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WHITE PAPER
SEISMIC STABILITY ANALYSIS

LAKE PETIT DAM
BIG CANOE, GEORGIA

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EXECUTIVE SUMMARY

This white paper provides information regarding the seismic stability analyses presented by GeoSyntec Consultants (GeoSyntec) in a December 1998 report entitled "*Evaluation of Stability and Rehabilitation Measures, Lake Petit Dam, Big Canoe, Georgia*" (December 1998 report). Specifically, this white paper demonstrates that these analyses comply with the governing regulations for seismic stability analyses of dams contained in the Georgia Safe Dams Program "*Rules for Dam Safety.*" The demonstration involves comparing the approach used in the December 1998 report to that given in engineering guidance documents prepared by the U.S. Army Corps of Engineers (USCOE), the U.S. Department of Interior, Bureau of Reclamation (USBR), and the Federal Energy Regulatory Commission (FERC). These agencies are specifically identified in the "*Rules for Dam Safety*" as acceptable design or evaluation references. In showing consistency with engineering guidance from these agencies, it is demonstrated that the approach used for seismic stability analyses in the December 1998 report conforms to accepted practice within the engineering profession and the dam safety industry.

This white paper focuses on the following key topics relating to seismic stability evaluation: (i) analysis method; (ii) seismic coefficient; and (iii) soil shear strength. For each topic, information on agency guidance is presented followed by a comparison to the approach used in the December 1998 report. The white paper provides the conclusions given below based on consideration of the site-specific conditions at the Lake Petit Dam and on the comparisons between agency guidance and the approach used in the December 1998 report.

- The pseudo-static stability analysis used in the December 1998 report to assess the potential for seismic slope deformation is consistent with USCOE, FERC, and USBR guidance. In fact, this approach is considered extremely conservative with respect to FERC and USBR, given that both these agencies' guidance indicate that seismic deformations should not be a problem for the Lake Petit Dam because of the relatively favorable site location and dam characteristics.

- The absence of a liquefaction potential analysis in the December 1998 report is consistent with USCOE, FERC, and USBR guidance. In accordance with the guidance provided by all three agencies, liquefaction analysis is not needed due to the fact that none of the soils at the site are susceptible to liquefaction under the expected seismic loading.

- The seismic coefficient used in the December 1998 report, 0.183 g, is very conservative with respect to "*Rules for Dam Safety*" criteria, agency guidance, and standard practice. In fact, a seismic coefficient of only 0.09 g to 0.10 g would be consistent with the criteria, guidance, and standard practice.

- The use of undrained shear strengths for pseudo-static slope stability analyses of the Lake Petit Dam embankment fill materials is appropriate and consistent with agency guidance. Furthermore, undrained shear strength parameters used in the December 1998 report are conservative with respect to the laboratory strength test measurements on intact samples of dam material and are therefore consistent with the "*Rules for Dam Safety*" directive for use of conservative soil shear strength parameters.

In conclusion, this white paper presents information that shows that the seismic stability analyses presented in the December 1998 GeoSyntec report entitled "*Evaluation of Stability and Rehabilitation Measures, Lake Petit Dam, Big Canoe, Georgia*" are consistent with USCOE, FERC, and USBR guidance for safety evaluation of earth dams. These analyses are also consistent with "*Rules for Dam Safety*" requirements regarding factors of safety, seismic acceleration levels, and conservative selection of soil shear strength parameters. Therefore, the seismic stability analyses in the December 1998 GeoSyntec report satisfy the "*Rules for Dam Safety*" requirement for conformance with accepted practices of the engineering profession and dam safety industry.

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1. INTRODUCTION

1.1 Terms of Reference

This "white paper" document was prepared by GeoSyntec Consultants (GeoSyntec), Atlanta, Georgia at the request of the Big Canoe Property Owners Association (POA), Big Canoe, Georgia. This white paper presents information regarding seismic stability analyses performed by GeoSyntec for the Lake Petit Dam, located within Big Canoe. This document was authored by Dr. Gary R. Schmertmann, P.E. and was reviewed by Dr. Edward Kavazanjian, P.E.

1.2 Purpose

The purpose of this white paper is to present additional information and background material regarding the seismic stability analyses performed by GeoSyntec for the December 1998 report entitled "*Evaluation of Stability and Rehabilitation Measures, Lake Petit Dam, Big Canoe, Georgia*" (December 1998 report). Specifically, this white paper demonstrates that the seismic stability analyses performed for the December 1998 report conform to accepted practice within the engineering profession and the dam safety industry. GeoSyntec understands that the Big Canoe POA intends to provide this white paper to the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (GaEPD Safe Dams) for their use in review of the December 1998 report.

1.3 Background

A slope stability evaluation of the Lake Petit Dam was included in Section 7 of the December 1998 report. The slope stability analyses performed for seismic loading conditions (seismic stability analyses) are the focus of this white paper. The December 1998 report presents a brief overview of the approach used to perform the seismic stability analyses but does not include extensive background information on the topic. One of the primary purposes of this white paper is to provide additional background information on the current state of practice for seismic stability analysis of dams and other earth structures.

GeoSyntec understands that the governing regulations for seismic stability analyses of the dam are contained in the GaEPD Safe Dams "*Rules for Dam Safety*" (Chapter 391-3-8 of the Rules of the Georgia Department of Natural Resources). The following relevant information is summarized from the "*Rules for Dam Safety.*"

- General guidance for engineering analyses is provided in Rule 398-3-8-.09(1), as follows: design and evaluation of dams "... *shall conform to accepted practices of the engineering profession and dam safety industry. Design manuals, evaluation guidelines, and procedures used by the following agencies can be considered as acceptable design or evaluation references...*

    - *U.S. Army Corps of Engineers*
    - *National Resource Conservation Service*
    - *U.S. Department of Interior, Bureau of Reclamation*
    - *Federal Energy Regulatory Commission*"

- More specific guidance relevant to seismic stability analyses is provided as indicated below.

    - Earth dams must be stable under various construction and/or operating conditions, including the condition of steady-state seepage with seismic loading. For this condition a minimum safety factor of 1.1 indicates acceptable stability. [Rule 398-3-8-.09(3)(a)]
    - All dams shall be capable of withstanding seismic accelerations with a 2 percent chance of exceedance in a 50-year period, as defined in a National Earthquake Hazard Reduction Program (NEHRP) seismic hazard map. [Rule 398-3-8-.09(3)(c)]
    - "*Conservative selections for soil strength values should be used for analyses or evaluations.*" [Rule 398-3-8-.09(3)(b)]

This white paper will reference a number of relevant engineering guidance documents from the agencies identified above. The major documents referred to herein are as follows:

- U.S. Army Corps of Engineers (USCOE), *Earthquake Design and Evaluation for Civil Works Projects*, Engineer Regulation ER 1110-2-1806, July 1995;

- USCOE, *Stability of Earth and Rock-Fill Dams*, Engineer Manual EM 1110-2-1902, April 1970;

- Federal Energy Regulatory Commission (FERC); *Engineering Guidelines for the Evaluation of Hydropower Projects*, FERC 0119-2, April 1991; and

- U.S. Department of Interior, Bureau of Reclamation (USBR), *Design Standards No. 13, Embankment Dams, Chapter 13: Seismic Design and Analysis*, December 1989.

1.4 Organization

The remainder of this white paper is organized as follows:

- information on the different types of seismic stability analyses typically performed for earth dams, and the specific type performed for the December 1998 report is presented in Section 2;

- information regarding the seismic acceleration coefficient used for seismic stability analyses is presented in Section 3;

- information regarding the soil shear strength values used for seismic stability analyses is presented in Section 4;

- conclusions as to whether the seismic stability analyses performed for the December 1998 report conform to accepted practices of the engineering profession and dam safety industry and satisfy the requirements of the "*Rules for Dam Safety*" are presented in Section 5; and

- a list of technical references is provided in Section 6.

