GEOSYNTEC CONSULTANTS

1100 Lake Hearn Drive, Suite 200
Atlanta, Georgia 30342-1523 • USA
Tel. (404) 705-9500 • Fax (404) 705-9400

1 April 2002

Mr. Francis E. Fiegle II, P.E.
Program Manager
Georgia Department of Natural Resources
Environmental Protection Division – Safe Dams Program
4244 International Parkway – Suite 110
Atlanta, GA 30354

Subject: Response to 17 December 2001 Comments
Lake Petit Dam
Pickens County, Georgia

Dear Mr. Fiegle:

This letter was prepared to provide relevant information, clarifications, and comments related to the 36 comments that were provided to GeoSyntec Consultants (GeoSyntec) by the Safe Dams Program (GaSDP) in a 17 December 2001 letter from Mr. Francis E. Fiegle, II, P.E. The GaSDP comments were related to the December 1998 document titled *Evaluation of Stability and Rehabilitation Measures, Lake Petit Dam, Big Canoe, Georgia* (1998 report) that was prepared by GeoSyntec and provided to GaSDP for review. In the remainder of the letter, each GaSDP comment is presented in italic type, followed immediately by the GeoSyntec response.

Comment 1: *Our office is unfamiliar with the software SEEP/W. Please provide a copy of the software documentation/user's guide for our review.*

Response 1: SEEP/W is a finite element analysis computer program that models flow through a porous media. Output from the program can be used to generate flowlines, equipotential lines, and flow nets in saturated and unsaturated soil and rock materials. The analytical methods employed by SEEP/W are described in the Technical Overview Section of the SEEP/W users manual. The users manual for SEEP/W is provided in its entirety in Attachment R1.

Comment 2: *Our office surveyed the downstream slope. That survey showed the slope to vary from 2.17 H to IV to 2.67H to IV with the steepest section at the bottom of the dam. Attached is a copy of the sketch depicting the slope. It would appear that evaluating the entire slope at 2.5 H to IV is* *not entirely representative of what exists in the field, especially with the steepest section being at the downstream toe of the dam.*

Response 2: The cross section selected for analysis was defined using ground survey information for the Lake Petit Dam obtained by Jordan, Jones & Goulding, Inc. of Atlanta, Georgia. This survey information is presented in Attachment R2. The cross-section was shown in Figure 2-1 of the 1998 report. The slopes shown in this cross section of the downstream embankment (as measured between incremental benches) range between 2.25H:1V to 2.61H:1V. These slopes are generally consistent with the slopes measured by GaSDP. GeoSyntec notes that the sketch referred to in Comment 2 was not provided with the 17 December 2001 letter from GaSDP.

Comment 3: *Your report references geotechnical evaluations and reports completed by Piedmont Geotechnical Associates dated May 29, 1997 and April 1, 1998. Our office does not have copies of those reports. Please submit them because they should include drilling logs, shear testing information, etc.*

Response 3: GeoSyntec inadvertently misquoted the date of the earlier of the two reports mentioned in Comment 3 as that of a previous revision. The correct date for the current revision of this report is 6 June 1997. GeoSyntec also inadvertently misquoted the date of the later report. The correct date of this report is 3 February 1998. This document is also provided in Attachment R3.

Comment 4: *In your executive summary, you note on page 1 that there is a high phreatic surface may develop near the downstream toe at the time of seasonal high water (ie spring/winter rains). However, your permeability testing of soil samples indicate 10^{-6} permeability. How can enough rain infiltrate the slope to cause this condition to occur?*

Response 4: GeoSyntec notes that permeability testing of samples was not performed as part of the 1998 investigation. Values of the vertical and horizontal hydraulic conductivity for dam shell and core materials were established based on published correlations to particle size and plasticity information and based on seepage analysis results. For the shell material, a vertical hydraulic conductivity of 1.4×10^{-4} cm/sec was used. This value corresponds to an SM soil classification and "*pervious to semipervious*" characterization of the hydraulic conductivity attributed to these type materials [USBR, 1987 Figure 5-14] according to information reported in Design of Small Dams (Figure 5-14). The horizontal hydraulic conductivity used for the shell material was 1.4×10^{-3} cm/s which reflects anisotropic characteristics typical of compacted materials [Sherard et al., 1963, Sections 3.4, 5.4, and 7.6]. GeoSyntec considers the embankment shell material to be sufficiently pervious to allow the progressive rise in water level within the embankment due to surface infiltration, as observed in piezometer readings. It is noted that because of this potential for progressive rise, the GeoSyntec recommendations for rehabilitation included measures to reduce infiltration of ponded or flowing runoff though benches located on the downstream embankment.

