Closure Completion Notification for Closure by Removal

January 15, 2025
Closure Completion Notification
Flint Creek Plant
Primary Bottom Ash Pond

On December 24, 2024, the Flint Creek Plant Primary Bottom Ash Pond was transitioned to closure status in accordance with 40 CFR 257.102. This notice of completion of closure is being placed in the operating record in accordance with 40 CFR 257.102(h).

Effective with the Closure Completion Notification, the former ash storage site is no longer a CCR unit. The following operating record documents are no longer required going forward:

- Hazard Potential Classification
- Emergency Action Plan (EAP)
- Face to Face Meeting Documentation for EAP
- History of Construction and Revisions for Surface Impoundments
- Structural Stability Assessments
- Safety Factor Assessments
- Fugitive Dust Plan
- Inflow Design Flood System Control Plan

CLOSURE CERTIFICATION BY QUALIFIED PROFESSIONAL ENGINEER

I certify that the AEP Flint Creek Primary Bottom Ash Pond has been closed in accordance with the most recent written closure plan specified by 40 CFR 257.102(b) and the requirements of 40 CFR 257.102.

David Anthony Miller
Printed Name of Licensed Professional Engineer

David Anthony Miller
Signature

15296	Arkansas	01.15.2025
License Number	Licensing State	Date

4.0 PROFESSIONAL ENGINEER CERTIFICATION

This Closure By Removal Certification and Construction Summary Report confirms that the construction activities associated with the closure of the PBAP (refer to Section 3.0) were performed in accordance with the requirements contained in the Closure Plan; and therefore, meet the closure criteria defined in 40 CFR §257.102.

The following certification statement provides confirmation that this report was prepared by a qualified professional engineer registered in the state of Arkansas and that there is sufficient information to demonstrate that the closure of the PBAP meets the requirements of the Closure Plan and 40 CFR §257.102.

Professional Engineer's Certification

Based on the work completed, the visual observation of the pond bottom by CEC field personnel during CCR removal, and review of the final topographic survey, I certify that the CCR contained in the Flint Creek Primary Bottom Ash Pond has been removed in general accordance with the Closure Plan prepared by American Electric Power dated September 2016, revised October 2023.

Jeff A Shepherd, P.E.

Printed Name of Professional Engineer

Signature

10836	**Arkansas**	**01-30-2024**
Registration No.	**Registration State**	**Date**

SAFETY FACTOR ASSESSMENT PERIODIC 5-YEAR REVIEW

CFR 257.73e

Primary Bottom Ash Pond

Flint Creek Plant
Gentry, Arkansas

October, 2021

Prepared for: Southwestern Electric Power Company

Prepared by: American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215

Document ID: GERS-21-027

SAFETY FACTOR ASSESSMENT
PERIODIC 5-YEAR REVIEW
CCR 257.73(e)
FLINT CREEK PLANT
PRIMARY BOTTOM ASH POND

Document ID: GERS-21-027

PREPARED BY: ____________ DATE: 09-15-2021
Shah S. Baig, P.E.

REVIEWED BY: Brett A. Dreger DATE: 9/15/2021
Brett A. Dreger, P.E.

APPROVED BY: Gary F. Zych DATE: 9/21/2021
Gary F. Zych, P.E.
Section Manager – AEP Geotechnical Engineering

9/21/2021
STATE OF
ARKANSAS
LICENSED
PROFESSIONAL
ENGINEER
No. 17243
GARY F. ZYCH
Gary F. Zych

I certify to the best of my knowledge, information, and belief that the information contained in this safety factor assessment meets the requirements of 40 CFR § 257.73(e)

SAFETY FACTOR ASSESSMENT
PERIODIC 5-YEAR REVIEW
FLINT CREEK PLANT
PRIMARY BOTTOM ASH POND

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Attachment A

- Flint Creek Power Plant: CCR Rule Structural Stability Evaluation, February 2016

1.0 OBJECTIVE

This report was prepared by AEP- Geotechnical Engineering Services (GES) section to fulfill requirements of CCR 257.73(e) for the safety factor assessment of CCR surface impoundments. This is the first periodic 5-year review of the safety factor assessment.

2.0 DESCRIPTION OF THE CCR UNIT

The Flint Creek Power Plant is located near the City of Gentry, Benton County, Arkansas. It is owned and operated by Southwestern Electric Power Company (SWEPCO). The facility operates one surface impoundment for storing CCRs, referenced as the Primary Bottom Ash Pond.

The Primary Ash Pond dam is a cross valley dam on a tributary to the Little Flint Creek. The dam is 45 feet high and has side slopes of 3H:1V. The downstream slope is partially submerged by the Little Flint Creek Reservoir.

3.0 SAFETY FACTOR ASSESSMENT 257.73(e)

The periodic 5-year review was conducted to evaluate if any physical changes have been made to the earthen dam and/or operating changes that could impact the loading on the structure. The assumptions, material properties and operating pools defined in the initial assessment were reviewed. The review concluded that there have been no changes to the structure (e.g. materials, geometry, operating condition, etc.) that would impact the stability analyses that were previously conducted. Therefore, the previous report and analyses are still applicable to the current condition of the facility.

The results indicate that the calculated factors of safety meet or exceed the minimum values defined in Section 275.73(e).

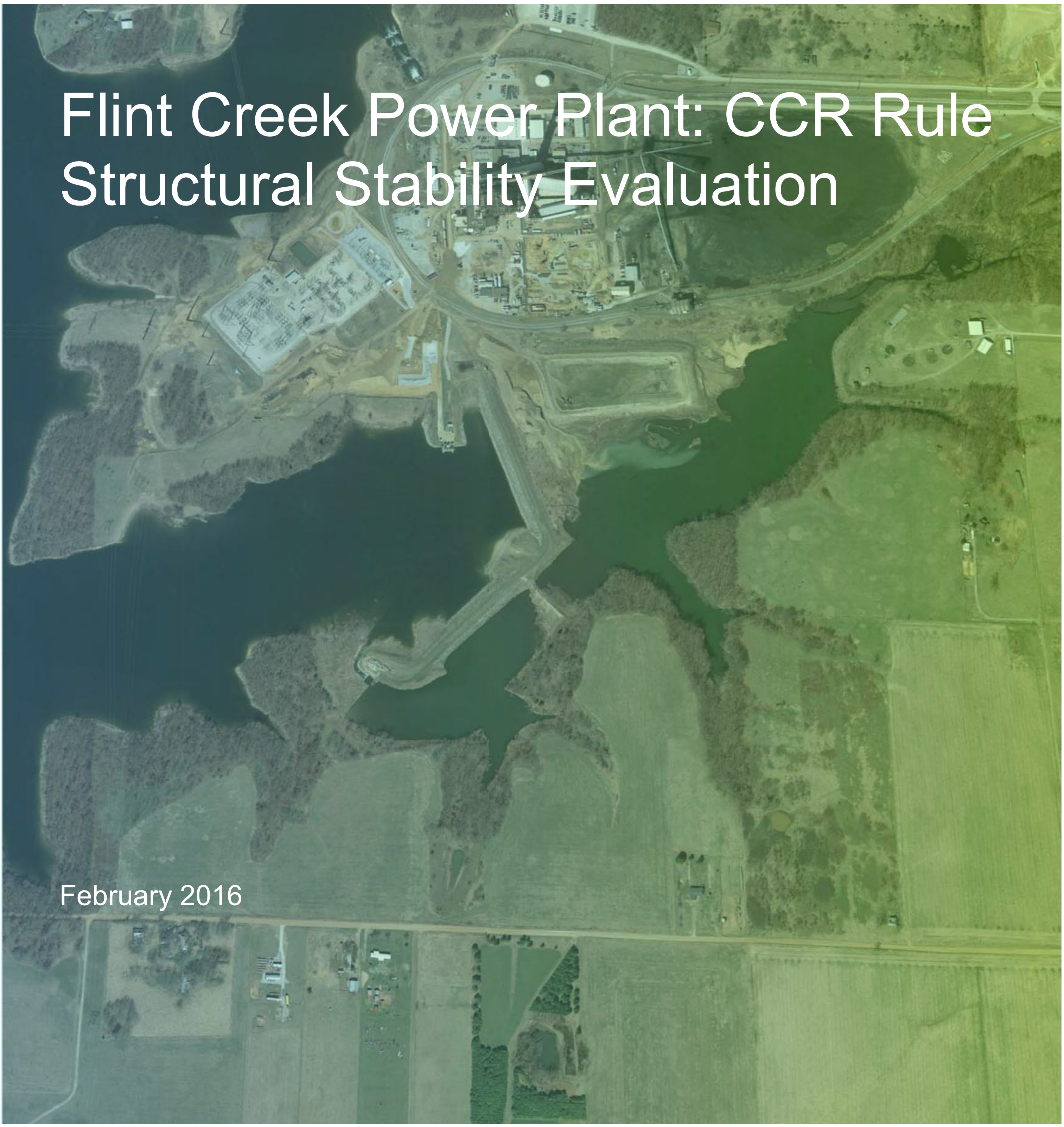
ATTACHMENT A

AECOM

Submitted to
American Electric Power
1 Riverside Plaza
Columbus, OH 43215-2372

Submitted by
AECOM
277 West Nationwide Blvd
Columbus, OH 43215

Flint Creek Power Plant: CCR Rule Structural Stability Evaluation



February 2016

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Tables

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Acronyms

AEP – American Electric Power

ASCE – American Society of Civil Engineers

CCR – Coal Combustion Residuals

CH – fat clay

CIU – isotropically consolidated undrained

CL – lean clay

El – Elevation (in feet MSL)

ETTL – ETTL Engineers & Consultants, Inc.

FoS – Factor of Safety

GC – clayey gravel

IBC – International Building Code

k_h – pseudostatic coefficient

MSL – Mean Sea Level

pcf – pounds per cubic foot

PGA – Peak Ground Acceleration

PMF – Probable Maximum Flood

psf – pounds per square foot

$\sigma_1 - \sigma_3$ – maximum deviator stress

σ_1 / σ_3 – maximum ratio of principal effective stresses

SC – clayey sand

S_{MS} – maximum earthquake spectral response acceleration

SPT – Standard Penetration Test

USEPA – United States Environmental Protection Agency

USGS – United States Geological Survey

1.0 Introduction and Background

This report presents the results of AECOM's review and independent analyses of the geotechnical investigation in *Flint Creek Power Station, Existing Ash Storage Ponds Embankment Investigation* prepared by ETTL Engineers & Consultants, Inc. (ETTL) on August 18, 2010. The Flint Creek Power Station is located at 21797 SWEPCO Plant Road in Benton County, Arkansas, near Gentry. The power plant is located on the northeast side of Lake Flint Creek, which serves as the cooling water source for the power plant. The Primary and Secondary Ash Ponds are located to the south of the plant on the east side of the Little Flint Creek Reservoir (see site plan on cover page). ETTL (2010) evaluated the subsurface stratigraphy within the limits of borings; evaluated the classification, strength and permeability characteristics of the embankment and foundation soils; and performed slope stability and seepage analyses of the existing embankments.