2. OVERVIEW OF SEISMIC STABILITY EVALUATION

2.1 Introduction

This chapter discusses the seismic stability evaluation approaches used by the USCOE, FERC, and USBR and indicates how the approach adopted in the December 1998 report is consistent with these approaches. A discussion is also provided regarding a key issue in selection of the appropriate approach: the presence or absence of liquefiable soil materials at the site.

The terminology used by the agencies for seismic stability evaluation varies somewhat. The following two terms are generally used to refer to analysis performed where soils are not prone to liquefaction.

- Pseudo-static analysis is a type of slope stability analysis where the complex effects of earthquake shaking are represented by a single acceleration parameter, the seismic coefficient. Pseudo-static analysis, as generally performed, provides assessment of the relative degree of permanent earthquake-induced deformations, if any.

- Dynamic deformation analysis considers the response of the embankment to earthquake shaking and provides a prediction of the magnitude of permanent earthquake-induced deformation. The dynamic deformation procedures used in practice incorporate pseudo-static analysis as a component.

The following two terms are generally used to refer to analysis performed where soils are prone to liquefaction.

- Liquefaction analysis provides an assessment whether soils will liquefy, or undergo significant shear strength reduction, during earthquake shaking.

- Excessive deformation analysis, or post-liquefaction stability analysis, is a form of slope stability analysis performed when analysis indicates that liquefaction will occur. It provides an assessment of the relative degree of liquefaction-induced deformations, if any.

2.2 Agency Reference Documents

USCOE

Direction regarding the USCOE approach to seismic stability evaluation of dams is provided in USCOE [1995]. This document makes reference to the seismic zonation map presented in the Uniform Building Code (UBC) [International Conference of Building Officials; 1994] to establish the level of seismic analysis required at any specific project location. The UBC map, shown in Figure 1, indicates that the Lake Petit Dam site is in UBC seismic zone 2A. With respect to the UBC zonation scheme, the severity of potential seismic loading is smallest for UBC zone 0 and largest for UBC zone 4.

USCOE [1995, section 5h] describes two types of seismic studies as indicated below:

- standard studies –use values of ground motion from published seismic zone maps and involve simplified assessments of embankment deformation and soil liquefaction to determine if seismic loads control the design; such studies may be satisfactory for final evaluation of dam safety in UBC seismic zone 2A if seismic loads do not control the design; and
- site-specific studies – use the results of site-specific seismological and geotechnical investigations to assess earthquake motion propagation, soil liquefaction potential, slope stability, and slope deformation; such studies are required for projects in UBC seismic zones 3 and 4.

USCOE [1995, section 9d] also discusses analyses to assess susceptibility to liquefaction or excessive deformation for embankments. This discussion indicates that, while required for projects in UBC seismic zones 3 and 4, such analyses are not needed for projects in UBC seismic zone 2A unless the presence of materials susceptible to liquefaction or excessive deformation is suspected.

Overall, the USCOE guidance document [USCOE, 1995] suggests that, since the Lake Petit Dam is in UBC seismic zone 2A, standard studies may be satisfactory for final seismic evaluations as long as the results of the standard studies indicate that seismic loads do not control the design. In addition, the USCOE guidance document suggests that liquefaction and excessive deformation analyses are not needed at the Lake Petit Dam because, as indicated in Section 2.3 below, there is no indication that liquefiable soils are present.

FERC

Direction regarding seismic stability evaluation of earth dams is provided in FERC [1991, sections 4-6.6.5 and 4-7]. While FERC [1991] presents its own approach for seismic stability evaluation, as discussed below, it also directs the reader to several USCOE documents for additional guidance, most notably a previous (1983) version of USCOE [1995].

FERC [1991, sections 4-6.6.5 and 4-7.3a] makes reference to seismic zones 1, 2, 3, and 4 to establish the level of seismic analysis required at any specific project location. FERC [1991] does not, however, define the where these zones are located. Based on the fact that the FERC uses the same numbering of seismic zones (i.e., 1 to 4) employed in UBC seismic hazard maps, and that FERC references USCOE documents that employ UBC seismic hazard maps, GeoSyntec infers that the seismic zones referred to by FERC [1991] are the same as UBC seismic zones.

FERC [1991, section 4-7] states that the following three categories of seismic stability evaluation may be used:

- pseudo static methods;
- simplified procedures; and
- dynamic analyses of embankment stability and deformation.

FERC [1991, sections 4-7.1 and 4-7.3a] indicates that the pseudo-static method for stability analysis may be used to evaluate dams located in seismic zones 1 and 2 having well-compacted embankments and dense foundation soils. The restriction of the pseudo-static method to dams with well-compacted embankments and dense foundation soils is presumably to preclude the use of the method when soils susceptible to liquefaction or excessive deformation are present. FERC indicates that for projects in seismic zones 3 and 4, where the potential exists for more severe or frequent seismic loading, the pseudo-static method is not always appropriate and simplified procedures and/or dynamic analyses should be used.

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Overall, the FERC [1991] guidance document indicates that if the dam is not in seismic zones 3 or 4, and has well-compacted embankment soils and dense foundations soils, then pseudo-static stability analysis are adequate for seismic stability evaluation. GeoSyntec believes that these conditions are met at the Lake Petit Dam. Specifically, the 1994 UBC seismic hazard map (Figure 1) and previous UBC maps that GeoSyntec is aware of depict the dam site in seismic zone 2 or 2A, never in zone 3 or 4. In addition, with respect to soil liquefaction potential, the embankment is considered well compacted and the foundation soils dense (see Section 2.3 below). Therefore, the FERC [1991] guidance document suggests that pseudo-static stability analysis is adequate for seismic stability evaluation of the Lake Petit Dam.

As an additional consideration, FERC [1991, section 4-7.3d] states that "*deformations can be assumed not be a problem if the dam is well-built (densely compacted) and peak accelerations are 0.2g or less*." GeoSyntec believes that these conditions are met at the Lake Petit Dam. Specifically, with respect to soil liquefaction potential, the dam is considered well-built (see Section 2.3 below). In addition, the peak acceleration at the base of the dam is expected to be less than 0.2g based on the 0.183g design bedrock acceleration given by the "*Rules for Dam Safety*," as reported in the December 1998 report. Therefore, the FERC [1991] guidance document suggests that seismic deformations can be assumed not be a problem at the Lake Petit Dam.

USBR

Direction regarding seismic stability evaluation of earth dams is provided in USBR [1989]. The USBR approach considers dynamic deformation analysis, liquefaction analysis, and excessive deformation analysis. USBR [1989] does not directly mention pseudo-static analysis, although it is indirectly referred to as a component of dynamic deformation analysis.

With respect to dynamic deformation analyses, USBR [1989, section 13.5.1A] indicates that:

"*For a dam and foundation <u>not subject to liquefaction</u>, dynamic deformations should not be a problem and need not be analyzed for such if the following conditions are satisfied.*

1. *The dam is a well-built dam (densely compacted), and peak accelerations at the base of the dam are 0.2g (gravity) or less; or the dam is constructed of clay soils, is on clay or rock foundations, and peak accelerations or 0.35g or less;*

no so

2. *The slopes of the dam are 3:1 (H:V) or flatter;*
3. *The static factors of safety of the critical failure surfaces involving loss of crest elevation (i.e., other than the infinite slope case) are greater than 1.5 under loading conditions expected prior to an earthquake;*
4. *The freeboard at the time of the earthquake is a minimum of 2 to 3 percent of the embankment height (not less than 3 feet (0.9)) ...*"

USBR states that if the above conditions are not satisfied then a deformation analysis, including dynamic response calculations, should be made.

The above conditions are discussed below with respect to the Lake Petit Dam:

1. The dam is considered well-built, with respect to liquefaction potential, and the embankment and foundation materials are considered non-liquefiable. The characteristics of the dam embankment materials and foundation soils are discussed below in Section 2.3. The peak acceleration at the base of the dam is expected to be less than 0.2g based on the 0.183g design bedrock acceleration given by the "*Rules for Dam Safety*," as reported in the December 1998 report.

2. The average inclination of the downstream face of the dam (including benches) is between 2.9H:1V and 3.0H:1V, or essentially 3H:1V, based on the dam section geometry given in the December 1998 report.