Comment 5: *A seepage flow net needs to be developed for the dam and submitted. Furthermore, how the seasonal wet phenomenon can occur given the lab testing results must be rectified.*

Response 5: Seepage flow nets were presented as Figures 6-3 and 6-5 of the 1998 report for the "23 October 1998" and "EML scenarios", respectively, as described in Section 5.3 of the 1998 report. Regarding the seasonal wet phenomenon and laboratory testing results, GeoSyntec provided an explanation in the response to Comment 4. GeoSyntec's rehabilitation recommendations will address this observed phenomenon.

Comment 6: *Do you have more extensive data now from the piezometers that were installed? If so, please submit it. If not, why not? Do the piezometer readings show a significant rise in the spring? Do you have any site rainfall gauge records that account for any type of response in the piezometers that support your theory about rainfall infiltration?*

Response 6: Water level data for the vibrating wire piezometers and the instruments referenced as the 1998 standpipe piezometers are presented in Attachment R6. Data from the vibrating wire piezometers were collected for the following time intervals: (i) June 1999 to September 1999; (ii) July 2000 to November 2000; and (iii) December 2001 to March 2002. Data from the 1998 standpipe piezometers were collected from December 1998 to April 1999 and in February 2002. The 1998 standpipe piezometer data indicate a significant rise in water level (0.3 to 4.1 ft) within the embankment between 23 October 1998 and April 19, 1999, as exhibited by the vibrating wire piezometer readings of the previous year.

The collection of readings from the 1998 standpipe piezometers was discontinued in April 1999 in favor of monitoring water levels within the dam using the vibrating wire piezometers. The monitoring of the vibrating wire piezometers is ongoing and is expected to be continued in the future at regular time intervals.

Monthly total rainfall data from October 1997 to October 1998, recorded at a location approximately 1 mile from the dam, are presented in Attachment R6. These data are compared to the data from Jasper, Georgia that were presented in Figure 4-1 from the 1998 report. The seasonal magnitude and variation of the rainfall data are generally consistent with the near-site rainfall data reported in the 1998 report for Jasper, Georgia.

Comment 7: *Generally speaking, the correlation of the SPT blow count (N) values to effective shear stress are skewed by the presence of gravel of 2 to 12 percent as reported. The gravel encountered drove the blow count up and the documentation for ϕ' correlations notes that it is for clean to silty sands and not gravel.*

Response 7: GeoSyntec agrees with the first portion of GaSDP's above comment. As stated in the 1998 report, GeoSyntec recognized that the Schmertmann [1975] and Hatanaka and Uchida [1996] correlations were developed for sandy to silty sandy soils and that these correlations are expected to become less reliable with increasing amounts of gravel. That is one of the reasons why GeoSyntec used the correlations as a secondary basis for evaluating the shear strength of the embankment fill. The primary basis for selecting the effective stress friction angles used in static steady-state slope stability analyses was the results of the laboratory testing program using materials recovered from the site.

Comment 8: *Page 8 of your report references the engineering plans and material specifications in Section 2.3.1. Please provide copies of that information including all the construction plans, construction and material specifications, and as-built plans if they are available for our review. If you do not have a complete set, who does?*

Response 8: The engineering plans and material specifications referenced in Section 2.3.1 of the 1998 report are as-built drawings prepared by Cranston, Robertson & Whitehurst, P.C. (formerly Baldwin and Cranston Associates, Inc.) of Augusta, Georgia. These plans were prepared for the original construction of the dam and are dated November 1971. A set of these drawings was provided to GaSDP by Cranston, Robertson & Whitehurst, P.C. under a cover letter dated 26 February 2002.

Comment 9: *In Section 2.3.3, on page 10, you note that the compression wave velocities are consistent with published values for unsaturated and nearly saturated soils. What is the range for saturated soils? What constitutes "nearly saturated" soils?*

Response 9: Published values of compression wave velocity generally range from 600 ft/s to 3,000 ft/s for unsaturated soils and from 1,500 ft/s to 7,000 ft/s for saturated soils [Burger, 1992]. In general, GeoSyntec believes that the term "nearly saturated" should be adjusted to simply "saturated" and apologizes for this confusion.

Comment 10: *In Section 3.2.1 on page 12, you note that total stress strength parameters should be used for the seismic stability analysis if the soil is not prone to cyclic pore water pressure build-up during an earthquake.*

What type of soils are not prone to this build up? How is this applicable in this situation given the high phreatic line?

Response 10: Well-compacted and overconsolidated fill materials commonly develop little or negative pore pressure buildup (i.e., are not prone to cyclic pore pressure buildup) during the rapid cyclic loading characteristic of earthquake loadings [Mitchell, 1993 – Section 11.7; Mutsui et al., 1980]. Negative pore pressures have been shown to increase under cyclic loading and persist for a much longer duration than is characteristic of the design earthquake magnitude for the site [Matsui et al., 1980; Weigel, 1970]. In the context of Lake Petit Dam, eleven of the 16 triaxial specimens tested as part of the 1998 GeoSyntec laboratory testing program exhibited negative pore pressure at the end of shearing under static triaxial testing. GeoSyntec notes that the presence of a high phreatic surface will not affect the tendency to develop either negative or positive pressures within the dam fill during cyclic loading.