1.1 Purpose

AECOM was contracted to perform evaluations and verify that the United States Environmental Protection Agency (USEPA) Coal Combustion Residuals (CCR) rule's minimum requirements for structural stability are met for the following conditions in Section 257.73 for the Bottom Ash Complex (Primary and Secondary Ash Ponds) at the Flint Creek Power Plant near Gentry, Arkansas:

a. The calculated Factor of Safety (FoS) under the steady state, long term, maximum storage pool loading condition must equal or exceed 1.50;
b. The calculated FoS under the short term, surcharge pool loading condition must equal or exceed 1.40;
c. The calculated pseudostatic seismic FoS must equal or exceed 1.00;
d. For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction FoS (also known as post-earthquake slope stability FoS) must equal or exceed 1.20.

2.0 Evaluation of Analysis Parameters

AECOM conducted a review of *Flint Creek Power Station, Existing Ash Storage Ponds Embankment Investigation* (ETTL, 2010) for this study. Specifically, AECOM examined the existing geotechnical information and performed an assessment as to whether the information is sufficient to perform independent slope stability analyses, or whether additional investigation and laboratory analyses are required in order to complete the required analyses.

2.1 Soil Parameters

The fill material in the embankment consists primarily of stiff to very stiff lean clay (CL) or fat clay (CH) with gravel and medium dense clayey gravel (GC) or clayey sand (SC). The native soils underlying the fills are predominantly clayey gravel (GC) and hard lean clay (CL) with gravel over the limestone formation. ETTL performed three triaxial tests under drained and undrained conditions to obtain shear strength parameters at the site. In areas where triaxial tests could not be performed (areas with significant gravel), ETTL chose the average shear strength values of the fill and native soils based on soil types and Standard Penetration Test (SPT) blow count correlations. These results are shown in Table 1 below.

Table 1. Summary of Soil Test Results (ETTL, 2010)

Pond	Material Type	Effective Stress Parameters			Total Stress Parameters		
		Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Primary Ash Pond	Fill	129	24	460	129	14.1	575
	Native Soil	130	33	90	130	18.3	275
	Native Rock	148	38.5	1000	148	38.5	1000
Secondary Ash Pond	Fill	130	33.7	0	130	15.9	345
	Native Soil	130	33	90	130	18.3	275
	Native Rock	148	38.5	1000	148	38.5	1000

The results of the Isotropically Consolidated Undrained (CIU) triaxial tests were plotted by AECOM on p'-q and p-q plots (see Figures 1 and 2). Failure was defined using the maximum stress difference criteria (σ_1 - σ_3 or the maximum deviator stress), as the ETTL report does not contain sufficient data to also define failure using the maximum ratio of principal effective stresses during the triaxial test (σ_1 / σ_3 or maximum obliquity). Failure at maximum deviator stress was plotted as a single point for the two different material types (fill and residuum/native soil) present at both ponds. In reviewing Figures 1 and 2, AECOM found that the embankment fill and residuum soils all plotted consistently on a single failure envelope for both ponds, indicating that the two materials have similar shear strengths. This is not unexpected as the embankment fills are most likely well-compacted residuum. Appendix A presents the background and findings for the development of the design shear strengths. Table 2 provides a summary of the soil parameters selected by AECOM for our independent analyses.

Table 2. Summary of Soil Parameters Selected by AECOM

Pond	Material Type	Effective Stress Parameters			Total Stress Parameters		
		Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Primary and Secondary Ash Ponds	Fill and Native Soil	130	31	50	130	14	500
	Native Rock	148	38.5	1000	148	38.5	1000
	Riprap	130	40	0	130	40	0

For the slope stability analyses, ETTL reduced the shear strength parameters (shown in Table 1) by 15% in an attempt to accommodate potential variations in the soil as well as to compensate for the limited amount of data. AECOM has not typically reduced the shear strength data in the past based on sparse data and instead has used the peak shear strengths (as shown in Table 2) for our independent slope stability analyses. AECOM also included a 2 foot thick layer of riprap along the downstream face of the slope extending from the top of the dam to the toe. The riprap face was observed during the site visit as well as from aerial imagery in Google Earth. The parameters assumed for the riprap are provided in Table 2 and were developed using engineering judgment and experience. AECOM also reviewed ETTL's shear strength values for Native Rock, and found them to be somewhat conservative for weathered limestone. However, the strength of the Native Rock is unlikely to substantially affect the slope stability analyses, as most slip surfaces will be confined to the lower-strength fill and residuum.

ETTL used effective stress parameters for steady state and seismic conditions, and total stress parameters for drawdown conditions. AECOM agrees that effective stress parameters should be used in steady state conditions; however total stress parameters should be used in seismic conditions. Typically, seismic loading occurs rapidly enough that induced excess pore water pressures do not have time to dissipate and undrained conditions and soil strengths are applicable. An analysis of drawdown conditions is not required by the CCR Rule, and has not been performed by AECOM.

2.2 Water Levels

A summary of the water levels for this project is shown in Table 3. All elevations listed in this report are given in feet above mean sea level (MSL). Currently, neither pond is on the Arkansas Natural Resources Commission's (ANRC) list of dams, and therefore does not have a State hazard classification, which would determine the design inflow event. AEP has recently conducted a Hazard Classification for both ponds per the EPA CCR Rule and determined that both ponds classify as "Low" hazard, which would correspond to a 100-year flood event. That event and higher intensity storms up to the full (Probable Maximum Flood) PMF were analyzed in the latest hydraulic report available for the site (*the Hydraulic Analysis of Flint Creek Power Plant Ash Ponds* by Freese and Nichols (2011)). For conservatism, AEP has requested that the ponds be analyzed with the pool elevation corresponding to the 50% PMF event. The steady state pool elevations are based on normal operating levels reported by AEP. Seasonal variations in the lake level (tailwater) ranges from 1130 feet MSL in October through December to 1137 feet MSL in May. ETTL used 1140 feet MSL (spillway elevation) for the lake level in their analyses.

Table 3. Summary of Water Levels

Ash Pond	Headwater (feet MSL)		Tailwater (feet MSL)	
	Normal (Steady State)	Flood (50% PMF)	Normal and Flood	Seasonal Lake Variation
Primary Ash Pond	1146	1151.96	1130	1130 – 1137
Secondary Ash Pond	1143	1150.8	1130	1130 – 1137

Note: 100-year headwater elevations for the two ponds are 1149.48' and 1148.35' for the Primary and Secondary Ponds respectively.

2.3 Seismic Design Parameters and Liquefaction

ETTL determined that under the International Building Code methodology (IBC), the embankment soils are Site Class D (Stiff Soil Profile). In their seismic analyses, they used the IBC methodology to establish the maximum earthquake spectral response acceleration parameter, S_{MS}, equal to 0.217 for 10% probability of exceedance in 50 years. ETTL used the computer program, GSTABL7, to evaluate slope stability. Pseudostatic earthquake (seismic) analyses are performed in this program with the input of a pseudostatic coefficient. There are numerous references for selecting the pseudostatic coefficient, k_h, based on the Peak Ground Acceleration (PGA), with most ranging from 1/3 to 2/3. Since the USEPA CCR rule does not stipulate a value for k_h and since there is no formal, definitive reference on it, the selection of k_h can be left up to the experience of the user. Based on AECOM's past experiences and popular references such as Hynes-Griffin and Franklin (1984) and Kramer (1996), half of the PGA tends to be a reasonable estimate for the pseudostatic coefficient for earthen dams with a FoS greater than 1.0. Generally, AECOM does not use the S_{MS} as the pseudostatic coefficient for analyses; however ETTL's approach is on the conservative side.

Generally, clean sandy soils below the groundwater level are susceptible to liquefaction conditions during an earthquake. The embankment soils at the Flint Creek Power Station are predominantly clayey gravels (GC) and lean clays with gravel (CL) and AECOM agrees with ETTL that the liquefaction potential at the site is low. No further liquefaction analysis is required to show that the embankment and foundation materials are not susceptible to liquefaction under the design seismic event.

3.0 Site Visit

Mr. Colin Young, P.E. performed a brief walkdown of the site on August 21, 2015. Mr. Young was accompanied by Mr. Greg Carter, P.E. of AEP. The purpose of the walkdown was to verify whether any conditions to the ash pond dikes had changed since the ETTL study in 2010. It was verified that no changes had been made to the dikes during that time period from 2010 to August 2015 and that physical conditions of the dikes were substantially similar to those existing at the time of ETTL's study.

4.0 Geotechnical Analysis

AECOM performed stability analyses appropriate to determine if the impoundments meet the Section 257.73 stability criteria. The Primary and Secondary Ash Ponds were both analyzed for these purposes. Results are presented in the following sections.

4.1 Slope Stability Analyses

Slope stability analyses were conducted using the 2-dimensional limit equilibrium software, SLOPE/W (GEO-SLOPE International, Ltd., 2012). Circular failure surfaces were evaluated using Spencer's Method, which considers force and moment equilibrium. Non-circular slip surfaces are generally not applicable in mostly homogeneous soil profiles similar to the conditions at this site. The grid and radius, and entry and exit methods were both used to define the circular slip surfaces. The following load cases were considered per the CCR Rule Section 257.73:

1) Steady state, long term, maximum storage pool condition with a FoS requirement of 1.50;
2) Short term, surcharge pool condition (short term flood load) with a FoS requirement of 1.40, this was performed at the 50% PMF pool levels;

3) Pseudostatic seismic using horizontal ground accelerations from published USGS peak PGA for 2% probability of exceedance in 50 years (e.g. 2,475-yr return period) with a FoS requirement of 1.00;
4) Post-seismic or post-liquefaction condition for dikes constructed of soils susceptible to liquefaction with a FoS requirement of 1.20.

All of the above cases were analyzed except the post-seismic/post-liquefaction load case. As mentioned previously in Section 2.3 of this report, AECOM does not consider the site soils susceptible to liquefaction under the design seismic event.