3. The static factor of safety for potential slip surfaces extending to the dam crest is greater than 1.5 (prior to rehabilitation) based on the slope stability analyses in the December 1998 report.

4. The freeboard at the time of the earthquake, taken as the crest elevation (1648 ft) minus the normal pool elevation (1635 ft), is 13 ft. This value, equal to approximately 11 percent of the embankment height of 115 ft., significantly exceeds the minimum value of 2 to 3 percent.

Based on the above conditions, the USBR (1989) guidance document indicates that seismic deformations should not be a problem for the Lake Petit Dam and need not be analyzed.

USBR [1989, section13.8.3] further indicates that:

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"Based on current state-of-the-art, analysis of any structures located in Algermissen (1969) seismic zones 1 and 2 would not be expected to show intolerable deformations if full analyses were performed. For this reason, seismic analyses of these structures are not scheduled unless there exist strong indications of static stability problems or strong indications that low-density, cohesionless materials are present in critical embankment or foundation zones."

GeoSyntec believes that the indicated conditions under which seismic analysis is not required are met at the Lake Petit Dam site. Specifically, a map showing the Algermissen (1969) seismic zones referred to above (Figure 2) indicates that the Lake Petit Dam site is in zone 2. In addition, there are no indications of static stability problems at the dam, other than evidence of minor surficial instability in local areas. Finally, there are no indications that low-density, cohesionless materials are present at the site (see Section 2.3). Therefore, the USBR [1989] guidance document again suggests that seismic deformations should not be a problem at the Lake Petit Dam and need not be analyzed.

Summary

Guidance for seismic stability evaluations of earth dams from USCOE, FERC, and USBR is consistent in most aspects. All three agencies indicate that the evaluation must assess the potential for both: (i) permanent dynamic deformations of dam slopes resulting in loss of freeboard; and (ii) liquefaction of embankment or foundation soils resulting in mass instability and excessive dam deformations. The agencies further indicate that the required scope of the dynamic deformation and liquefaction assessments at any particular dam depends on the potential severity and frequency of seismic loading at the site.

All three agencies indicate that, in seismic zones not anticipated to be exposed to severe and frequent seismic loading, the dynamic deformation and liquefaction assessments may be performed using only simple analyses, or in some cases no analyses at all. This direction applies to north Georgia, including the Lake Petit Dam site, due to its location (Figures 1 and 2). Further, due of the lack of potentially liquefiable soil materials at the site (Section 2.3), guidance from FERC and USBR suggests that it can be safely assumed that deformation and liquefaction will not be a problem at the dam and that no dynamic deformation and liquefaction analyses are needed.

2.3 Potential for Liquefaction of Embankment and Foundation Soils

Introduction

The embankment and foundation soils at the Lake Petit Dam are considered non-liquefiable and not susceptible to excessive deformations under likely imposed seismic loading based on the characteristics described below. Reference is made to laboratory and field tests conducted by GeoSyntec on intact samples of dam embankment material (Chapter 3 of December 1998 report). A summary of the laboratory testing results from the December 1998 report is provided as Figure 3.

Shear Wave Velocity

Down-hole shear wave velocity measurements were made in the field over 5-ft depth intervals at two boring locations. The measurements in the deeper boring were made to a maximum depth of 100 ft. The shear wave velocity measurements are considered to be particularly representative of in-situ conditions because they reflect the characteristics of the entire soil column at each boring location and because they do not involve testing of potentially non-representative or disturbed samples. The measured shear wave velocities were presented in Section 2.3.3 of the December 1998 report and are summarized as follows:

- a total of 23 individual shear wave velocity, v_s, measurements were made in the embankment shell materials;

- v_s values for the 23 measurements ranged from approximately 450 to 1850 ft/sec (fps), with an average of 1120 fps;

- all except three of the 23 v_s measurements are greater than 800 fps; the three measurements less than 800 fps were made at shallow depths (0 to 15 ft.) in unsaturated soils near the dam crest; and

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13 < 1200 fps

- ten of the 23 v_s measurements exceeded 1200 fps.

USBR [1989, sections 13.5.2G.2 and 13.5.3C] guidance regarding shear wave velocity and soil liquefaction resistance is based on a 1986 lecture by K.H. Stokoe of the University of Texas. The USBR [1989] guidance indicates that materials with v_s between 800 and 1200 fps can be considered likely to be non-liquefiable but supporting evidence should be obtained to rule out liquefaction, and that materials with v_s greater than 1200 fps can be considered non-liquefiable. A recent publication by Stokoe on the same subject [Andrus and Stokoe, 1998] indicates that all soils with v_s greater than approximately 600 fps can be considered non-liquefiable for earthquakes up to magnitude 7.5. comp to age peak

With respect to the v_s measurements on the embankment shell materials, the USBR [1989] guidance indicates that ten of the 23 measurements represent non-liquefiable soils (i.e., $v_s > 1200$ fps). An additional ten measurements indicate likely non-liquefiable soils (i.e., $800 < v_s < 1200$ fps); and for these soils evidence is presented below regarding observed dilative behavior, relative compaction, and standard penetration test (SPT) blow counts, to support that they are non-liquefiable. The recent publication by Andrus and Stokoe [1998] also indicates that these soils should be considered non-liquefiable as they have a v_s greater than 600 fps. For the three v_s measurements below 800 fps, all were made at shallow depths in soils that are non-liquefiable because they are not saturated. Therefore, based on the USBR [1989] guidance document, and a recent publication by a USBR-recognized expert, the embankment shell materials at the Lake Petit Dam are considered non-liquefiable.

Dilation in Laboratory Strength Tests on Intact Samples

Intact samples of the embankment soils exhibited generally dilative behavior when subjected to isotropically consolidated, undrained triaxial shear loading (ICU TX loading). Specifically, for 11 out of 16 ICU TX test specimens, negative excess pore

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Dilation in Laboratory Strength Tests on Intact Samples

Intact samples of the embankment soils exhibited generally dilative behavior when subjected to isotropically consolidated, undrained triaxial shear loading (ICU TX loading). Specifically, for 11 out of 16 ICU TX test specimens, negative excess pore water pressures (i.e., suction pressures) were measured in the specimens. A negative excess pore pressure indicates that specimen tended to dilate (i.e., expand) in response to loading. The magnitude of the measured negative pore pressures, expressed as the ratio of excess pore water pressure to total applied axial stress at peak loading, A_f , averaged –0.09. The magnitude of the positive excess pore pressures measured in the remaining 5 ICU TX test specimens yielded an average A_f of +0.05. Detailed information on the testing results is provided in chapter 3 of the December 1998 report. The dilative behavior exhibited by the test specimens indicates that the intact embankment fill material is generally in a relatively dense condition and, as discussed in USBR [1989, section 13.5.2D], is not prone to liquefaction.

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Relative Compaction

The level of compaction achieved for the embankment fill soils during construction is a major factor affecting the susceptibility of these soils to liquefaction. The design drawings for the dam include a specification table that indicates that the required relative compaction for construction was 100 percent, relative to standard Proctor maximum density [Baldwin and Cranston Associates; 1971]. No construction records for the dam are known to exist from which to assess the level of compaction that was actually achieved. Alternatively, the density of intact samples can be measured and compared to maximum dry densities achieved in compaction tests to assess the level of compaction.

Density measurements of intact samples of the embankment shell soils produced dry densities ranging from 97 to 108 lb/ft^3, with an average value of 103 lb/ft^3. As no compaction testing was performed for the current laboratory study, compaction test results presented in borrow source studies performed prior to dam construction [Coleman; 1971a,b] were used to assess relative compaction. The Coleman studies present three standard Proctor compaction test results for embankment shell materials, indicating maximum dry densities ranging from 105 to 114 lb/ft^3, with an average value of 108 lb/ft3. Based on these average density values, the intact samples have a relative compaction of approximately 95 percent, relative to standard Proctor test results. This level of compaction, although approximately estimated, is consistent with the observed dilative character of the embankment fill and indicates that the fill is in a relatively dense condition.