Comment 11: *Please provide a copy of the "Report of Engineering Evaluation Lake Petit Dam" by Law Engineering dated March 18, 1974 for our review.*

Response 11: A copy of the above report was provided to GaSDP by Cranston, Robertson & Whitehurst, P.C. under a cover letter dated 26 February 2002.

Comment 12: *Section 4.3 and Section 4.5.1 imply that the embankment has a porous, homogeneous medium, but Section 4.3 qualifies that reasoning by stating that the embankment medium is also anisotropic. Generally speaking, the implied conclusion is not consistent depending on which set of circumstances you are addressing. The embankment is either a porous, homogeneous medium or a porous, anisotropic medium.*

Response 12: As stated in Section 4.3 of the 1998 report, GeoSyntec concluded from an initial evaluation of vibratory wire piezometer data that these data were "indicative of steady-state seepage through a porous homogeneous, but anisotropic medium." The qualification of the flow regime as anisotropic was in consideration of the common condition for compacted earth dam embankments where hydraulic conductivity is greater in the horizontal direction than the vertical direction. GeoSyntec then constructed a preliminary flow net using the existing vibratory wire piezometer data that was used as a planning aid to select locations and monitoring intervals for additional piezometers (i.e., the subsequently installed 1998 standpipe piezometers). Based on a review of a combined set of vibrating wire and standpipe piezometer data, GeoSyntec concluded that the flow regime within the embankment reflected steady-state seepage conditions through a zoned earth dam with a core and downstream shell. Both the shell and the core were assumed to exhibit anisotropic hydraulic conductivity values.

Comment 13: In Section 4.5.2, you relate the monthly precipitation records for Jasper, Georgia to the piezometer readings at the dam. Do you have any rainfall data that is nearby as opposed to 20 miles away? Do you have any more current data that supports this conclusion?

Response 13: In response to the first question of Comment 13 regarding rainfall data, monthly total rainfall data recorded from October 1997 to October 1998 at a location approximately one mile from the dam were presented in Attachment R6. As previously discussed in Response 6, the seasonal variation and magnitude of these rainfall data are consistent with the rainfall data reported in the 1998 report for Jasper, Georgia. Water level measurements for the 1998 standpipe piezometers recorded between October 1998 and April 1999 show an increasing trend consistent with that exhibited by vibrating wire piezometer readings from the same months of the preceding winter/spring seasons.

Comment 14: Given that the maximum variation for piezometer P-4A is 6.9 feet, are you sure there is no surface infiltration occurring down the borehole?

Response 14: GeoSyntec has inspected the ground in the vicinity of the piezometer and is not aware of any conditions which would suggest that surface-water infiltration at the piezometer casing contributed to the relatively large water level variation measured for piezometer P-4A.

Comment 15: *In Section 5.2.1, how did you arrive at an average moist unit weight of 125 lbs/ft^3 given that the boring logs note significant mica in the samplings which is further confirmed by a number of dry unit weights of less than 100 lbs/ft^3 for the shell materials?*

Response 15: As stated in the referenced section, the average moist unit weight of 125 lb/ft^3 was calculated directly from dry unit weight and moisture contents obtained from triaxial shear testing specimens (i.e., intact specimens obtained from Shelby tubes). This average value included data for all specimens identified in Table 3-1 as "Shell material" and incorporates the effect of the actual amount of mica present in the specimens.

As a clarification, GeoSyntec notes that the boring logs from the 1998 GeoSyntec field investigation (1998 investigation) describe the soil encountered as "micaceous". This description was not intended to imply that the fill within the Lake Petit Dam contains an unusually high mica content. Rather, this description simply indicates that mica was evident. It is generally understood that mica in most residual soils contributes to low total unit weights and corresponding high void ratios [Sowers and Richardson, 1983]. Undisturbed samples taken from the Lake Petit Dam exhibited void ratios ranging from 0.47 to 0.73, and averaging 0.62. This range does not indicate an unusually high mica content [Sowers, 1994].

Comment 16: *Please provide the mohrs circle plots, void ratio information, etc that led to the conclusion that the peak strength condition of ϕ' angle which seems high for soils with high mica content.*

Response 16: The stress paths used to obtain effective stress friction angles were provided in Appendix C of the 1998 report. The Mohr circles used as the basis for the stress paths along with the calculated peak friction angles and initial specimen void ratios are presented in Attachment R16. The use of stress paths to obtain friction angles is described in Holtz and Kovacs [1981]. The use of stress paths to obtain friction angles for Piedmont residual soils is illustrated in the paper "Residual Soils of the Piedmont and Blue Ridge" by Sowers and Richardson [1983].

GeoSyntec wishes to clarify how effective stress friction angles, for both peak and ultimate strength conditions, were calculated from triaxial test results. Friction angles for both conditions were calculated as the arcsine of the ratio of values of q and p' corresponding to peak and ultimate strength conditions. To obtain values of p' and q for peak strength conditions, a plane of maximum obliquity was drawn from the origin of the plot to the curve representing the measured test data (this line is shown along with selected values of p' and q on the plots provided in Attachment R16). Values of p' and q for ultimate strength conditions were taken directly from tabulated test results presented in Appendix C of the 1998 report for end of test conditions.

GeoSyntec wishes to emphasize that the effective stress shear strength parameters used in static, steady-state seepage analyses were selected as the lowest test value calculated for the ultimate strength condition (at approximately 15% axial strain). These test results are presented in Attachment R16 on a p'-q plot from Sowers and Richardson [1983] which presents results from numerous triaxial tests for the Metropolitan Atlanta Rapid Transit Authority which includes a range of different gneiss and schist parent rocks, different depths, and different void ratios. Test results presented in Sowers and Richardson appear to be from undisturbed samples obtained from natural deposits, it is unclear whether these test results reflect peak or large strain conditions.

Regarding mica content, no testing of soil for mica content was performed. As stated in GeoSyntec's response to Comment 15, boring logs do not describe the soils as having a high mica content. The calculated friction angles were obtained directly from test results that include the effect of the actual amount of mica present in the specimens.

Comment 17: *In Section 5.2.2, how did you arrive at an average moist unit weight of 130 lbs/ft^3 given the boring log notes about mica content of the samplings and some dry unit weight of 105 lbs/ft^3 and less?*

Response 17: As stated in the referenced section, an average moist unit weight of 131 lb/ft^3 was calculated directly from dry unit weight and moisture contents obtained from triaxial shear testing specimens (i.e., intact specimens obtained from Shelby tubes). This average value included data for all specimens identified in Table 3-1 as "Core material" and incorporates the effect of the actual amount of mica present in the specimens.

Comment 18: *Please provide the mohrs circle plots, void ratio information, etc that led to the conclusion that the peak strength condition of 40° to 41° for the core materials. Did this value occur because of gravel within the samples? Use of a 34° ϕ' angle seems high for soils with high mica content.*

Response 18: The stress paths used to obtain shear strength parameters were provided in Appendix C of the 1998 report. The Mohr circles used as a basis for these stress paths plots along with the calculated peak friction angles and initial specimen void ratios are presented in Attachment R18. Peak friction angles for core material specimens were obtained directly from triaxial test results as described in GeoSyntec's response to Comment 16. For clarification and as stated in Section 5.2.2 of the 1998 report, GeoSyntec notes that the effective stress friction angle used in stability analyses for core material is 32°. This value is 2° below the lowest value measured for the ultimate strength condition recorded in the laboratory tests.

Regarding mica content, no testing of soil for mica content was performed and, as stated in GeoSyntec's response to Comment 15. Boring logs do not describe the soils as having a high mica content. The calculated friction angles were obtained directly from test results that include the effect of the actual amount of mica present in the specimens.

Regarding the potential affect of gravel on the measured friction angles, GeoSyntec believes that considering the relatively low gravel content within the specimens and the relatively small size of the particles, that the presence of this gravel in triaxial specimens had little influence on the measured strengths. Index testing results using material from the triaxial samples including: (i) gravel content; (ii) maximum particle size; and (iii) peak effective stress friction angle are tabulated in Attachment R18. These data show that specimens with lower gravel content (between 2 and 6 percent) exhibited friction angles higher than for specimens with gravel a higher gravel content (between 7 and 12 percent) which infers that the gravel within the samples was not sufficient to increase the measured strength. It is generally understood that in cases where the gravel is embedded in a matrix of much smaller particles, shearing takes place through the matrix hence the presence of the gravel has little influence on test results [Lambe and Whitman, 1969].

Comment 19: *What was the final ratio of the horizontal to vertical hydraulic conductivity (k_n/k_y) used for the shell and core material? Similarly, what was the vertical hydraulic conductivity used or did it vary by location? If so, how and why?*

Response 19: For the analyses performed, the vertical hydraulic conductivity of the dam shell material was assumed based on grain size and plasticity information. The assumed vertical hydraulic conductivity of 1.4×10^{-4} cm/s was applied to the entire dam shell. The vertical hydraulic conductivity of the dam core material and the ratio of horizontal to vertical hydraulic conductivity for both the dam shell and dam core were developed as part of the SEEP/W analyses. Values were selected to obtain a best fit of porewater pressure head to piezometer measurements recorded on 23 October 1998 for the "23 October 1998" scenario. The results indicated a best-fit ratio of horizontal to vertical hydraulic conductivity (i.e., k_n/k_y) for both the dam shell and dam core is approximately 10. The best-fit vertical hydraulic conductivity for the dam core is 4.9×10^{-5} cm/s. These values were presented in Table 6-2 of the 1998 report.