The soil parameters used in the stability analyses are provided in Table 2. Per the IBC (2012) and ASCE 7-10 (2013), the site classification was evaluated based on the average blow count in the upper 100 feet of the soil profile. The most critical soil profile (exploratory boring with the thickest fill layer) was selected and an average SPT blow count per formational material was estimated (see Appendix B). The average blow count in the upper 100 feet is approximately 39, which corresponds to Site Class D (Stiff Soil Profile). Using the Site Class information and site coordinates of the ash ponds, the US Seismic Design Maps (USGS, 2008) web tool was used to obtain the base PGA. The design maps detailed report (USGS web tool output) is provided in Appendix B and shows that the base PGA was calculated to be 0.072. The plot shown in Figure 3 shows the upper bound relationship between the peak transverse base acceleration and the peak transverse crest acceleration as developed by Harder (1991) and presented in FHWA (2011). The crest PGA that corresponds to the 0.072 base PGA is equal to 0.27. Based on AECOM's past experiences and popular references such as Hynes-Griffin and Franklin (1984) and Kramer (1996), half of the PGA tends to be a reasonable estimate for the pseudostatic coefficient for earthen dams with a FoS greater than 1.0. The pseudostatic coefficient used in AECOM's analyses is 0.135 (50% of 0.27).

The slope stability cross sections were developed based on information from ETTL (2010), Freese and Nichols, Inc. (2011) and past AEP inspection reports. The top of dam for both the Primary and Secondary Dams is 1155 feet MSL with a crest width of 12 feet and side slopes of 3H:1V for the upstream and downstream faces. The fill material was assumed to be the maximum height at the center of the dam corresponding to 46 feet at the Primary Dam and 35 feet at the Secondary Dam. The soil profile used in AECOM's analyses was taken directly from the ETTL slope stability analyses (2010) and verified using the applicable boring logs (ETTL, 2010).

The graphical slope stability analysis results are provided in Figures 4 through 6 for the Primary Ash Pond and Figures 7 through 9 for the Secondary Ash Pond. A summary of the slope stability FoS results are shown in Table 4. Each analyzed case meets the rule's minimum FoS requirements.

Table 4. Slope Stability Results

Pond	Conditions	Water Level (feet MSL)		Pseudostatic Coefficient, k_h[a]	Figure Number	FoS[b]	FoS Required
		Head	Tail				
Primary Ash Pond	Steady State Max Storage Pool	1146	1130	0	4	1.66	1.50
	Surcharge Pool (50% PMF)	1151.96	1130	0	5	1.51	1.40
	Pseudostatic Seismic	1146	1130	0.135	6	1.05	1.00
Secondary Ash Pond	Steady State Max Storage Pool	1143	1130	0	7	1.76	1.50
	Surcharge Pool (50% PMF)	1150.80	1130	0	8	1.58	1.40
	Pseudostatic Seismic	1143	1130	0.135	9	1.19	1.00

Notes:
a) The pseudostatic coefficient is taken to be half of the crest PGA.
b) FoS reported in table is the lower of the two FoS calculated using entry and exit and grid and radius methods.

5.0 Summary and Conclusions

In reviewing the existing field and lab data as well as the stability and seepage analyses, AECOM concludes that there is sufficient data to conclude that the ash ponds meet the CCR rule stability criteria.

Using the full peak shear strength data, AECOM performed slope stability analyses of both the Primary and Secondary Ash Ponds for the following conditions: 1) long term, steady state maximum storage pool; 2) short term flood at 50% PMF; and 3) pseudostatic seismic. All conditions met minimum FoS criteria..

6.0 Certification

I, Colin Young, being a Registered Professional Engineer in good standing and in accordance with the State of Arkansas, do hereby certify, to the best of my knowledge, information, and belief that the information contained in this report is true and correct and has been prepared in accordance with the accepted practice of engineering. I certify that the information contained in this report MEETS THE REQUIREMENTS of the Coal Combustion Residual (CCR) Rule, Section 257, specifically, Section 257.73 (e) for the specific requirements of the Periodic Safety Factory Assessments. This certification is for the Initial Assessment only and this certification does not certify that any other previous or future Periodic Assessments meet the requirements stated in Section 257.73 (e). This certification is for compliance with the section referenced and is not applicable for any other sections of the CCR Rule. Requirements within Section 257.73 that are not included within subsection 257.73 (e) are excluded from this certification. Exclusions within the reference section 257.73 (e), and within section 257.73 that pertains to all subsections, that are not covered by this certification include:

1. 257.73 (e)(2), Initial and each subsequent periodic safety factor assessment except the specific assessment being certified with this statement,
2. 257.73 (f), Timeframes for periodic and subsequent assessments, and
3. 257.70 (g), Recordkeeping.

These exclusions are not the responsibility of the certifying engineer and are outside the control of the certifying engineer.

Colin J. Young PE

Printed Name

02-22-2016

7.0 Limitations

Some of the information in this report and on supporting figures, drawings, and calculations is based on information provided by AEP and their subcontractors. AECOM has assumed this information is accurate, correct, valid, and was developed following current engineering practice.

The conclusions in this report are based on AECOM's understand of current plant operations, ash handling procedures, stormwater management, and conditions at the Flint Creek Power Plant, as of the date of this report, as provided by AEP. Changes in plant operations, stormwater management, or ash handling procedures may invalidate the findings in this report, until AECOM has had the opportunity to review the changes and, if necessary, modify our findings accordingly.

8.0 References

AECOM, 2015. *Scope and Fee Estimate for CCR Rule Structural Stability Certification, AEP Flint Creek Power Plant, near Gentry, Arkansas,* prepared for American Electric Power, May 15.

American Electric Power, Evaluation of Flint Creek's Primary and Secondary Ash Ponds for Dam Safety Hazard Classification, February 16, 2016.