SPT Blow Counts

Standard penetration test (SPT) blow counts, also referred to as N-values, were obtained by GeoSyntec at five boring locations, predominantly in the embankment shell material. SPT blow counts are widely used to provide an indication of the density and strength of soil materials. However, for sites where soils contain a significant amount of gravel-size particles, as is the case for the embankment shell materials at the Lake Petit Dam, SPT blow counts may be artificially high due to gravel particles interfering with penetration of the SPT split-spoon sampler. The measured SPT blow counts were presented in Section 2.3.2 of the December 1998 report and are summarized as follows:

- a total of approximately 60 blow count measurements were made in the embankment shell materials; only 50 of these measurements were considered potentially representative as ten of the higher value were discarded due to suspected gravel interference with the sampler; and

- the 50 potentially representative blow count measurements ranged from N= 9 to 57, with only six being less than N= 15, and the majority being in the range of N= 15 to 26 (uncorrected N-values).

These blow counts values indicate that the embankment shell material is generally well-compacted. This indication is consistent with the information presented above regarding shear wave velocity, dilative behavior, and relative compaction. Due, however, to the unknown degree to which gravel particles have caused elevated blow counts, the blow count information is considered of secondary significance compared to the other information presented above.

Foundation Soils

The foundation soil at the Lake Petit Dam was penetrated in two borings advanced by GeoSyntec. In both borings the soil was found to be dense, saprolitic material with high SPT blow counts (greater than 50). Based on these characteristics, the foundation soils are considered non-liquefiable.

Design Earthquake Magnitude and Duration

The magnitude and duration of the design earthquake is a consideration when assessing soil liquefaction potential. The "*Rules for Dam Safety*" provide direction on the peak bedrock acceleration associated with the design earthquake (i.e. the NEHRP seismic hazard map), but do not provide direction on magnitude or duration. GeoSyntec has assessed the magnitude of the design earthquake using the deaggregated seismic hazard information that accompanies the NEHRP map.

The deaggregated seismic hazard information provides an indication of the magnitude and epicentral distance of potential earthquakes that contribute to the overall seismic hazard at a limited number of cities across the United States. For the Lake Petit Dam site, the deaggregated seismic hazard information for Knoxville, Tennessee is most relevant because Knoxville is the closest city that lies in the same seismic source zone as the site. The deaggregated seismic hazard information for Knoxville indicates that almost two-thirds of the overall seismic hazard, with respect to peak ground acceleration, is contributed by magnitude 6 and smaller earthquakes, primarily occurring within an epicentral distance 25 kilometers. Therefore, an appropriate design earthquake for the Lake Petit Dam site is a magnitude 6 event occurring within 25 kilometers of the site. For this design earthquake, GeoSyntec estimates the duration of strong shaking to be 8 seconds, based on the Abrahamson and Silva duration model [1996, 1997]. In comparison, the design earthquake recently used for seismic analysis of the Tennessee Valley Authority (TVA) Tellico Dam, located 23 miles southwest of Knoxville, was a magnitude 5.8 event [Anderson et al.; 1996].

50 – 100 miles over away

2.4 Approach Used for December 1998 Report

The December 1998 report did not include any soil liquefaction analyses or assessment of excess deformation due to cyclic earthquake loading. This approach is consistent with agency guidance for sites where soils are non-liquefiable under the expected seismic loading. The Lake Petit Dam site meets these soil conditions (Section 2.3).

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With respect to dynamic deformation analyses, FERC and USBR guidance suggests that no dynamic deformation analyses are needed at the dam. In the case of the USBR guidance, it appears clear that even pseudo-static analyses are not needed.

However, USCOE guidance and the "*Rules for Dam Safety*" indicate that simple analyses should be performed. Therefore, consistent with USCOE guidance and the "*Rules for Dam Safety,*" the December 1998 report included pseudo-static slope stability analyses. Significant details of the pseudo-static analyses in the December 1998 report are discussed in the following chapters of this white paper.

It is noted that the December 1998 report indicates, in Section 7.2, that GeoSyntec originally anticipated that seismic loading would control the design of the dam rehabilitation. GeoSyntec therefore originally planned to perform a detailed seismic displacement analysis for the dam. However, after initial analyses were performed, it became apparent that seismic loading would not control the design. This finding is not surprising considering the fact, as presented in Section 2.2, that FERC and USBR guidance suggests that seismic analysis is not required for the dam. The December 1998 report was completed using a simple seismic stability approach, i.e., pseudo-static stability analyses. As also indicated in the December 1998 report, detailed seismic displacement analyses are planned to be performed to complement the pseudo-static analyses at the time that a final dam rehabilitation report is developed.

3. SEISMIC COEFFICIENT

3.1 Introduction

This chapter provides information on the seismic coefficient used in pseudo-static stability analyses. The seismic coefficient is a key factor affecting analysis results. Available guidance from USCOE, FERC, and USBR on selection of seismic coefficient is presented herein to justify the value of seismic coefficient used in the December 1998 report.

3.2 Agency Reference Documents

USCOE

USCOE [1995] does not provide any specific guidance on selection of the seismic coefficient for dams. It does, however, reference a separate USCOE publication [Hynes-Griffin and Franklin; 1984] that provides specific guidance on the selection of seismic coefficient for evaluating the potential for earthquake-induced instability. The Hynes-Griffin and Franklin procedure should not be used where great earthquakes (magnitude 8 or larger) are possible, where soil materials susceptible to liquefaction are present, or where available reservoir free board is small. Therefore, the procedure is applicable to Lake Petit Dam because, as indicated in Section 2.4, the dam does not fit any of these categories. The Hynes-Griffin and Franklin procedure calls for pseudo-static slope stability analysis using a seismic coefficient equal to one-half of the predicted peak bedrock acceleration. This recommendation reflects the fact that appropriate seismic coefficients are typically significantly smaller than peak bedrock accelerations. If the minimum calculated factor of safety from this analysis exceeds 1.0 then the dam is judged to be "*clearly safe against earthquake-induced sliding failure*." The Hynes-Griffin and Franklin procedure also includes recommendations for soil shear strength parameters to be used in the pseudo-static analyses, as discussed in the next chapter of this white paper.

USCOE [1970, p. 19] recommends that traditional (i.e., pseudo-static) stability analyses be conducted. A general guide to selection of seismic coefficient is provided in a map (Figure 4). The map indicates that the seismic coefficient may range from 0 to

0.15g in the United States, with a value of 0.10g assigned at the Lake Petit Dam site. USCOE [1970, p. 25] indicates that the minimum acceptable factor of safety using these seismic coefficients is 1.0. The fact that the maximum recommended seismic coefficient for the U.S. is 0.15g, a value much smaller than possible peak bedrock acceleration in high-activity seismic zones in the U.S., indicates that appropriate seismic coefficients are typically significantly smaller than peak bedrock accelerations.

FERC

FERC [1991, sections 4-6.6.5 and 4-7.3a] indicates that the seismic coefficient for pseudo-static stability analysis should be at least as large as that given for each seismic zones in specific seismic hazard maps. GeoSyntec has not, however, been able to obtain the referenced maps. The map referenced in section 4-6.6.5 is apparently a 1982 version of USCOE [1970]. The list of publications on the USCOE internet site indicates that the 1970 version of the document is the most current and does not include any 1982 version. The reference in section 4-7.3a refers to a 1983 version of USCOE [1995] that is no longer available.

USBR

USBR [1989] does not provide any specific guidance on selection of the seismic coefficient for dams.

Summary

Specific agency guidance on selection of the seismic coefficient for pseudo-static slope stability analysis is available from USCOE. USCOE [1970, p. 19] indicates a range of values from 0 to 0.15g for the United States. It is noted that this approach is consistent with standard pseudo-static analysis as described by H.B. Seed in his 1979 Rankine lecture [Seed, 1979, p. 220]. The seismic coefficient from USCOE [1970] for the Lake Petit Dam site is 0.10g.