Comment 20: *On page 28, you use the October 23, 1998 data to develop your model. You note the phreatic line is 10 to 20 feet below ground surface depending on location within the dam. Given that October is the driest month of the year typically and there was a drought as well, have you re-evaluated your model with more current data during the rainy periods of late winter/early spring to see if it is valid?*

Response 20: GeoSyntec wishes to clarify that although the 23 October 1998 data were used to develop the general model, the EML scenario seepage model data were used to obtain pore pressure data used in the slope stability evaluation of the Lake Petit Dam. Pore pressures as presented in the "23 October 1998" scenario seepage model have not been revised for piezometer data recorded since submission of the 1998 report because it reflected conditions at that time. However, a comparison was made between the piezometer data recorded since submission of the 1998 report and pore pressure head values used in stability analyses. The results of this comparison showed that standpipe piezometers readings from the spring of 1999 were on average 3.0 ft below those used in stability analyses. Thus, the values of pore pressure head used in the 1998 stability analyses are conservative with respect to this more recent data. Accordingly, GeoSyntec recommends no revision of the general seepage model represented by the "23 October 1998" scenario.

Comment 21: *On page 33, you set a boundary condition of 1517 msl to maintain the water level at the ball field at the same evaluation as the outlet elevation of the drain into the creek. If the drain collects water upstream, then the invert (1520) would be higher than 1517 as well as the water table at the field. This boundary condition should be set higher unless there is piezometeric data that the ball field/beyond the toe that shows otherwise.*

Response 21: GeoSyntec wishes to clarify that the elevation of 1517 ft above mean sea level (MSL) stated in the 1998 report corresponds to the discharge point of the drain pipe at the creek and the elevation of 1520 ft MSL corresponds to the approximate elevation of the invert of the drain pipe below the downstream embankment of the dam consistent with as-built drawings for the original dam construction. Accordingly, for use in the seepage analyses, GeoSyntec defined the water level within the field at 1517 ft MSL. GeoSyntec agrees that the water level within the area of the ball field may be higher than 1517 ft MSL. Test pits dug at the ball field in 1998 encountered groundwater at an average depth of approximately 8 ft or approximately 1522 ft MSL. However, the effect of an increase in the water level (i.e., of 5 ft) within the ball field area has no effect on the calculated static stability minimum factor of safety.

Comment 22: *How do you arrive at an existing static slope stability safety factor of 1.52 for static steady-state seepage conditions and a seismic safety factor of 1.46 for the steady-state seepage condition with a horizontal load of 0.183 g? This is not consistent with stability analyses that our office has seen for the last 23 years by numerous geotechnical firms.*

Response 22: As demonstrated in the 1998 report, laboratory testing results using specimens from the site were used to estimate undrained (i.e., short-term) and drained (i.e., long-term) strengths. These results were used in the stability analyses, therefore, different shear strength parameters were used for the static steady-state seepage and seismic stability analyses. The relatively small difference between the calculated minimum factors of safety for static and seismic loading conditions reflects the relatively greater shear resistance of the dam fill during undrained loading characteristic of earthquake motions.

The technical justification for using the technical approach adopted by GeoSyntec was discussed at length in the white paper "*Seismic Stability Analyses, Lake Petit Dam, Big Canoe, Georgia*" (1999 white paper). This paper was transmitted to GaSDP in March 1999. As demonstrated in the white paper, the GeoSyntec approach for the seismic analyses conducted for the Lake Petit Dam project are consistent with guidance used by USACE, USBR and procedures that have been reviewed/accepted by FERC on similar dam projects.

Comment 23: *Are elevations for the Law Piezometers in msl? Where are they located? In Table 4-1, is the October 29, 1997 value elevated due to a poor surface seal for L5?*

Response 23: The elevations for the Law piezometers reference MSL. These piezometers are located at soil boring locations L3, L4, and L5 shown on Figure 2-1 of the 1998 report. It is unclear, whether or not (or how much) surface-water infiltration may have contributed to the Piedmont Geotechnical Associates reported 6.9 ft water level rise in piezometer L5 between the 13th and 29th of October 1997. However, GeoSyntec notes that at the Jasper Weather Station, a total of 8.75 inches of rainfall was recorded between 25 and 28 October 1997. This relatively large amount of rainfall suggests, at least, the potential that the 29 October 1997 reading for L5 may have been influenced by surface-water infiltration.

Comment 24: *What elevations are the results for Table 4.2 referenced to?*

Response 24: The water level data presented in Table 4-2 were inadvertently labeled as "water elevation". These data represent height of water in feet above the piezometer transducer. A corrected Table 4-2 is presented in Attachment R24.

Comment 25: *What is the value of Table 4-3? It only covers seven days worth of readings?*

Response 25: GeoSyntec agrees that the piezometer readings presented in Table 4-3 represent conditions over a relatively short period of time. The value of these readings is to demonstrate that water levels in the 1998 standpipe piezometers had stabilized by 23 October 1998. These 23 October 1998 piezometer readings were used in combination with vibrating wire piezometer readings, obtained on 23 October 1998, to calibrate the seepage model that was used to represent the flow regime within the dam on that date. This combined piezometer data represented the largest set of data available at the time the 1998 report was prepared. For application to stability analyses, this 23 October 1998 calibrated seepage model was modified to consider the historical seasonal high water levels as recorded by the vibratory wire piezometers.

Comment 26: *Were the ball field soils noted in Table 6-1 actually tested for vertical permeability or just estimated?*

Response 26: The vertical hydraulic conductivity for the soils beneath the ball field that were shown in Table 6-1 was assumed based on a visual classification of soils encountered in test pits that were advanced within the area of the ball field. These soils were generally classified as silty sands. Permeability testing of ball field soils was not performed as part of this project.

Comment 27: *In Table 6-4 for the EML Scenario, when was the measured pressure head taken? If it was not measured, why is it labeled as such? What does it represent?*

Response 27: The labeling of pore pressure head values in the third column of Table 6-4 as "measured" was made to differentiate these values from the "computed" values presented in column 2. These "measured" values for the vibrating wire piezometers are the maximum of measured values (after stabilization of installation pore pressures) recorded between March and June of 1998. The "measured" values for the 1998 standpipe piezometers are extrapolated values, increased from measured 23 October 1998 values for use as part of the "EML scenario." This set of measured and extrapolated pore pressure head values represent seasonal high water levels in the dam and were used to develop a grid of pore pressures for input into stability analyses.

Comment 28: *In Boring G-1B, at a depth of 95 feet, the drilling log notes zero blow count material and a resulting drop by the weight of the rods. It was not mentioned in the report. Why not? Is it significant?*

Response 28: The zero blow count referred to in Comment 28 was reported for the first of four 6-in. long penetration intervals beginning at 95 ft of depth. GeoSyntec notes that the blow counts for succeeding 6-in. long penetration intervals of the SPT test referenced in Comment 28 are 12, 11, and 15. In practice, the first blow count of a standard penetration test (SPT) is considered suspect and not necessarily representative of undisturbed in situ soil. The significantly higher blow counts after the initial 6-inch long penetration indicate that the zero blow count reading is due to local disturbance and does not reflect an undisturbed soil response. A review of all boring logs advanced as part of the 1998 field investigation indicate no other zero blow counts. GeoSyntec does not consider the zero blow count recorded within Boring G-18 as significant with respect to stability of the dam.

Comment 29: *Law's October 21, 1998 letter notes that they were unable to lower the geophones below 103 feet and the hole had water at 27 feet. Does this affect the test results?*

Response 29: The presence of water in the test casing does not adversely affect seismic down-hole test measurements used to obtain shear and compression wave velocities. The inability of Law to lower the geophones below 103 ft is reflected in the maximum reported test depth interval of 98 to 103 ft.

Comment 30: *Table 1 at G-1A has Poisson's values that range from –1.5 to 0.47. According to Law's Report, these values imply materials from dissimilar fill (rock/soil mixture) to saturated unconsolidated soils. Similarly, Table 2 has a number of values at 0.45 to 0.48, which indicates saturated, unconsolidated soils. Is this accounted for in the stability analyses? If so, how?*

Response 30: GeoSyntec notes that the values of Poisson's ratio reported in Table 1 range from –1.05 to 0.47, with the negative and very low values of Poisson's ratio likely due to the presence of gravel (i.e., rock) within the dam fill. However, GeoSyntec wishes to clarify that the positive range of Poisson's ratio indicates the presence of unsaturated to saturated unconsolidated *material* (i.e., soil, not rock) and not "unconsolidated *soil*" as stated in the Law report.

With regards to stability analyses, the presence of gravel within the dam fill was directly evident from drilling and sampling activities performed as part of the 1998 investigation. Laboratory strength testing was conducted using undisturbed samples collected during the 1998 investigation and water levels monitored using piezometers installed in the dam fill. Therefore, GeoSyntec believes that the effect of this gravel was accounted for in stability analyses.

Comment 31: *When the hammer blows were delivered to the metal plate and the wooden beam, were they delivered by hand or by a mechanical method? Why is this test valid and what does it really mean?*

Response 31: In response to the question regarding the means by which hammer blows were delivered, these blows were delivered manually by striking the metal plate/wooden beam with a sledge hammer.

In response to the first part of the second question in Comment 31 regarding the validity of the test, the seismic down-hole test is a widely accepted method for obtaining shear and compression wave velocities of subsurface materials [Sabatini et al., in press; Kavazanjian et al., 1997; Arman et al., 1997; and Kramer, 1996]. An advantage of this test method over laboratory test methods is that the results represent *in-place* materials and can be performed at depth intervals to reflect the characteristics of the entire soil column at each boring location.

In response to the question regarding the meaning of the test. In the context of the 1998 report, the seismic down-hole testing was originally envisioned to be used to develop a site-specific stiffness profile for use in site-specific site response analyses for the seismic stability assessment. These analyses were anticipated based on the relatively low factors of safety previously calculated for the dam by another consultant. After, the results of the simplified seismic evaluation (i.e., pseudo-static stability analyses) described in the 1998 report indicated that seismic loading would not control the design. Site response analyses were not necessary, and thus were not performed.

Comment 32: *It appears that only one triaxial test was done per tube sample. Typically, our office has received three tests per tube to confirm that the results are consistent.*

Response 32: GeoSyntec recognizes the benefits of testing multiple specimens from a single Shelby tube to evaluate variability of the sample within the tube. However, GeoSyntec's approach to characterizing the shear strength of the dam earth fill included obtaining a large number of tube samples (a total of 27) from a broad range of elevations and at regular intervals of depth for all borings. Considering the relatively large number of sample tubes obtained and the greater likelihood of variability between tubes as compared to within an individual tube, GeoSyntec selected a single specimen from each of 15 tubes. The results of triaxial tests performed for the specimens indicate relatively uniform shear strengths within the dam fill. For example, a range of between 34° and 37° was calculated for effective stress friction angles for ultimate strength conditions. GeoSyntec notes that shear strengths selected for use in stability analyses represented minimum measured values (or less) for the tested specimens.

Comment 33: *How is the pheratic surface traced in the stability cross sections? A trace of the phreatic surface should be shown on the cross section.*

Response 33: Pore pressures were input to the slope stability analyses as a grid of 740 pore pressure values and not as a single phreatic surface. These pore pressure values were calculated for the "EML scenario" as part of seepage analyses and were presented as part of the XSTABL program output files in Appendix E of the 1998 report. The program XSTABL does not plot a phreatic surface on graphical output files where a pore pressure grid is utilized, such as shown in Figures 7-2 to 7-4 of the report. However, GeoSyntec has added a line representing the phreatic surface (i.e., an equipotential line of zero pressure) to these figures and provided them in Attachment R33.

Comment 34: *Are the pore pressures estimated by the W/Seep model appropriate? There was no hydraulic conductivity testing done in the lab or in the field with the insitu soils?*

Response 34: The SEEP/W analyses performed for the "23 October 1998 scenario" used pore pressures measured at 18 different locations within the dam. After assuming a reasonable baseline value for the vertical hydraulic conductivity of the dam shell, other parameters included in the seepage analyses were adjusted until the best-fit pore water pressure distribution within the dam to the 23 October 1998 measured pore pressures was calculated. For example, if the calculated pore pressures from the SEEP/W analyses were much lower than the pore pressures measured on 23 October 1998 on the downstream side of the dam, it would indicate that the hydraulic conductivity of the core used in the SEEP/W analyses was too low. The value assumed for this parameter was then modified accordingly. Thus, these analyses used *measured in situ* data to calculate a mass hydraulic conductivity of the dam. GeoSyntec believes that this approach is technically more accurate than a laboratory permeability test. Moreover, the pore pressures used in the stability analyses (corresponding to the "EML scenario") are considered to be conservative (i.e., on average the measured seasonal high pore pressures within the dam are lower than those used in the stability analyses).

Comment 35: *What happens if the earthquake event has a long duration and the dam does dissipate the pore water pressures? Is the embankment still stable?*

Response 35: Consistent with conclusions presented in the previously presented 1999 white paper, GeoSyntec considers it is appropriate to assume that negative pore pressures which would develop in the dam fill during an earthquake event persist for the duration of strong shaking associated with the design event. The design earthquake event was defined in the 1999 white paper as a magnitude 6.0 event, with an acceleration of 0.185 g and a duration of strong shaking of 8 seconds. GeoSyntec considers this duration to be too short for significant pore water dissipation to occur from within the dam fill, or even within a relatively higher permeability fill. If this duration were doubled this duration would still be too short for significant pore water dissipation to occur within the dam fill. GeoSyntec refers GaSDP to Section 4 of the 1999 white paper for more detailed discussion of this issue.

As additional supporting information to the 1999 white paper, GeoSyntec wishes to refer GaSDP to a laboratory study presented in Matsui et. al. [1980]. As part of this study soil specimens in varying degrees of overconsolidation were subjected to repeated cycles of loading to simulates the effect of earthquake loadings. The results of these tests showed that negative pore pressures can be observed in overconsolidated specimens over substantial numbers of cycles, which for the referenced study corresponded to a duration of approximately 50 seconds.

Comment 36: *The estimated effective stress level (su) along the potential slip surfaces appears to vary from 1000 psf to 5400 psf. For Piedmont Residual Soils, this is not adequately conservative. The assumed strength envelope must be adjusted to intercept the vertical axis at the origin.*

Response 36: It appears that the first sentence of Comment 36 was intended to refer to "shear strength mobilized" along the potential slip surfaces rather than the "estimated effective stress level (su)", as the comment reads. Furthermore, since the effective stress strength parameters used for static steady-state seepage slope stability analyses included an effective cohesion of 0 psf and not 1,000 psf, GeoSyntec believes that the GaSDP may have intended to refer to the total stress shear strength parameters used for seismic slope stability analyses. Hence, GeoSyntec provides the following response to Comment 36 with the above understanding but requests from GaSDP clarification of the comment, if the intended meaning of the comment is other than that described above in this response.

In response to Comment 36 regarding the intersection of the strength envelope through the origin, GeoSyntec has rerun the seismic slope stability analyses with a revised strength envelope which incorporates an intercept of the vertical axis at 0 psf. The revised envelope is presented in Attachment R36 drawn on the lower half of the 1998 report Figure 3-2. The resulting calculated minimum factor of safety for the downstream embankment stability for seismic conditions with the EML porewater pressure scenario is 1.44. This represents a 0.02 reduction in the calculated minimum factor of safety of 1.46 reported in the 1998 report. This factor of safety also satisfies the minimum value of 1.1 required by the GaEPD Safe Dam Rules. The associated potential slip surface and output files from the XSTABL program are provided in Attachment R36. This approach is considered by GeoSyntec to be very conservative because of its observed tendency for the dam shell material to generate negative pore pressures during undrained loading.

GeoSyntec appreciates the opportunity to respond to each of the Safe Dams Program comments and hopes that these responses completely address any and all concerns regarding the evaluation of stability and recommendations for rehabilitation of Lake Petit Dam. Should Safe Dams Program have any further questions or require additional information, pleased do not hesitate to contact either of the undersigned.

Respectfully submitted,

Daniel G. Pass

Daniel G. Pass, P.E.
Project Engineer

Robert C Bachus

For R. Neil Davies, C.Eng., MICE, P.E.
Principal

Enclosure

Copy to: Troy Ledbetter, Big Canoe Property Owners Association

GG3012-02/GA020070

REFERENCES

Arman, A., Samtani, N., Castelli, R., and Munfakh, G., "*Geotechnical and Foundation Engineering Module 1 – Subsurface Investigations*", Report FHWA-H1-97-021, FHWA, Washington, D.C., March 1997.

Burger, H.R., *Exploration Geophysics of the Shallow Subsurface*, Prentice Hall PTR, Englewood Cliffs, N.J., 1992, 489 pages.

Bureau of Reclamation, *Design of Small Dams*, 3rd Ed, United States Department of Interior, Bureau of Reclamation, Washington, D.C., 1987, 860 pages.

Hatanaka, M. and Uchida, A., "Empirical Correlation Between Penetration Resistance and Internal Friction Angle of Sandy Soils," *Soils and Foundations*, Japanese Geotechnical Society, Vol. 36, No. 4, 1996, pp. 1-9.

Holtz, R.D., Kovacs, W.D., *An Introduction to Geotechnical Engineering*, Prentice Hall, Inc., Englewood Cliffs, N.J., 1981, 733 pp.

Kavazanjian, Jr., E., Matasovic, N., Hadj-Hamou, T., Sabatini, P.J., "*Geotechnical Earthquake Engineering for Highways-Design Principles*," Report FHWA-SA-97-076, Vol. 1, FHWA, Washington, D.C., May 1997, 186 p.

Kramer, S.L., *Geotechnical Earthquake Engineering*, Prentice Hall, Inc., Upper Saddle River, N.J., 1996, 653 pp.

Lambe, T.W., and Whitman, R.V., , John Wiley & Sons, New York, 1969.

Matsui, T., Ohara, H., and Ito, T., "Cyclic Stress-Strain History and Shear Characteristics of Clay", *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT10, Oct. 1980, pp. 1101-1120.

Mitchell, James K., *Fundamentals of Soil Behavior*, John Wiley & Sons, Inc., New York, 1993, 437 pages.

Sabatini, P.J., Bachus, R. C., Mayne, P.W., Schneider, J.A., and Zettler, T.E., "*Evaluation of Soil and Rock Properties*", FHWA, Washington, D.C., In Press.

Schmertmann, H.H., "Measurement of In-Situ Soil Strength," *Proceeding of ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Vol. 2, Raleigh, NC, 1975, pp. 57-138.

Sowers, G.F., "Residual Soil Settlement Related to the Weathering Profile", *Proceedings of Settlement '94*, ASCE Geotechnical Special Publication No. 40., College Station, Texas, 1994, pp. 1686-1702.

Sowers, G.F. and Richardson, Thomas L., "Residual Soils of the Piedmont and Blue Ridge", *Transportation Research Record, No. 919*, Transportation Research Bulletin, Washington, D.C., 1983.

Weigel, Robert C., *Earthquake Engineering*, Prentice-Hall, Inc., Englewood Cliffs, N.J, 1970.