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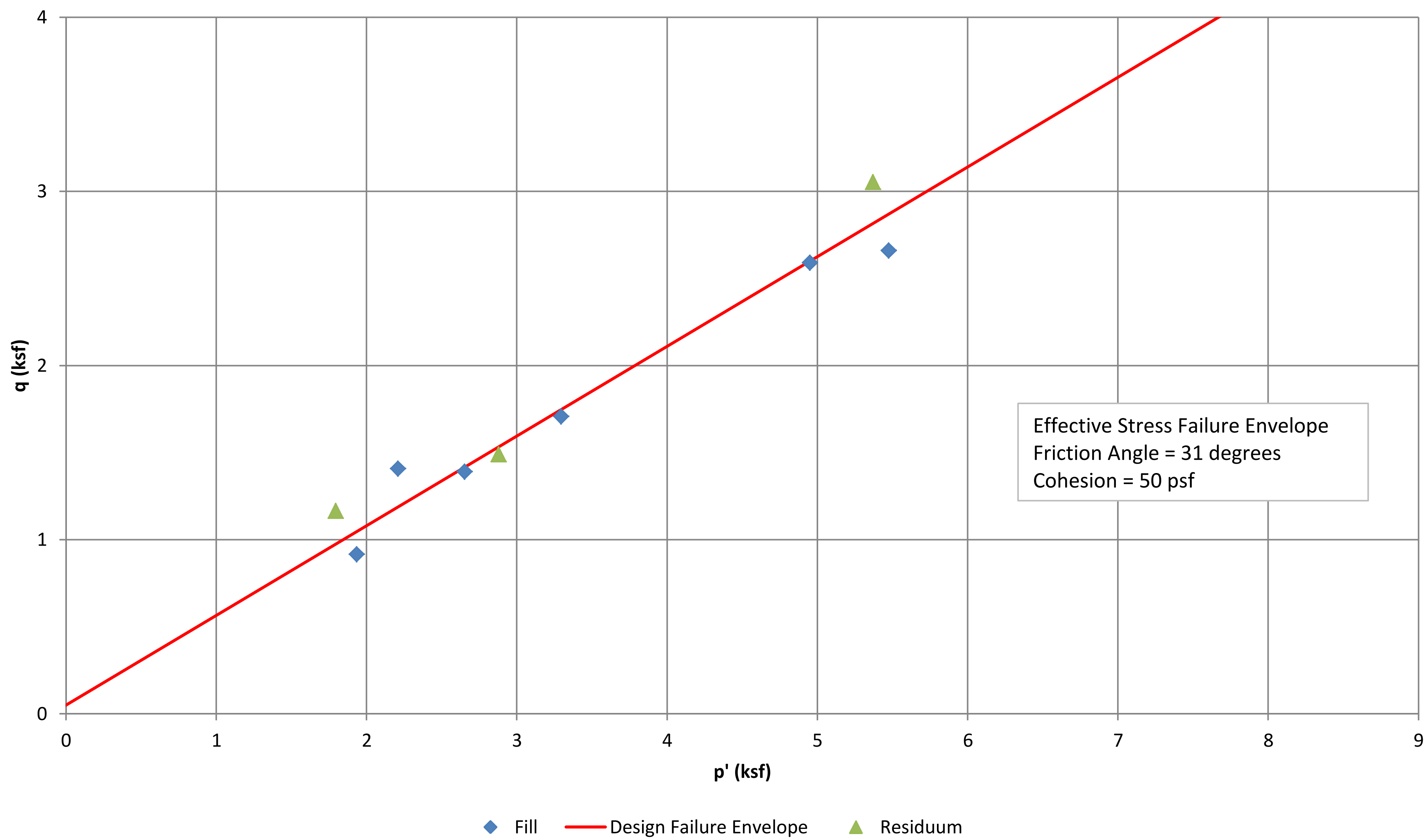
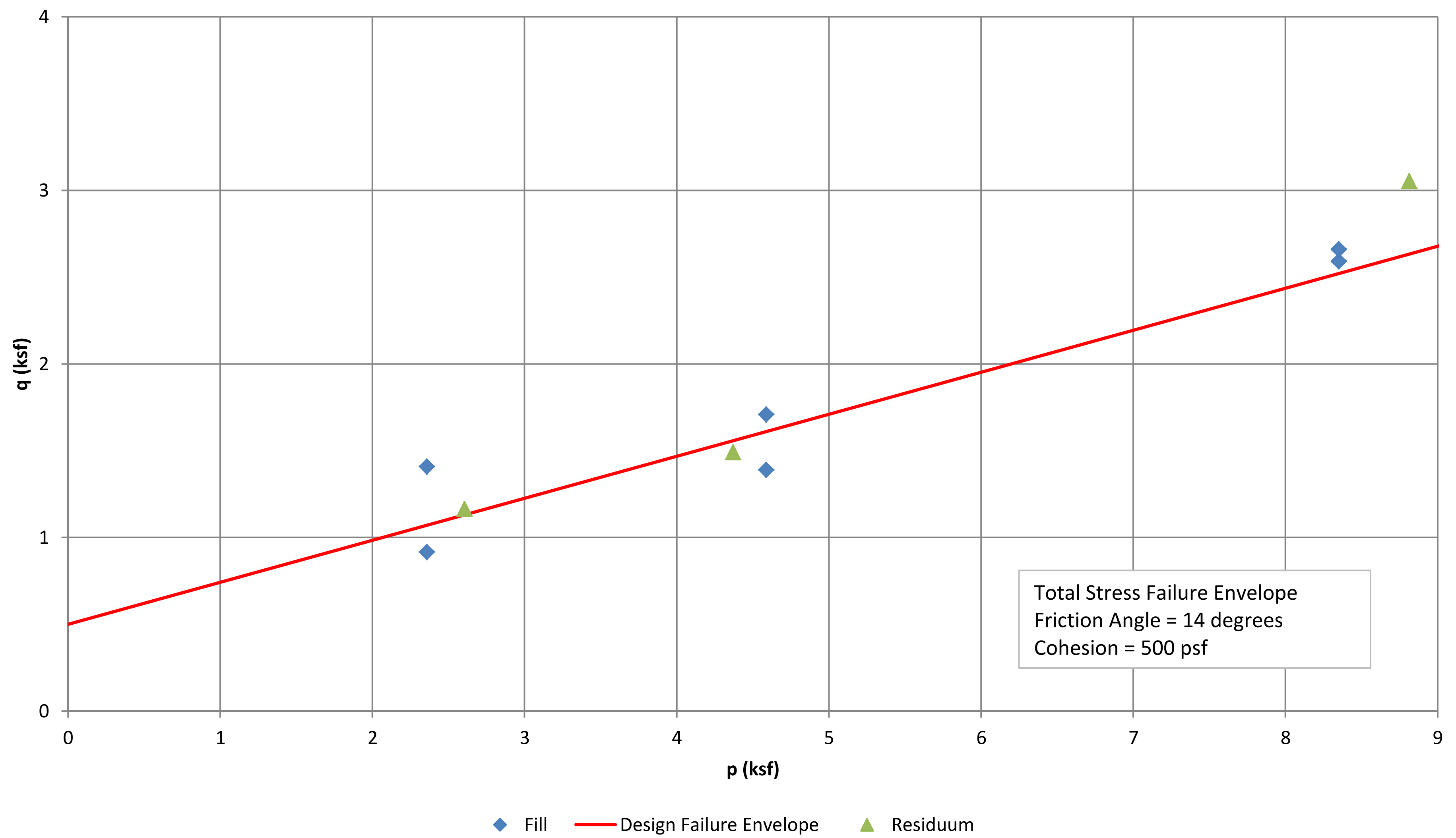
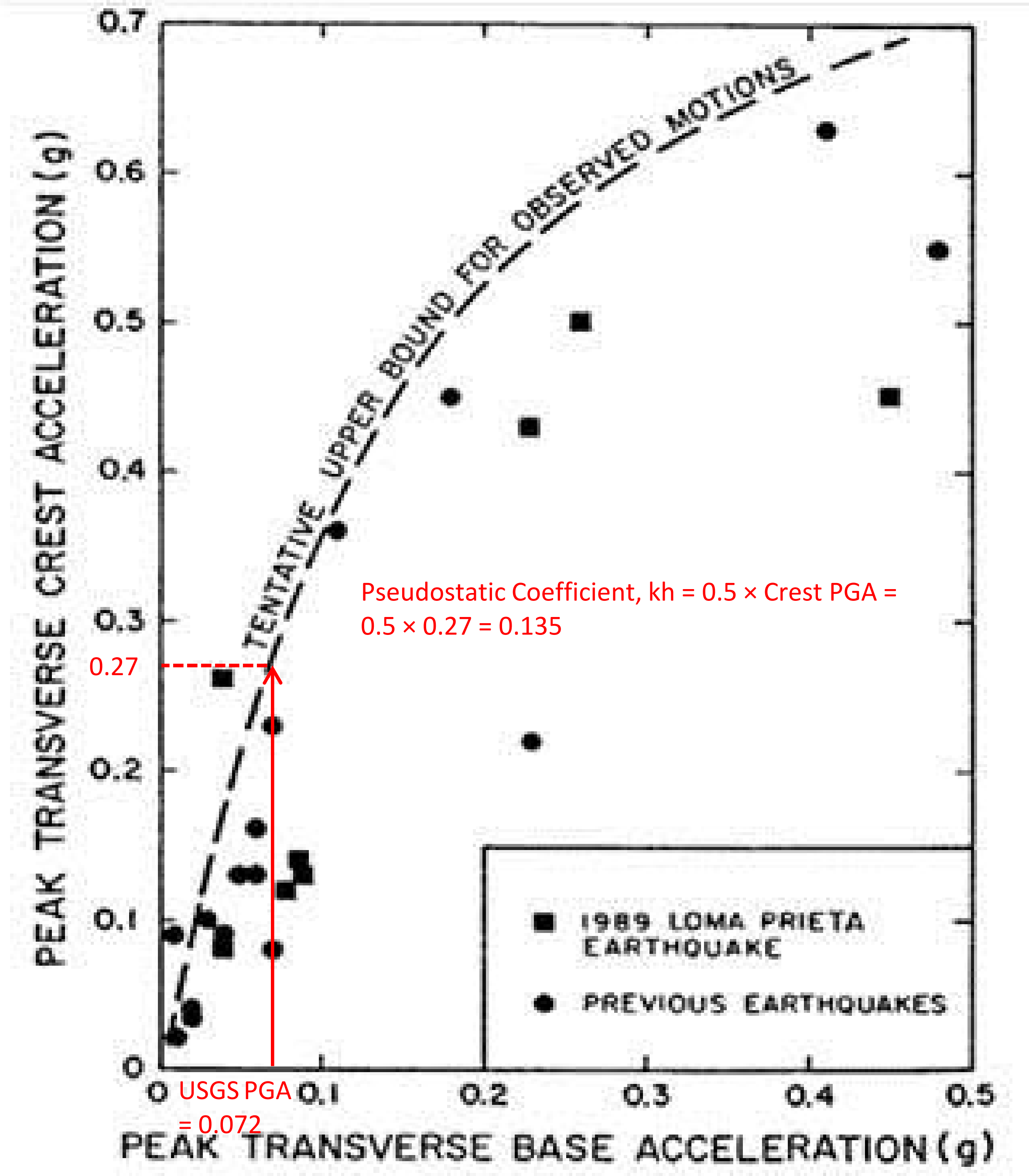
Figures



AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR	**p'-q Plot** **Drained Shear Strengths**	Figure 1
	Project No. 60437225 / Date: December 2015		

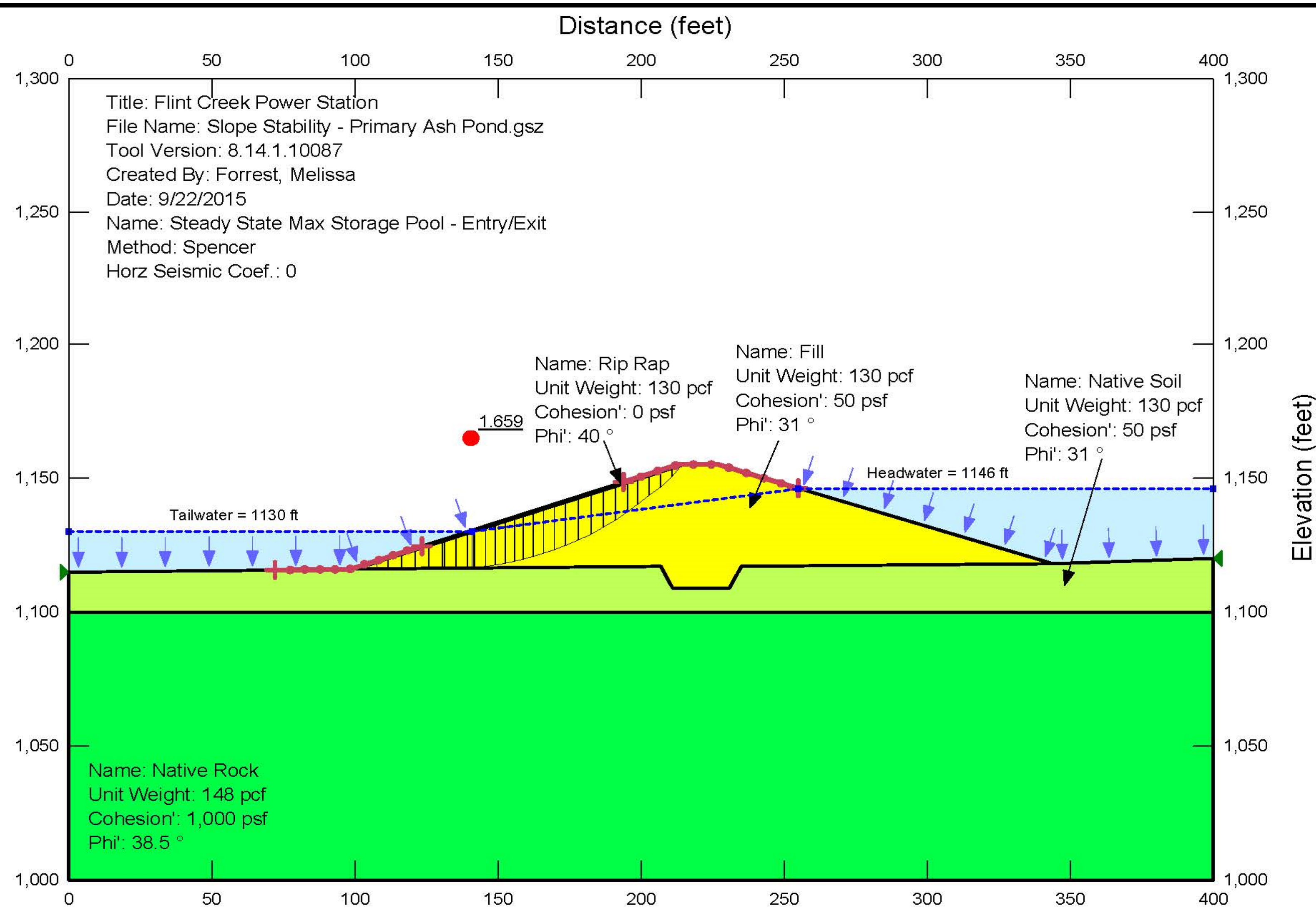


AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR		**p-q Plot** **Undrained Shear Strengths**	Figure 2
	Project No. 60437225	Date: December 2015		

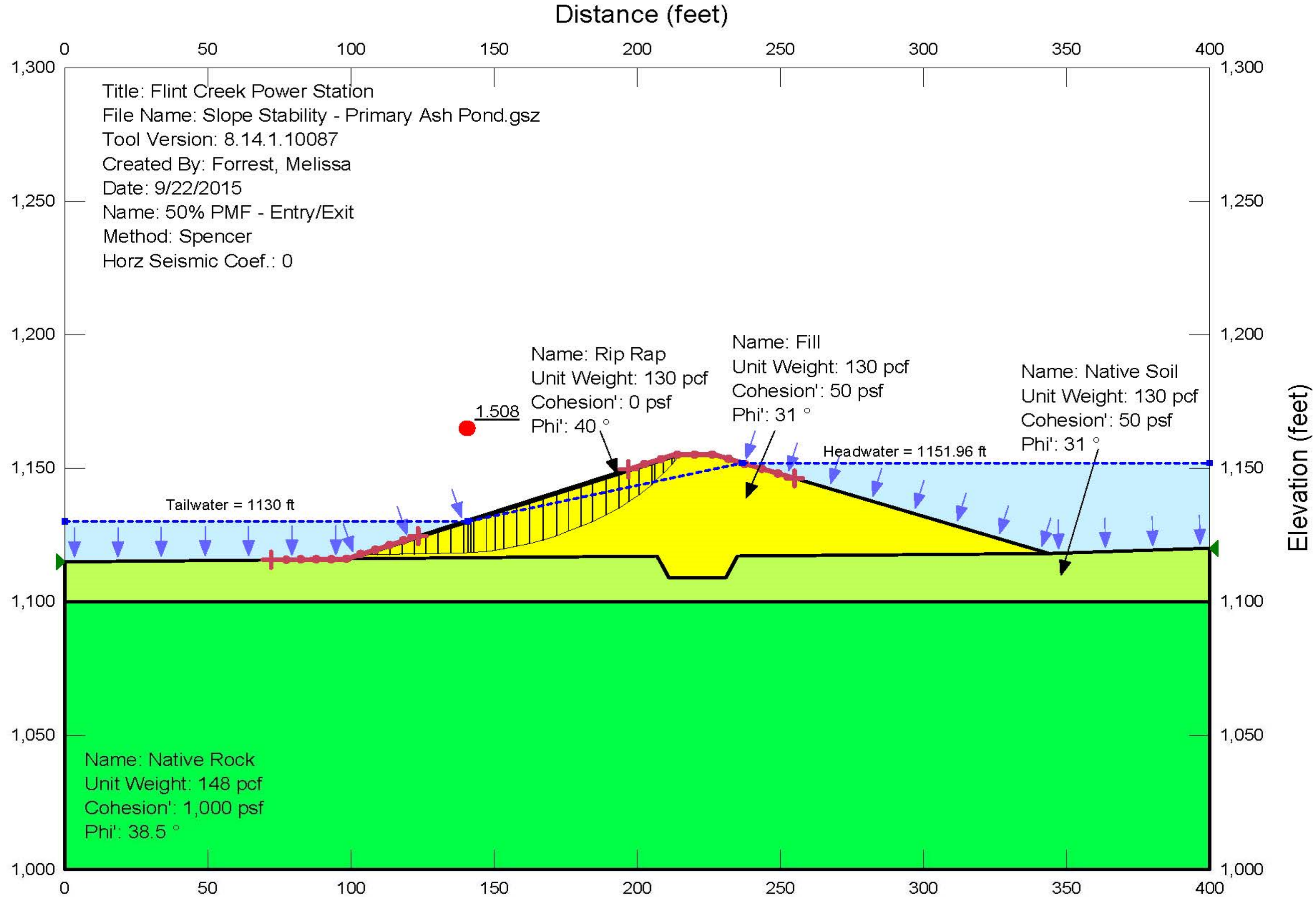


Reference: Base and Crest Peak Accelerations Recorded at the Earth Dams (Harder, 1991) as referenced in FHWA (2011)

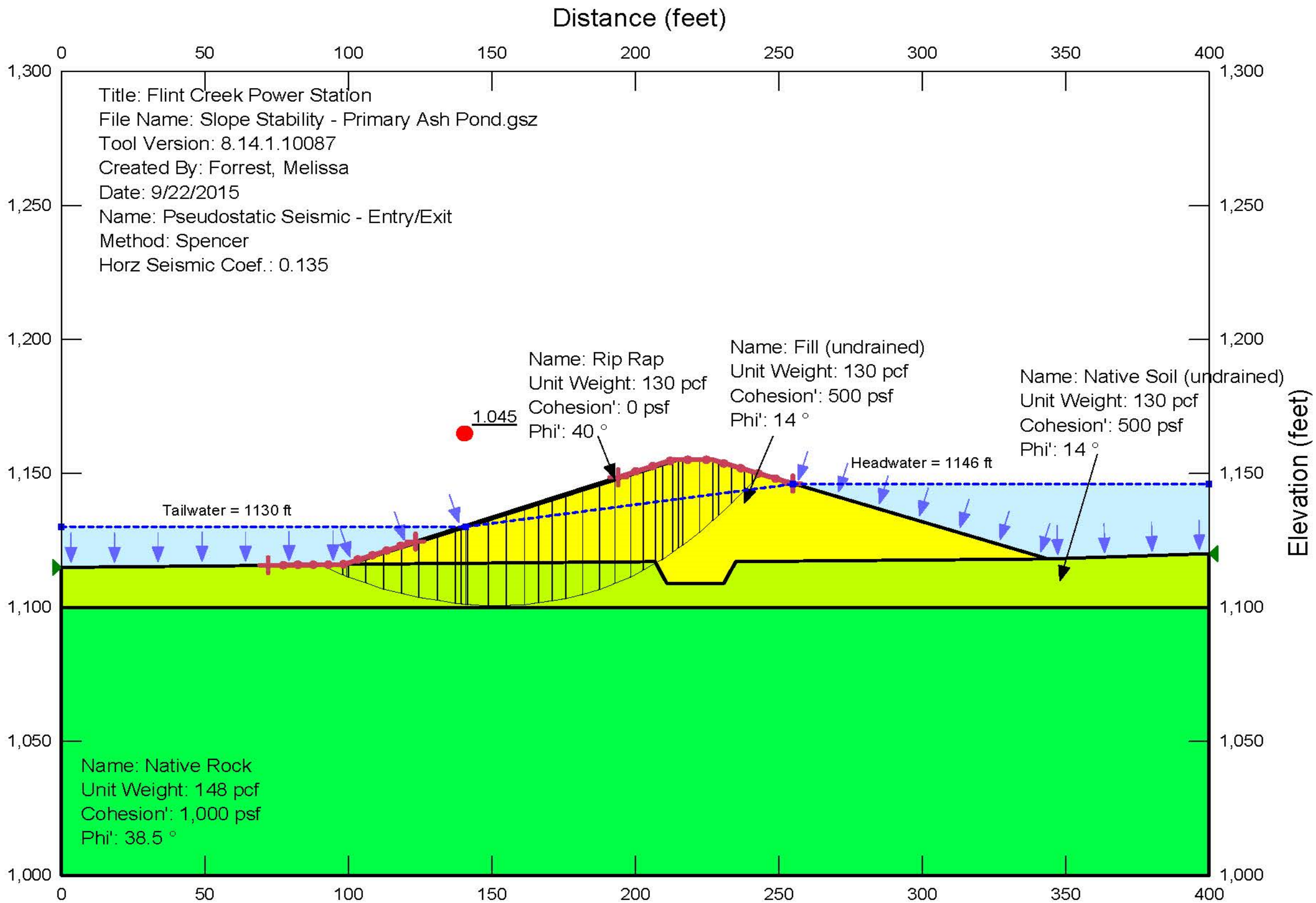
AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR		**Base and Crest Peak Accelerations**	Figure 3
	Project No. 60437225	Date: December 2015		



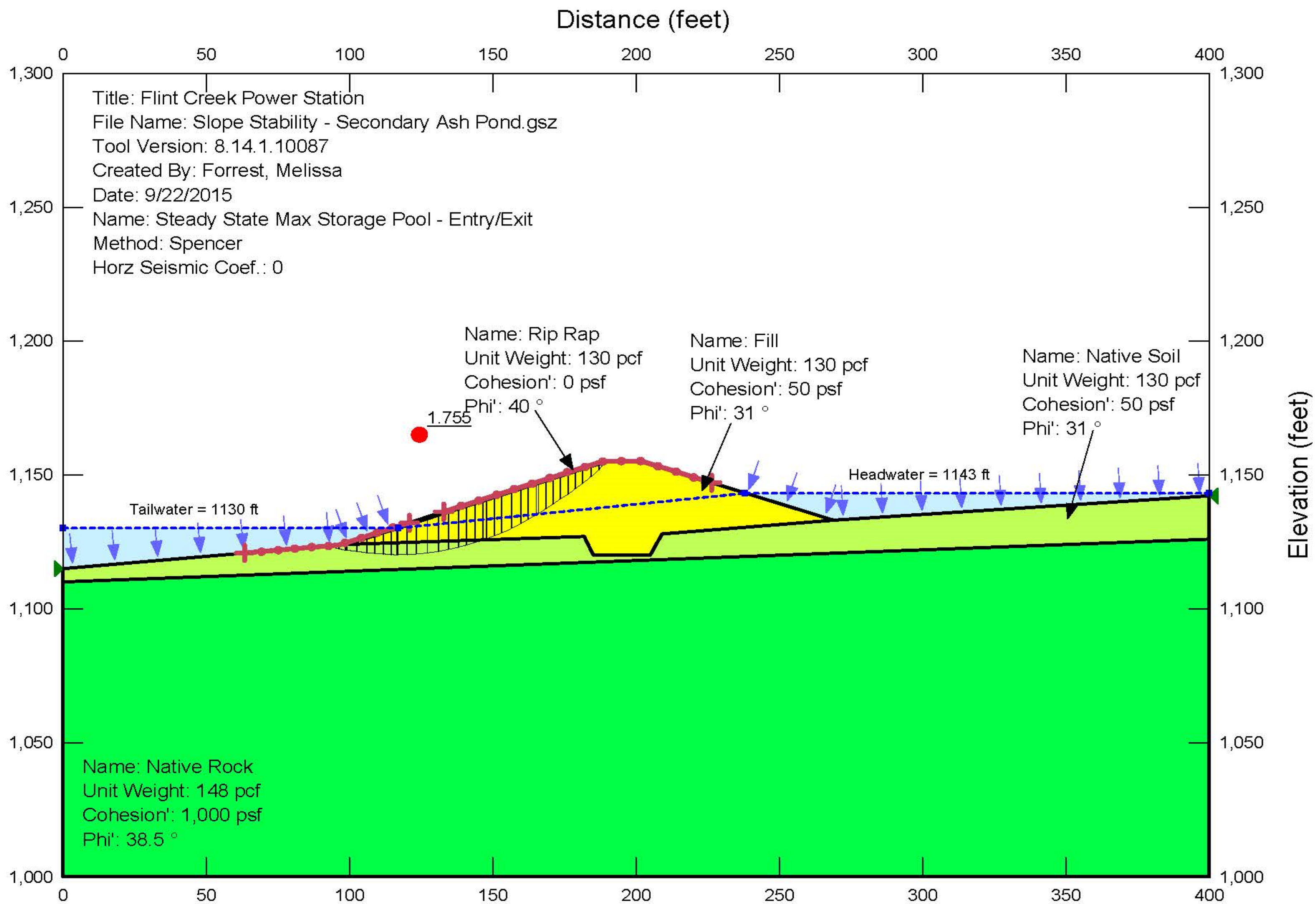
AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR		**Primary Ash Pond** **Steady State** **Max Storage Pool**	Figure 4
	Project No. 60437225	Date: December 2015		



AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR	**Primary Ash Pond** **Surcharge Pool (50% PMF)**	Figure 5
	Project No. 60437225 — Date: December 2015		



AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR		**Primary Ash Pond** **Pseudostatic Seismic**	Figure 6
	Project No. 60437225	Date: December 2015		



AECOM

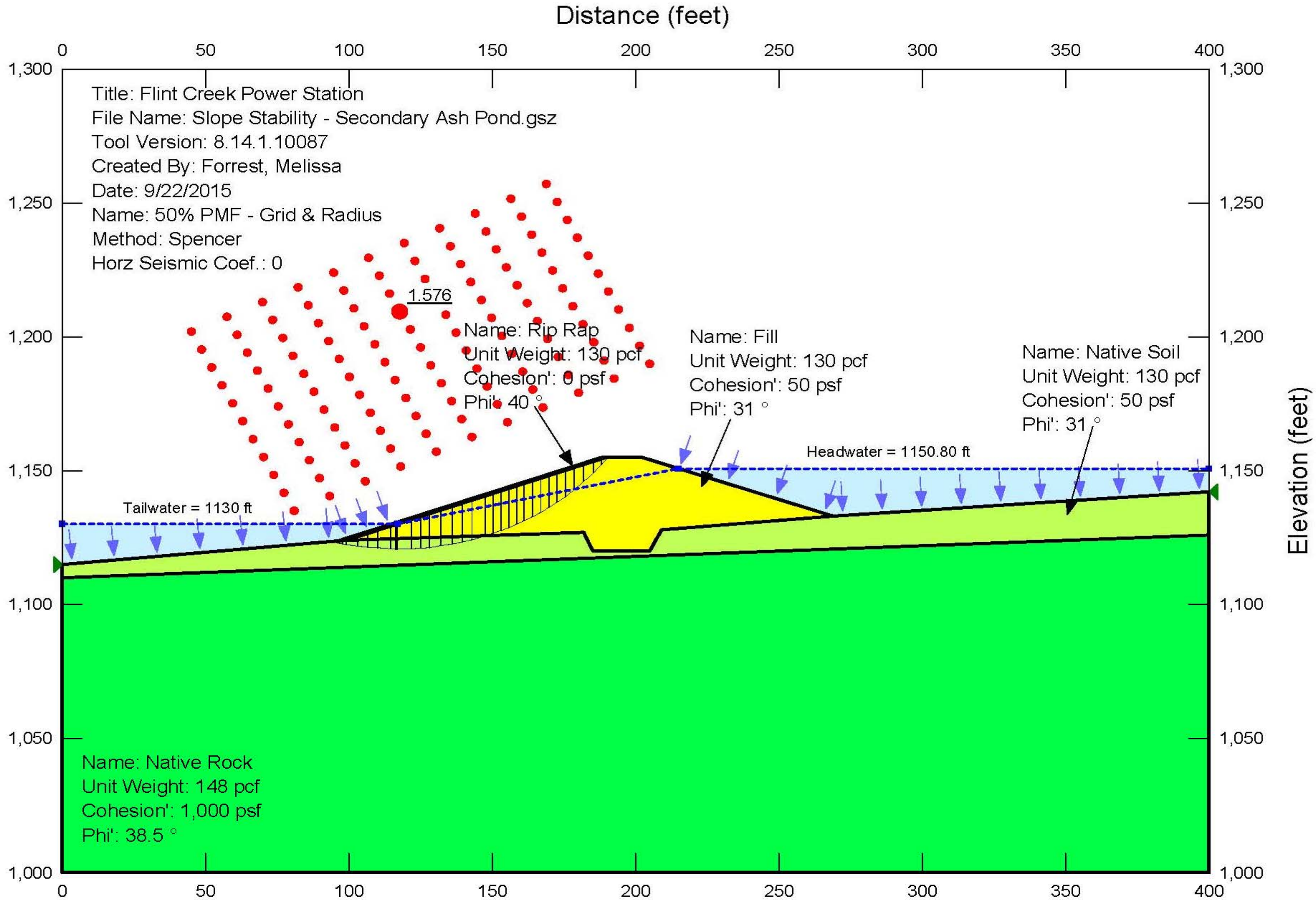
CCR Rule Structural Stability Certification
AEP Flint Creek Power Plant, Gentry, AR

Project No. 60437225

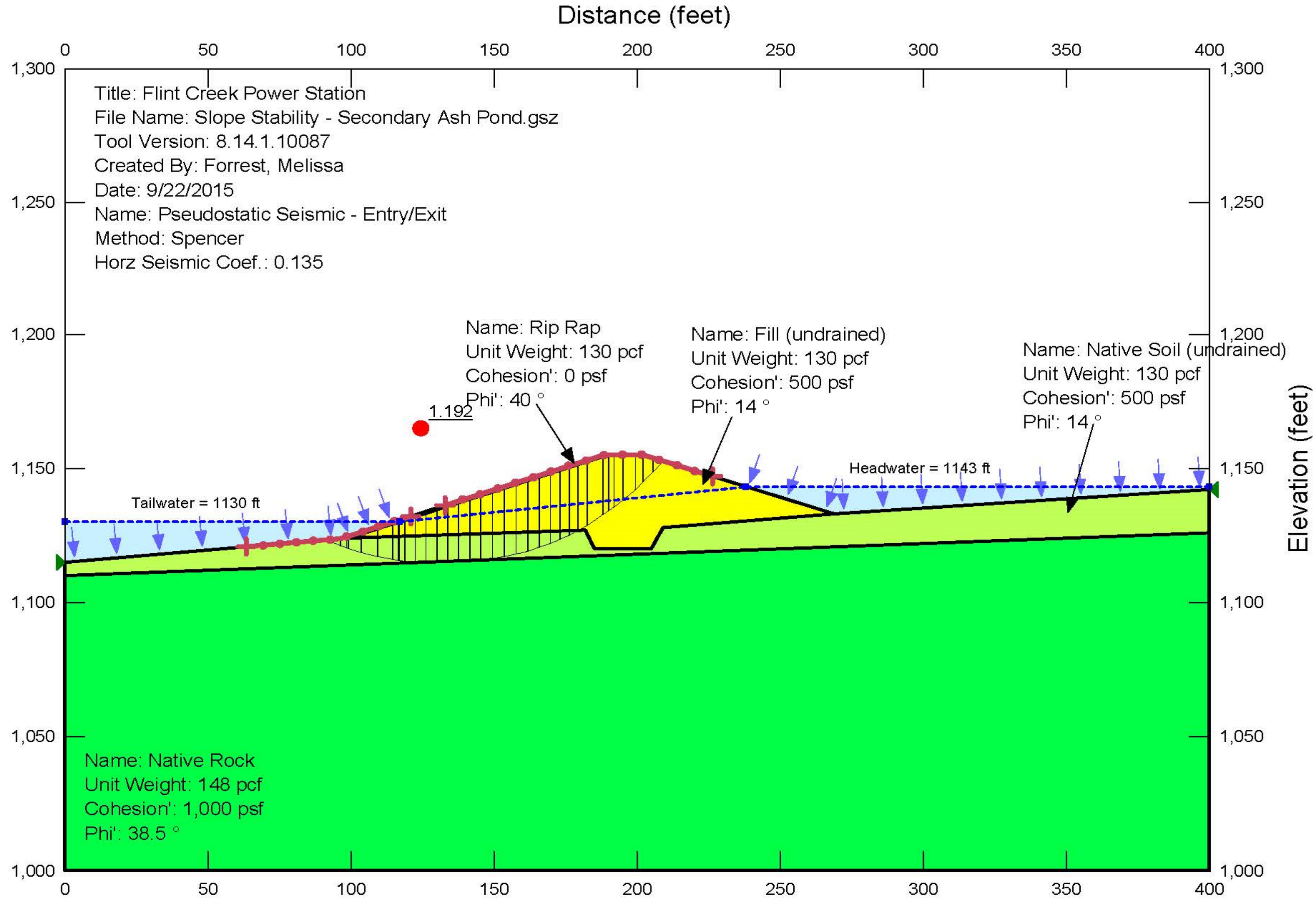
Date: December 2015

Secondary Ash Pond
Steady State
Max Storage Pool

Figure 7



AECOM	CCR Rule Structural Stability Certification AEP Flint Creek Power Plant, Gentry, AR	**Secondary Ash Pond** **Surcharge Pool (50% PMF)**	Figure 8
	Project No. 60437225 / Date: December 2015		



AECOM

CCR Rule Structural Stability Certification
AEP Flint Creek Power Plant, Gentry, AR

Project No. 60437225

Date: December 2015

Secondary Ash Pond
Pseudostatic Seismic

Figure 9

Appendix A

Development of Design Shear Strength

A. Objective

Develop Mohr-Coulomb drained and undrained strength properties for the embankment and residual soils at the Primary and Secondary Ash Ponds at the AEP Flint Creek plant in Benton County, Arkansas.

B. Procedure and Results

CIU triaxial tests were performed by ETTL, Incorporated, in 2009. The tests were performed on a total of 9 specimens (from three separate Shelby tubes). Two of the Shelby tubes were collected in the embankment fill, while one of the tubes was collected in the residual soils beneath the embankments. Shelby tubes of embankment soils were obtained in boring B-2 at the secondary pond and boring B-3 at the primary pond, while Shelby tubes of residual soils were only obtained in boring B-2 at the secondary pond. Additional samples were not collected due to the high gravel content in both the embankment and foundation soils, which caused difficulties in advancing and retrieving Shelby tubes.

The results of the CIU triaxial tests have been plotted by AECOM both p'-q and p-q plots. Failure was defined using the maximum stress difference criteria (σ_1-σ_3, or max deviator stress), as the ETTL report does not contain sufficient data to also define failure using the maximum ratio of principal effective stresses during the triaxial test (σ_1/σ_3, or maximum obliquity). Failure at max deviator stress was plotted as a single point, with the two different material types (fill and residuum) shown using different symbols. A review of the resulting plots found that the embankment fill and residuum soils all plotted in a consistent, relatively linear fashion, which indicates that the two materials have similar shear strengths. Therefore, a single set of design strengths were assigned for the combined materials.

For each plot, the design stress ratio at failure line (K_f) was then drawn through the p'-q and p-q plots to develop the Mohr-Coulomb shear strength properties. The K_f line is related to a normal ϕ and c failure envelope using $\sin\phi = \tan\Psi$ (Eqn. 10-24, Holtz & Kovacs, 1981).

Table 1 lists the design Mohr-Coulomb drained and undrained shear strength parameters, for both maximum deviator stress and maximum obliquity failure criteria.

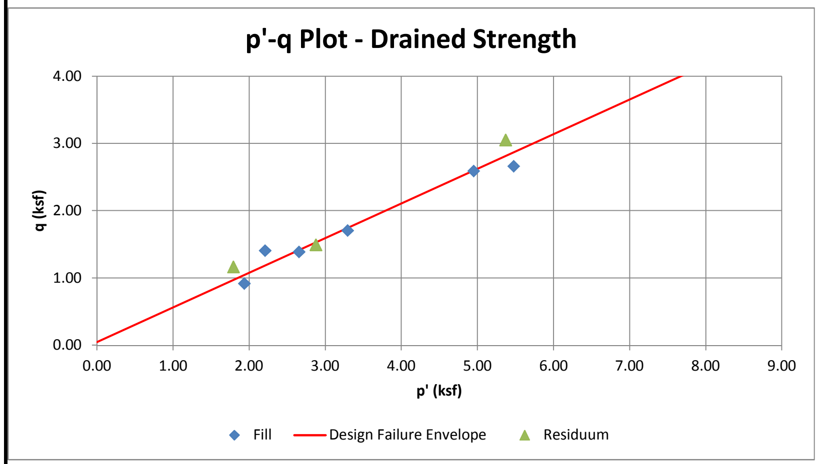
Table 1 – Residuum Strength Properties – Max Obliquity and Max Deviator Stress

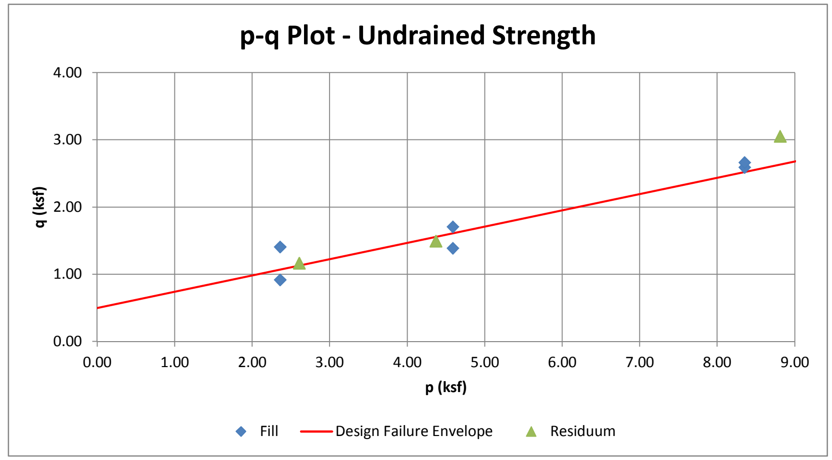
Material	Drained Strength		Undrained Strength	
	ϕ' (degrees)	**c' (psf)**	**ϕ (degrees)**	**c (psf)**
Embankment Fill and Residuum	31	50	14	500

Attachments

1. Test results and p-q plots
2. Laboratory testing forms from ETTL
3. Excerpts from Holtz and Kovacs (1981)

				Data Analysis - Maximum Deviator Stress Criteria							
				Total Stress		Effective		Total Stress		Effective	
Boring #	Test #	Sample #/Depth	Consolidation Stress (σ'_3), psi	σ_1, psi	σ_3, psi	σ'_1, psi	σ'_3, psi	p (ksf)	q (ksf)	p' (ksf)	q (ksf)
B-2	1	3-7	10	22.74	10.00	19.80	7.06	2.36	0.92	1.93	0.92
	2		20	43.73	20.00	34.74	11.01	4.59	1.71	3.29	1.71
	3		40	75.99	40.00	52.37	16.38	8.35	2.59	4.95	2.59
B-2	1	23-35	10	26.18	10.00	20.55	4.37	2.60	1.16	1.79	1.16
	2		20	40.70	20.00	30.34	9.64	4.37	1.49	2.88	1.49
	3		40	82.40	40.00	58.49	16.09	8.81	3.05	5.37	3.05
B-3	1	3-7	10	29.56	10.00	25.12	5.56	2.85	1.41	2.21	1.41
	2		20	39.31	20.00	28.08	8.77	4.27	1.39	2.65	1.39
	3		40	76.95	40.00	56.49	19.54	8.42	2.66	5.47	2.66



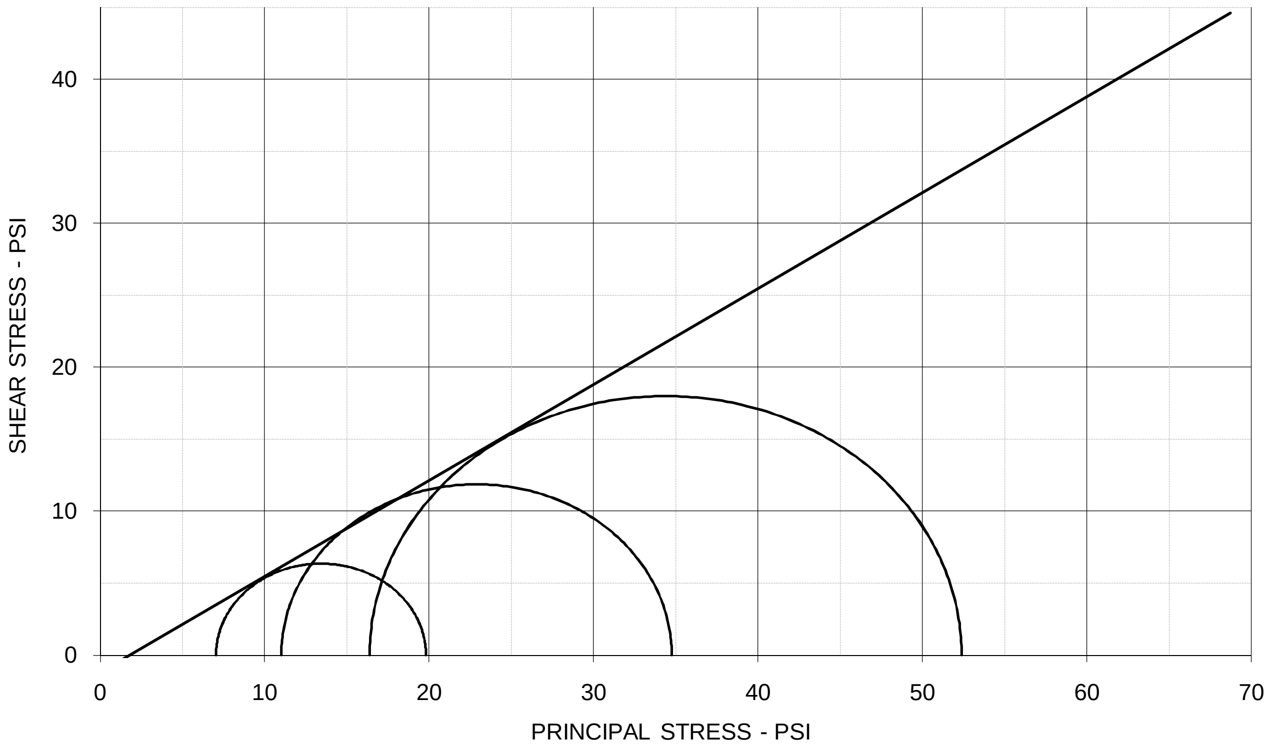


Effective Stress Failure Envelope		
c' =	50	psf
phi' =	31	deg

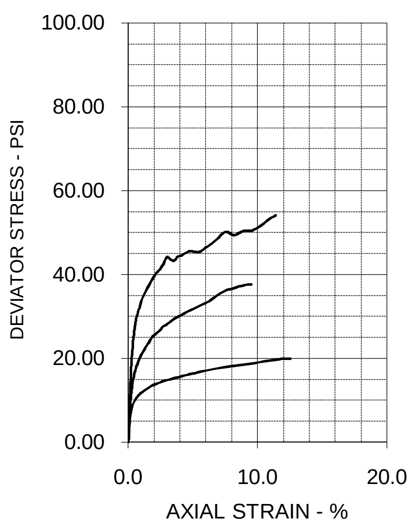
Total Stress Failure Envelope		
c =	500	psf
phi =	14	deg

Effective Stress Data - B-2, 3-7' Depth

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' = 33.7 deg	c' = -1.2 psi

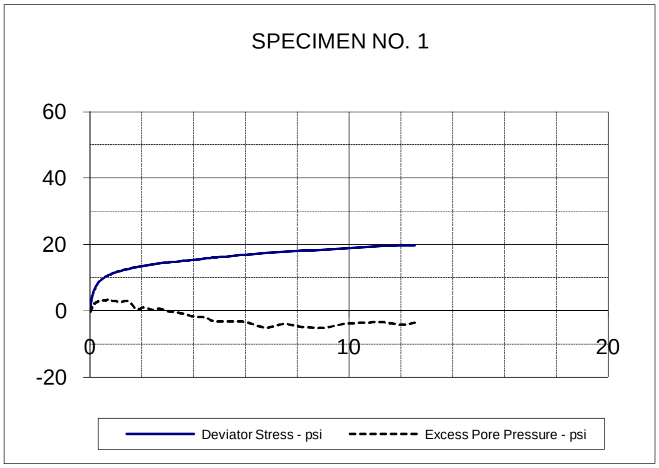


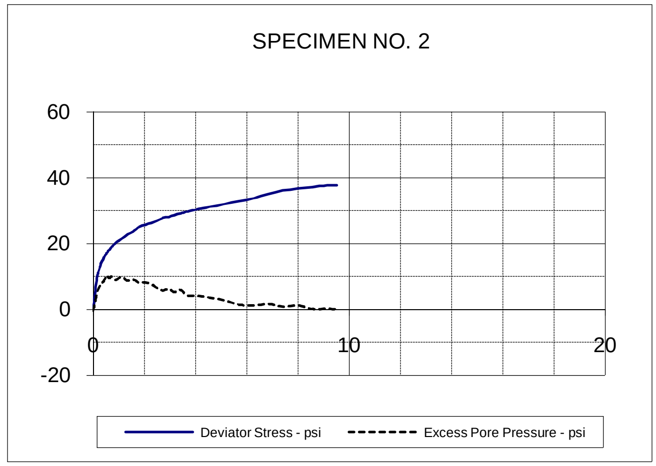
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	16.9	15.1	21.1	
Dry Density - pcf	108.9	113.4	107.0	
Diameter - inches	2.79	2.75	2.76	
Height - inches	5.68	4.33	5.19	
AT TEST				
Final Moisture - %	21.7	19.9	19.4	
Dry Density - pcf	109.4	114.8	109.2	
Calculated Diameter (in.)	2.79	2.74	2.73	
Height - inches	5.68	4.28	5.12	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.74	23.73	35.99	
Total Pore Pressure - psi	62.9	59.0	73.6	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	1.5	1.6	1.4	
σ_1' Failure - psi	19.80	34.74	52.37	
σ_3' Failure - psi	7.06	11.01	16.38	

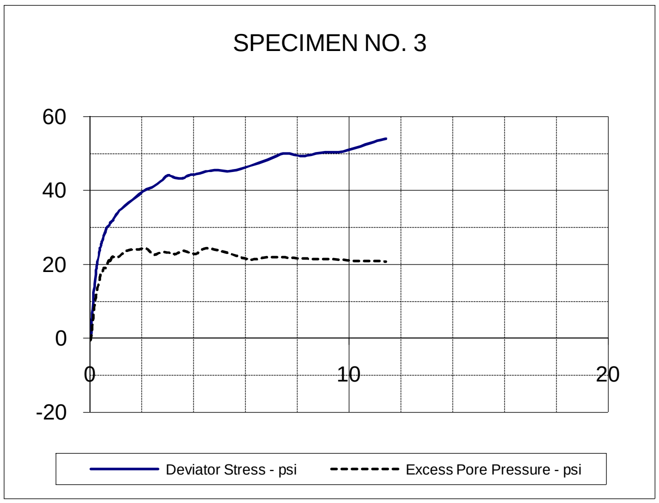
TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CU with PP	PROJECT: Flint Creek Power Plant
SAMPLE TYPE: Shelby Tube Sample	LOCATION: Centry, AR
DESCRIPTION: Redd. Brown & Tan & Gray Fat Clay w/ Gravel	PROJECT NO: G 3243 - 09
Sampled on Site, B-2 3' to 7' deep	CLIENT: AEP
ASSUMED SPECIFIC GRAVITY: 2.7 + 40 Sieve	December 2009
LL: PL: PI: Percent -200:	ETTL ENGINEERS & CONSULTANTS
REMARKS: Diameter and Both Ends Trimmed + # 4 Sieve	PLATE: B.1

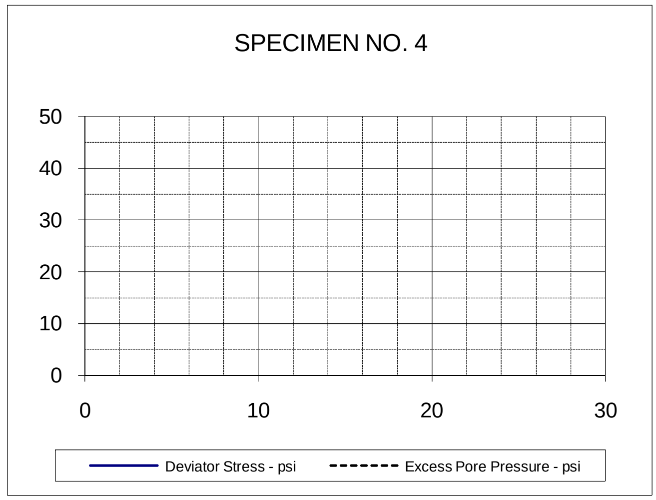
G 3243-09, B-2 3'-7' Flint Creek

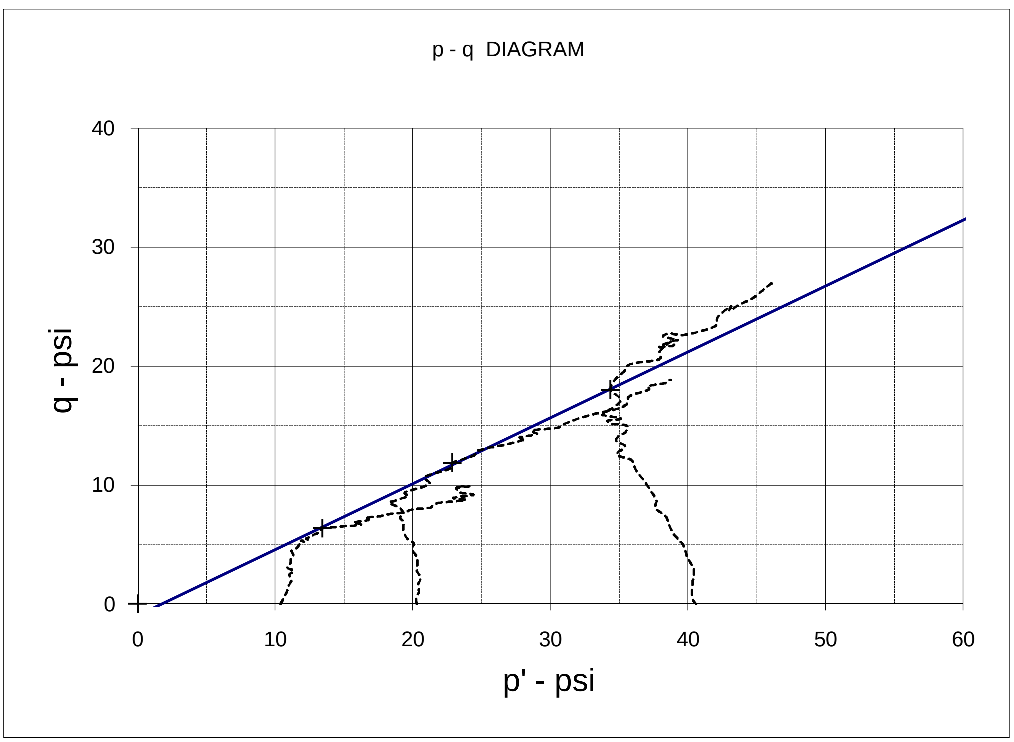
Effective Stress Data - B-2, 3-7' Depth







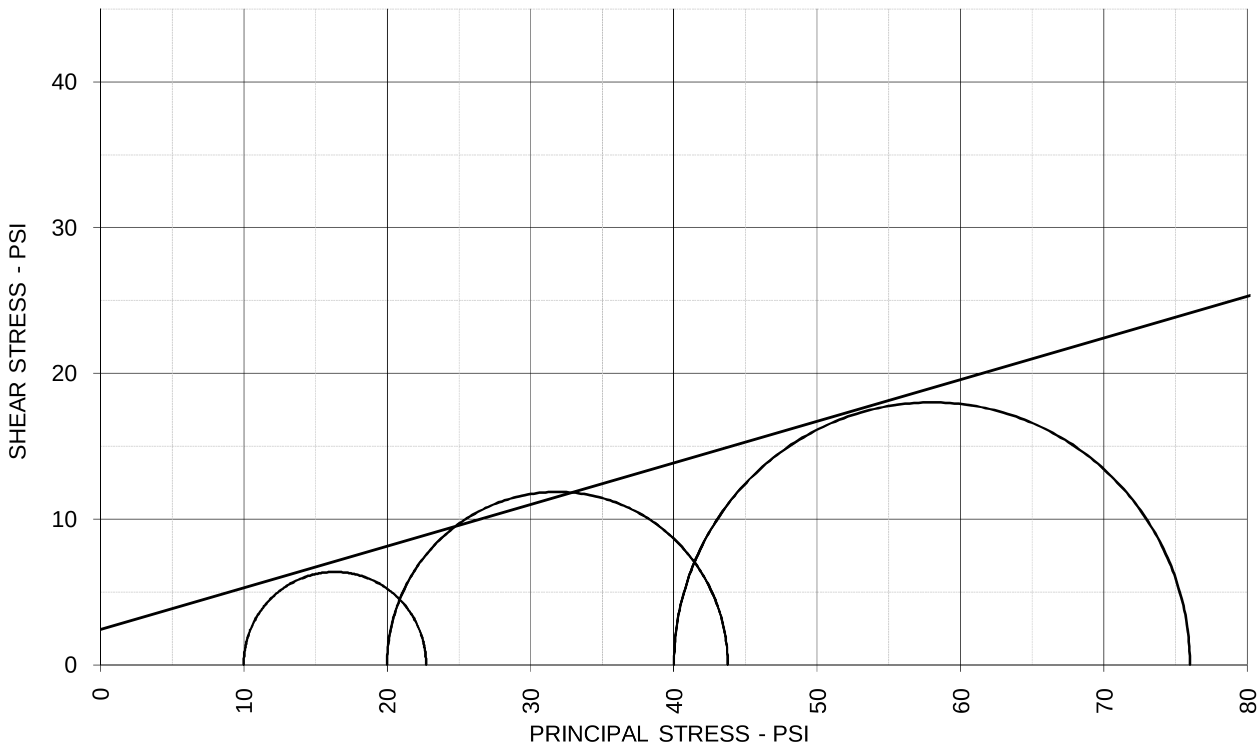




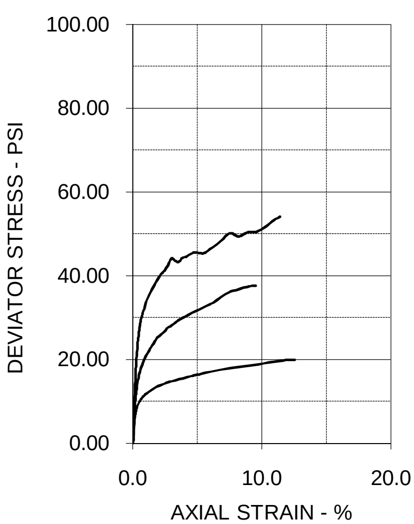
EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 29.0	a (psi) = -1.0
PROJECT: Flint Creek Power Plant		TYPE OF TEST & NO: CU with PP	
PROJECT NO: G 3243 - 09		**ETTL ENGINEERS & CONSULTANTS**	PLATE: B.2
DESCRIPTION: Redd. Brown & Tan & Gray Fat Clay w/ Gravel			

Total Stress Data - B-2, 3-7' Depth

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	φ = 15.9 deg	c = 2.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	16.9	15.1	21.1	
Dry Density - pcf	108.9	113.4	107.0	
Diameter - inches	2.79	2.75	2.76	
Height - inches	5.68	4.33	5.19	
AT TEST				
Final Moisture - %	21.7	19.9	19.4	
Dry Density - pcf	109.4	114.8	109.2	
Calculated Diameter (in.)	2.79	2.74	2.73	
Height - inches	5.68	4.28	5.12	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.74	23.73	35.99	
Total Pore Pressure - psi	62.9	59.0	73.6	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	1.5	1.6	1.4	
σ_1 Failure - psi	22.74	43.73	75.99	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

TYPE OF TEST & NO: CU with PP
SAMPLE TYPE: Shelby Tube Sample
DESCRIPTION: Redd. