more difficult to construct, requiring the use of specialized equipment. Although its performance could be measured using conventional instrumentation, predictability of its effectiveness together with potentially difficult maintenance provides reason to rate this measure lower than trench drains in several criteria.

Toe Buttress (Earth Fill) – this measure would consist of constructing an earthen buttress and blanket drain over a portion of the existing downstream slope of the dam. The additional mass of the buttress would provide increased resistance to rotational or translational failure of the downstream slope, thereby improving downstream slope stability factors of safety. The blanket drain would provide porewater pressure relief and prevent an increase in porewater pressure resulting from the increased load. This alternative could be constructed using conventional earth moving equipment, but would be time consuming due to the large amount of imported fill required. Its performance could be measured using conventional instrumentation (e.g., piezometers and inclinometers). Capital costs would be high relative to the benefits achieved. Operation and maintenance costs would be similar to present costs. This measure is similar to that recommended by Piedmont and JJ&G.

Toe Buttress (Reinforced Soil or Rock) – this measure would be essentially the same as the earth fill, but the fill material (imported soil or rock) would be reinforced using geogrids to allow the use of steeper fill slopes. This would result in more efficient use of fill materials (i.e., lower volume of material strategically located to provide optimum benefit). This alternative could be constructed using conventional earth moving equipment. The time required to implement this measure would be slightly less than the earth fill due to the reduced volume of imported fill required. Its performance could be measured using conventional instrumentation (e.g., piezometers and inclinometers). Capital costs would be high relative to the benefits achieved. Operation and maintenance costs would be higher than present costs due to the need to maintain steeper slopes (more difficult mowing access).

Toe Buttress (Retaining Wall and Backfill) – this measure would function in a similar manner to the previously described toe buttresses, but would utilize a vertical or near vertical retaining wall to further optimize the use of the backfill. This alternative could be constructed using conventional earth moving equipment. The time required to implement this measure would be similar to other buttress measures. Its performance could be measured using conventional instrumentation (e.g., piezometers and inclinometers). Capital costs would be very high relative to the benefits achieved due to the high cost of the retaining wall. Operation and maintenance costs would be relatively low assuming adequate design of the wall and a relatively flat slope to the retained fill.

Grouting – this measure would consist of pressure grouting strategically selected zones of the exiting dam in an attempt to reduce seepage through the structure. Reducing the seepage would result in a general lowering of the phreatic surface and a corresponding increase in the calculated factors of safety. Published literature indicates that the success of grouting in similar situations is typically low, due to the difficulties associated with both locating zones to be grouted and with controlling the placement of the grout. Implementation of this measure would be difficult and would require the use of specialized methods and equipment, with a high degree of uncertainty with respect to the ultimate success. The time required to implement this measure would be difficult to predict since successive grouting attempts may be needed. Although the effectiveness could be monitored using conventional instrumentation (i.e., piezometers), multiple instrumentation points could be required to assess localized effects. Due to the lack of predictability of the success of this measure, the capital cost and operation and maintenance costs associated with this measure are assumed to be high relative to the benefits achieved.

Subsurface Barrier – this measure would consist of installing a subsurface barrier such as a slurry wall or grout curtain in the core of the dam to provide a physical barrier of low permeability material for the purpose of reducing the rate of seepage. The barrier could be either partially or fully penetrating. Reducing the rate of seepage would result in a general lowering of the phreatic surface and a corresponding increase in the calculated factors of safety. Implementation of this measure would likely be effective, but would require the use of specialized construction techniques due to the height of the dam. The performance of this measure could be monitored using conventional instrumentation (i.e., piezometers). The capital cost would likely be very high compared to the benefits achieved. Present operational costs would be relatively unaffected.

Upgradient Impermeable Barrier – this measure would consist of installing a low permeability barrier on the upgradient face of the dam to reduce seepage. The barrier could consist of either a clay layer or a geosynthetic material. Although methods are available for installing geomembranes under water, it would likely be necessary to lower the lake pool level to some degree to ensure optimal installation and performance. Uncertainties exist with respect to the likely effectiveness of this measure due to incomplete characterization of seepage through the buttress of this relatively narrow dam. The time required to implement this measure would be dependent on the time frame required to lower the pool level. Effectiveness could be monitored using conventional instrumentation (i.e., piezometers). Both capital and operation and maintenance cost are expected to be high.

Impermeable Membrane (Downgradient Slope) – this measure consists of installing a geomembrane over the downstream slope of the dam, for the purpose of reducing infiltration of surface water. By substantially reducing infiltration, the effects of seasonal variations in the phreatic surface (described in Section 4) would be minimized, resulting in lower porewater pressures for long-term conditions. As described in Section 4 and subsequent sections, the effects of using lower porewater pressures such as those measured in October 1998, results in higher calculated slope stability factors of safety than those calculated for EML conditions. This measure could be implemented using conventional methods and equipment within an acceptable schedule. Its performance could be monitored using conventional instrumentation (i.e., piezometers). The capital cost of this alternative would be high compared to the incremental benefits. Operational and maintenance costs may increase slightly as a result of the installation (surface erosion may increase slightly).

Surface Water Drainage – this measure would consist of installing surface water drains or gutters on the existing downstream slope benches for the purpose of collecting and conveying surface water away from the slope. This would reduce infiltration having similar effects to the use of an impermeable membrane (described previously). Although slightly less effective than a membrane, this measure could be implemented using conventional equipment within a short schedule. The capital cost of this alternative would be low compared to the incremental benefits. Operational and maintenance costs would be minimal.

Paved Benches – this measure would be similar to surface water drainage (described above) but would involve paving the entire width of existing benches. As a result, effectiveness (in terms of reducing the potential for ponding of surface water and infiltration) would be increased. This measure could be implemented using conventional equipment within a short schedule. The capital cost of this alternative would be relatively low compared to the incremental benefits. Operational and maintenance costs would be minimal.

The results of the primary screening process are presented in Table 10-1. Based on this evaluation, the following rehabilitation measures were retained for further evaluation:

- toe drains;
- bench drains;
- trench drains;
- toe buttress (earth fill) – retained for comparison;
- surface water drainage; and
- paved benches.

Consistent with the Scope of Work for this project, secondary criteria used to evaluate potential rehabilitation measures were: (i) road traffic safety during construction; (ii) loss of amenities; and (iii) impact on environment. Clearly, the earth fill toe buttress will have the most significant impact on all three secondary criteria. Construction of this rehabilitation alternative would involve the movement of significant quantities of imported fill materials resulting in road traffic safety concerns within the Big Canoe development and surrounding areas. The construction of the toe buttress would also result in the loss of community amenities, i.e., the ball ground, jogging trail, picnic area, road at the base of the dam, and potentially several adjacent properties. Environmental impacts would be obvious at both the immediate vicinity of the dam and at the selected borrow pit locations.

Consistent with the objectives outlined in the Scope of Work, GeoSyntec has developed a rehabilitation alternative that consists of a combination of several of the remaining alternatives that survived the primary screening process. This rehabilitation alternative utilizes bench drains, trench drains, and paved benches (i.e., several of the highest scoring measures) that, when combined, provide a significant increase in the calculated factors of safety against both downstream embankment stability and surficial slip failures.

8.4 Recommended Rehabilitation Alternative

The recommended rehabilitation alternative consists of the combination of three of the evaluated measures i.e., bench drains, trench drains, and paved benches. Each measure is designed to provide the following function:

Trench Drains – will be provided as a primary method of lowering the porewater pressures within the downstream shell of the dam, resulting in a lowering of the phreatic surface locally. Trench drains will be strategically located on the lower benches, as presented in Figure 8-1, to provide optimum benefit with respect to control of seepage and attendant increase in calculated factors of safety.

Paved Benches – will be provided as the primary method of reducing infiltration of surface water into the downstream face of the dam. In addition to providing a relatively impermeable surface to ponded water, the surfaces will be sloped to convey surface water to the abutments where water will be conveyed by ditches or pipes to the tailwater creek.

Bench Drains – will be provided as a secondary measure to control the release of any seepage water or infiltration that comes close to the surface of the dam. The bench drains will be integrated into the paved benches. A typical cross section is presented in Figure 8-2.

The calculated minimum factors of safety against downstream slope failure under steady-state seepage conditions and seismic conditions are 1.7 and more than 1.4, respectively when the beneficial effects of proposed rehabilitation measures are incorporated into the slope stability analyses. These factors of safety were calculated using the EML porewater pressure distribution incorporating the effects of the trench drains. The use of the EML porewater pressure distribution incorporating the effects of trench drains is considered very conservative, since it essentially ignores the benefits provided by the paved benches (i.e., reduction in infiltration).

The conceptual design of the rehabilitation measures in Figure 8-1 shows a trench drain depth of 12-ft. for analysis. The location, number, and depth of trench drains will be further evaluated as the detailed design of the rehabilitation program is developed. In addition to the measures described, the final rehabilitation program will also incorporate appropriate instrumentation to monitor the effectiveness of the implemented measures. As a further commitment to operating a safe dam, the Big Canoe POA is also proposing to revise the recently submitted Emergency Action Plan to incorporate an active instrumentation monitoring program. This will ensure appropriate early notifications in the unlikely event that conditions deviate from design conditions, and will ensure that the Lake Petit Dam exceeds the regulatory requirements with respect to safety and monitoring of Category I Dams in the State of Georgia.

8.5 Proposed Schedule

Figure 8-3 presents the proposed schedule for development and implementation of the proposed rehabilitation program. Consistent with the GaEPD Safe dams Program, the schedule is based on an April 1999 start of construction. A detailed schedule for construction will be submitted as part of the final design. This detailed schedule will show the completion of all dam rehabilitation work during the 1999 construction season.