Another USCOE publication, Hynes-Griffin and Franklin [1984], recommends a seismic coefficient equal to one-half of the predicted peak bedrock acceleration. For the Lake Petit Dam site, with a peak bedrock acceleration of 0.183g based on the "*Rules for Dam Safety*" criteria, this corresponds to a seismic coefficient of approximately 0.09g.

Both the recommended seismic coefficients from USCOE [1970] and Hynes-Griffin and Franklin [1984] are to be used with a minimum acceptable factor of safety of 1.0. not by our rules

3.3 Approach Used for December 1998 Report

The December 1998 report used a seismic coefficient of 0.183 g for pseudo-static slope stability analysis. This value is equal to the peak bedrock acceleration criteria in the "*Rules for Dam Safety*" and is approximately twice as large as the USCOE guidance values of 0.09 to 0.10g. The use of a value of seismic coefficient of 0.183g is therefore quite conservative with respect to both "*Rules for Dam Safety*" criteria and agency guidance.

quote rules

The December 1998 report used a minimum acceptable factor of safety from the pseudo-static analyses of 1.1. This value is specifically required by the "*Rules for Dam Safety*." This value is more conservative than the minimum acceptable value of 1.0 recommended in agency guidance.

4. SOIL SHEAR STRENGTH

4.1 Introduction

This chapter provides information on selection of soil shear strength parameters for pseudo-static slope stability analyses. The information focuses on the use of undrained shear strength parameters versus drained shear strength parameters. Shear strength characterizations recommended by USCOE, FERC, and USBR are presented along with the characterization used in the December 1998 report. Discussion is also provided regarding a key issue in shear-strength characterization – the likelihood that negative excess pore pressures induced by earthquake deformation will dissipate before the earthquake motion ceases.

Both undrained and drained shear strength parameters were measured in laboratory tests conducted by GeoSyntec on intact samples of dam embankment material (Chapter 3 of December 1998 report). A summary of the laboratory testing results is included for reference as Figure 4.

The information provided in this chapter does not address the representativeness and quality of the intact samples tested in the laboratory by GeoSyntec. The December 1998 report addresses these issues and concludes, in Section 5.2.1, that "*The testing results therefore form an appropriate basis for evaluation of shear strength parameters for use in slope stability analyses*."

4.2 Agency Reference Documents

USCOE

USCOE [1970, p. 25] indicates that seismic stability analyses for steady-state seepage conditions should be performed using shear strength parameters that reflect a two-part strength envelope, as illustrated in Figure 5. The two-part strength envelope is formed using the S test envelope (consolidated, drained shear) at low confining pressures and the average of the S test envelope and R test envelope (consolidated, undrained shear) at higher confining pressures. This envelope is also used for static stability analysis. USCOE [1970] appears to indicate that strength envelope for seismic stability analysis should be formed as shown in Figure 5 for all soil types, regardless of their relative permeability.

USCOE [1995] does not provide any specific guidance on soil shear strength parameters for pseudo-static stability design. USCOE [1995] does, however, reference a separate USCOE publication [Hynes-Griffin and Franklin; 1984] that provides guidance. Hynes-Griffin and Franklin [1984, p. 7] recommend that a two-part shear strength envelope, similar to that shown in Figure 5, be used for pseudo-static analysis of pervious soils. With respect to less pervious soils, they state that "*For soils of low permeability, in which undrained conditions are more likely to exist during an earthquake, an undrained, R, strength envelope would be appropriate*." Hynes-Griffin and Franklin [1984] do not provide a quantitative distinction between "pervious" soils and "low permeability" soils. Hynes-Griffin and Franklin [1984] also recommend the use of 80 percent of the soil strength measured in monotonic (i.e., non-cyclic) S tests and R tests to define strength envelopes to account for possibility of strength reduction that may potentially develop in cyclic tests.

The USCOE practice of using the undrained shear strength envelopes in pseudo-static stability analyses is illustrated in a recent USCOE publication [Krinitzsky et al.; 1998]. This publication presents pseudo-static slope stability calculations performed as part of seismic displacement analyses for a powerhouse structure and associated embankment dam on the Cooper River, approximately 30 miles north of Charleston, South Carolina. The design earthquake magnitude was 7.5. Two SM fill materials, referred to as zone I and zone II fill, were included in the calculations. For both materials, undrained shear strength parameters were used in the pseudo-static stability analyses, as illustrated in a table from the publication presented as Figure 6.

Although USCOE [1970] appears to indicate that the strength envelope for pseudo-static stability analysis should be formed using both S test and R test envelopes (Figure 5) for all soil types, regardless of their relative permeability, the more recent USCOE publications strongly indicate that R test (consolidated, undrained) envelopes are more appropriate for low permeability soils. As the Lake Petit dam soils are SM soils with relatively low permeability, as discussed further below, USCOE guidance suggests that undrained strength envelopes are appropriate for these soils.

FERC

FERC [1991] does not provide any specific guidance on soil shear strength parameters for pseudo-static stability design. The only discussion of the issue located by GeoSyntec in the document was a mention (section 4-7.3c) that "*minimum strength values corresponding to the degree to which pore water pressures are generated in the soils by the earthquake*" can be used. This discussion appears to be related to pore pressure build up leading to liquefaction and is not directly relevant to conditions at Lake Petit Dam due to the lack of potentially liquefiable soils at the site.

USBR

USBR [1989, section 13.5.1D] provides the guideline given below for soil shear strength for compacted fills for dynamic deformation analysis. As noted in Section 2.1, pseudo-static stability analysis is an essential component of dynamic deformation analysis.

> *In general, if the fill is clay, sandy clay , or some mixture of clay, sand, gravel, etc., compacted to 100 percent of standard Proctor density or greater, no reduction in shearing resistance should be assumed. Drained (effective) shear strengths may be used because insufficient pore pressure buildup requiring dissipation is anticipated. For 95 to 100 percent of standard Proctor density in the fill, a 5- to 10-percent loss in shearing resistance modeled as a pore pressure increase or as reduced undrained strengths would be appropriate.*

This guideline appears to suggest that use of undrained strength for a compacted clayey fill may be appropriate, and that a small strength reduction should be applied to account for cyclic loading effects. As discussed below, the embankment shell material at the Lake Petit Dam contains a small amount of clay, along with a significant amount of silt, and is considered to be of sufficiently low permeability that use of undrained strengths are appropriate.

Other

A specific comment regarding drained versus undrained strengths for pseudo-static stability analysis is presented in a classic paper by a noted geotechnical engineer, H.B. Seed, on the failure of the Sheffield dam [Seed et al., 1969]. In a section of the paper titled "*Strength Values for Soil Determined by Drained Tests or Consolidated-Drained Tests,*" Seed et al. make the following statement: "*For many dams, there is virtually no possibility of drainage during the application of disturbing forces due to earthquakes and the soil is therefore loaded under essentially undrained conditions. Nevertheless, some engineers use drained strengths for analysis even under conditions where no drainage could conceivably occur. Where drainage can readily occur, even in the short loading period of an earthquake, the use of drained strengths is clearly appropriate.*" This statement clearly indicates that use of undrained strength is appropriate for many dams. As discussed below, GeoSyntec believes that the Lake Petit Dam is such as case.

Summary

Agency guidance is not entirely consistent on the selection of soil shear strength parameters for pseudo-static stability analyses. However, the following concept surfaces consistently in the agency documents: undrained strengths are appropriate for conditions where the soil can not dissipate pore pressures during the short period of earthquake shaking; and that such a condition would almost certainly be the case for low-permeability soil fills. As discussed in Section 4.4 below, GeoSyntec believes that the embankment soils at the Lake Petit Dam are of sufficiently low permeability that use of undrained strengths is clearly appropriate for pseudo-static stability analyses. As a general comment, it is noted that agency guidance clearly mandates the use of undrained strengths for liquefiable soils. The reason for this mandate is that even the relatively high permeability, cohesionless soils that are susceptible to liquefaction can not rapidly dissipate excess pore pressures.

Another concept that appears in the agency guidance relates to undrained strengths for cohesive fill materials for use in pseudo-static stability analyses. Undrained strengths, if measured in monotonic (non-cyclic) loading tests, should be reduced by 5 to 20 percent to account for potential strength reduction during cyclic loading. For the soils at Lake Petit Dam, however, GeoSyntec believes that the characteristics presented in Section 2.3 indicate that they are not susceptible to significant strength reduction during cyclic loading. why?

4.4 Dissipation of Pore Pressure During Earthquake

As discussed above, agency guidance indicates that the appropriateness of use of undrained shear strengths for pseudo-static stability analysis largely depends on the relative permeability of the soil. This is because undrained strengths are appropriate when soils cannot dissipate pore pressure during the short period of the earthquake. Agency guidance does not, however, provide definitive permeability criteria on this topic.

To assess the appropriateness of the use of undrained strengths for pseudo-static stability analysis of the embankment fill soils at the Lake Petit Dam, the grain size characteristics of these soils are considered. Grain size characteristics are considered because no direct measurements of permeability have been performed. The grain size characteristics of intact samples of the embankment fill soils at the Lake Petit Dam were measured and reported in the December 1998 report. A summary of the laboratory testing results is presented in Figure 3.

The testing summary indicates that the embankment shell material classifies as an SM material (silty sand) and contains a significant amount of fine-grained particles (23 to 41 percent silt and 2 to 7 percent clay). The testing summary also indicates that the embankment core material classifies as a ML material (low-plasticity silt) and contains more fine-grained particles than the shell material (35 to 42 percent silt and 10 to 18 percent clay). Because both materials have a significant percentage of fine-grained particles (at least 30 percent) and have been compacted to a relatively dense state (Section 2.3), GeoSyntec considers them to be low permeability materials. This interpretation is consistent with a well-known geotechnical engineering text book [Sowers; 1979] which indicates that silty sands are considered low permeability materials (Figure 7).

Another factor in considering the ability of the soils to dissipate pore pressure during the earthquake is the anticipated duration of the design earthquake event. As indicated in Section 2.3, the design earthquake is a relatively small magnitude event (magnitude 6), occurring close to the site, with an expected duration of strong shaking of eight seconds. This period is believed to be too short for significant pore water dissipation to occur from within the dam, even in high permeability soils.

In summary, based on the low permeability character of the embankment fill soils and the expected short duration of the design earthquake, it is appropriate to assume that pore water pressure dissipation will not occur during the design earthquake. Therefore, it is consistent with agency guidance to use undrained shear strengths in the pseudo-static stability analyses. This conclusion is also consistent with the analyses presented in the recent USCOE publication discussed above (i.e., Krinitzsky et al., 1998) where undrained shear strengths were used for pseudo-static stability analyses of a compacted SM material for a relatively long duration (magnitude 7.5) earthquake.

4.5 Approach Used for December 1998 Report

The December 1998 report used an undrained strength envelope for the pseudo-static analysis of the embankment fill soils. The development of the undrained strength envelope is described in detail in section 5.2.1 of the December 1998 report. The undrained strength envelope provides a conservative selection of shear strength parameters because it is based on a lower bound characterization of the measured undrained strength values.

Why not reduced?

The use of undrained strengths, as opposed to drained strengths, for the pseudo-static analysis of the embankment fill soils is consistent with agency guidance. This is based on the low permeability character of the embankment fill soils and the expected short duration of the design earthquake.

5. CONCLUSIONS

5.1 Seismic Stability Analysis Approach

The simple approach used in the December 1998 report to assess the potential for seismic slope deformation, i.e., pseudo-static stability analysis, is consistent with USCOE, FERC, and USBR guidance. In fact, this approach is considered extremely conservative with respect to FERC and USBR guidance, given that both these agencies' guidance indicate that seismic deformations should not be a problem for the Lake Petit Dam because of the relatively favorable site location and dam characteristics. USBR guidance is particularly clear, suggesting that no seismic analyses at all are needed.

The absence of a liquefaction potential analysis in the December 1998 report is consistent with USCOE, FERC, and USBR guidance. In accordance with the guidance provided by all three agencies, liquefaction analysis is not needed due to the fact that none of the soils at the site are susceptible to liquefaction under the expected seismic loading.

5.2 Seismic Coefficient

The seismic coefficient used in the December 1998 report, 0.183g, is very conservative with respect to "*Rules for Dam Safety*" criteria, agency guidance, and standard practice [i.e., Seed, 1979]. In fact, a seismic coefficient of only 0.09 to 0.10g would be consistent with the criteria, guidance, and standard practice.

5.3 Soil Shear Strength

The use of undrained shear strengths for pseudo-static slope stability analyses of the Lake Petit Dam embankment fill materials is appropriate and consistent with agency guidance. Furthermore, undrained shear strength parameters used in the December 1998 report are conservative with respect to the laboratory strength test measurements on intact samples of dam material and are therefore consistent with the "*Rules for Dam Safety*" directive for use of conservative soil shear strength parameters.

5.4 **Summary**

This white paper presents information that shows that the seismic stability analyses presented in the December 1998 GeoSyntec report entitled "*Evaluation of Stability and Rehabilitation Measures, Lake Petit Dam, Big Canoe, Georgia*" are consistent with USCOE, FERC, and USBR guidance for safety evaluation of earth dams. These analyses are also consistent with "*Rules for Dam Safety*" requirements regarding factors of safety, seismic acceleration levels, and conservative selection of soil shear strength parameters. Therefore, the seismic stability analyses in the December 1998 GeoSyntec report satisfy the "*Rules for Dam Safety*" requirement for conformance with accepted practices of the engineering profession and dam safety industry.

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LIST OF FIGURES

UNIFORM BUILDING CODE [1994] SEISMIC ZONE MAP

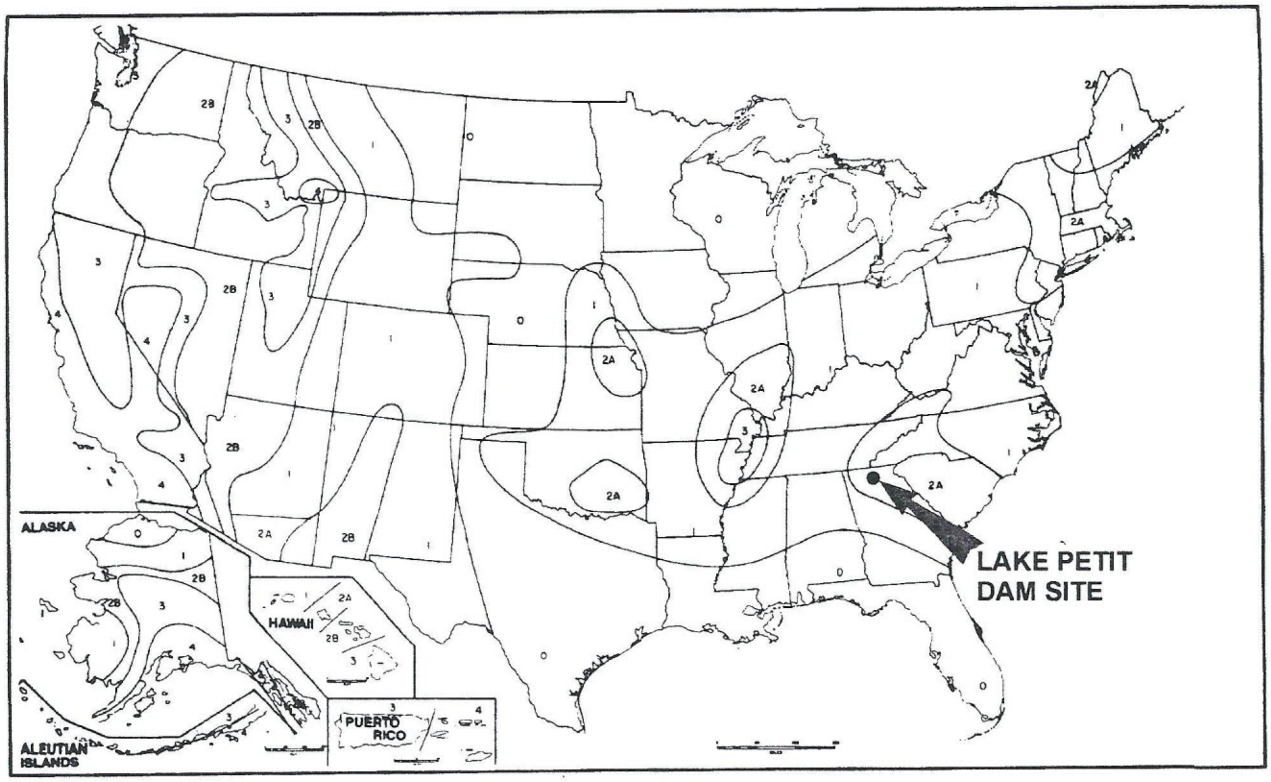

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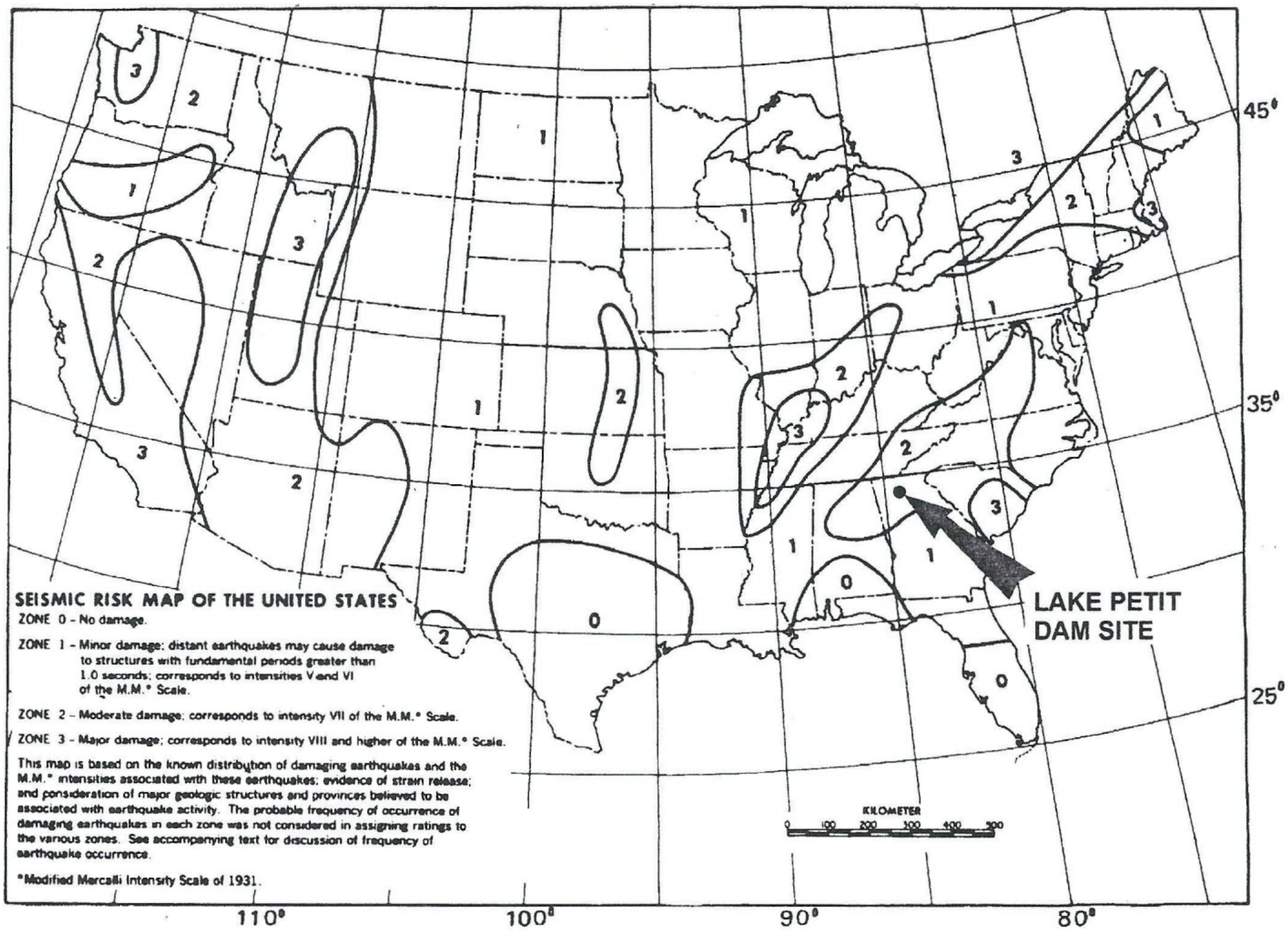
FIGURE NO.	1
PROJECT NO.	GL0625-100
DOCUMENT NO.	GA990283
FILE NO.	FIGS.cdr

ALGERMISSEN [1969] SEISMIC ZONE MAP



SOURCE: S.T. ALGERMISSEN

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FIGURE NO.	2
PROJECT NO.	GL0625-100
DOCUMENT NO.	GA990283
FILE NO.	FIGS.cdr

LABORATORY TESTING RESULTS SUMMARY
TABLE 3-1 FROM GEOSYNTEC [DECEMBER 1998]

TABLE 3-1

LABORATORY TESTING RESULTS

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

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

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FIGURE NO. 3
PROJECT NO. GL0625-100
DOCUMENT NO. GA990283
FILE NO. FIGS.cdr

SEISMIC ZONES AND SEISMIC COEFFICIENTS FROM USCOE [1970]

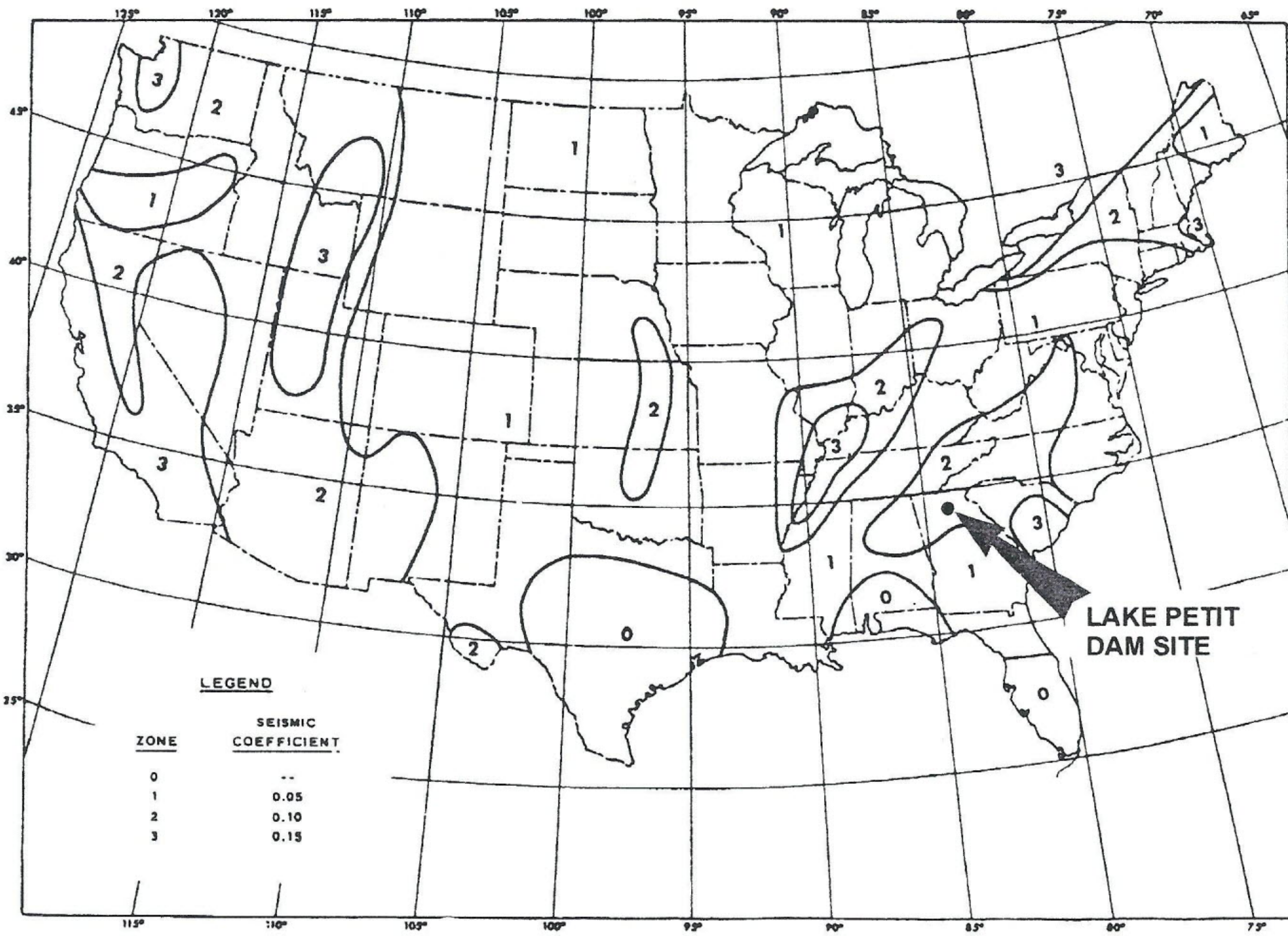

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FIGURE NO.	4
PROJECT NO.	GL0625-100
DOCUMENT NO.	GA990283
FILE NO.	FIGS.cdr

SOIL SHEAR STRENGTH ENVELOPE, STEADY-STATE SEEPAGE WITH SEISMIC LOADING FROM USCOE [1970]

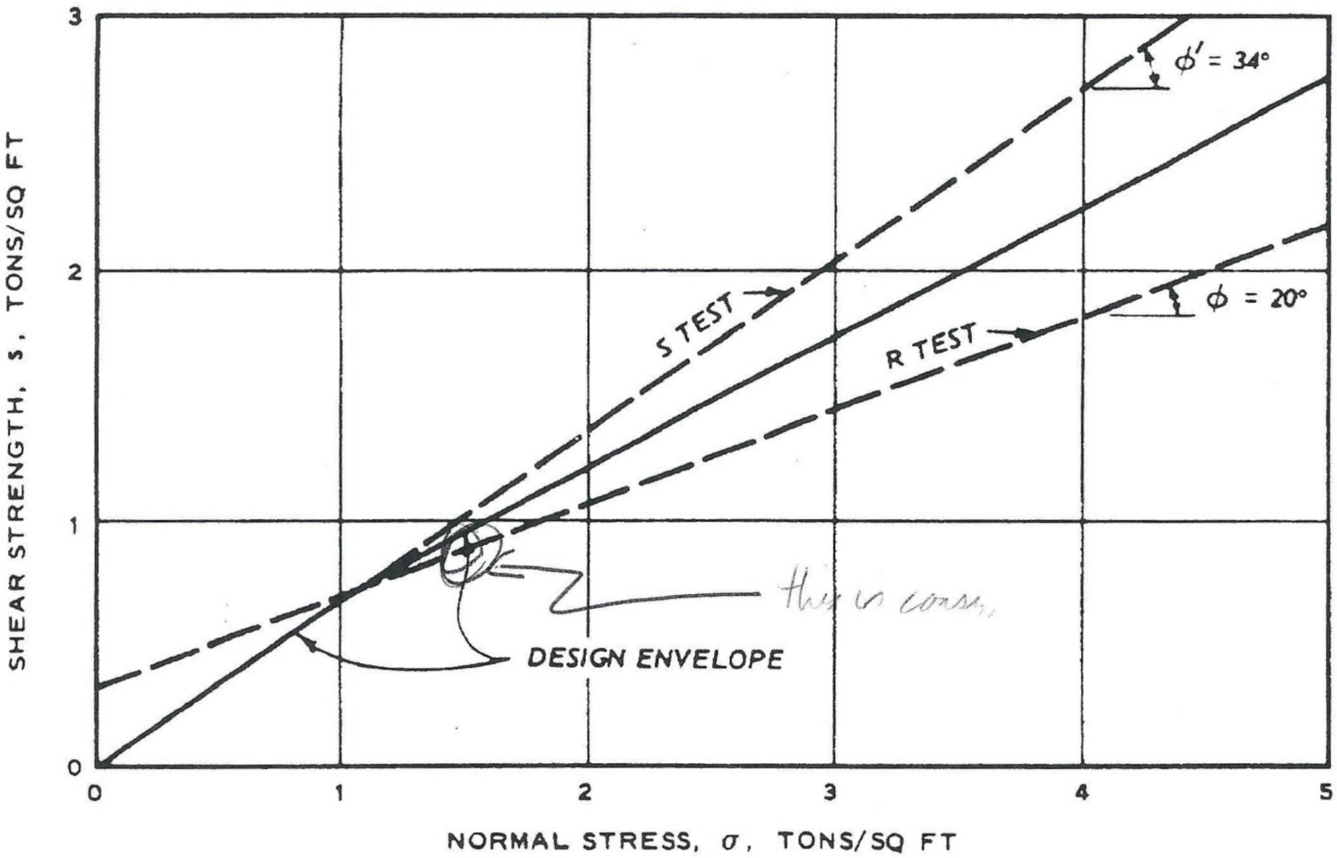

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FIGURE NO.	5
PROJECT NO.	GL0625-100
DOCUMENT NO.	GA990283
FILE NO.	FIGS.cdr

TABLE 8 FROM KRINITZSKY ET AL. [1998]

Material type		Layer to layer elevation Interface (feet)	total unit weight	Drained soil properties	Soil strengths used for slope stability calculations / Undrained soil properties
Select and pervious fill			120 pcf	$\phi_d = 35°$	$\phi_u = 35$
Impervious fill			120 pcf	$\phi_d = 28°$	$\phi_u = 13°$ $c_u = 600$ psf
Zone II fill			120 pcf	$\phi_d = 32°$	$\phi_u = 23°$ $c_u = 400$ psf
Zone I fill			125 pcf	$\phi_d = 31°$	$\phi_u = 13°$ $c_u = 600$ psf
Upper natural soil zone		70 ft	120 pcf	$\phi_d = 28°$	$\phi_u = 24°$ $c_u = 700$ psf
Middle natural soil zone	Non horizontal layers	41 ft	110 pcf	$\phi_d = 26°$	$\phi_u = 13°$ $c_u = 500$ psf
Middle natural soil zone	Short horizontal layers		110 pcf	$\phi_d = 18°$	$\phi_u = 13°$ $c_u = 500$ psf
Lower natural soil zone		18 ft	115 pcf	$\phi_d = 28°$	$\phi_u = 15°$ $c_u = 800$ psf
Shale		-28 ft	105 pcf	$\phi_d = 28°$ $c_d = 1000$	$\phi_u = 20°$ $c_u = 2600$ psf
Limestone		-41 ft	135 pcf	$\phi_d = 28°$ $c_d = 5700$	$\phi_u = 37°$ $c_u = 5700$ psf

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FIGURE NO. 6
PROJECT NO. GL0625-100
DOCUMENT NO. GA990283
FILE NO. FIGS.cdr

TABLE 3:1 FROM SOWERS [1979]

TABLE 3:1 / RELATIVE VALUES OF PERMEABILITY
(After Terzaghi and Peck)[3:5]

Relative Permeability	Values of k (mm/sec)*	Typical Formation
Very permeable	1	Coarse gravel, open-jointed rock
Medium permeability	1×10^{-2}	Sand, fine sand
Low permeability	$1 \times 10^{-2} - 1 \times 10^{-4}$	Silty sand, dirty sand
Very low permeability	$1 \times 10^{-4} - 1 \times 10^{-6}$	Silt, fine sandstone
Impervious	Less than 1×10^{-6}	Clay, mudstone without joints

*To convert to feet per minute, multiply above values by 0.2.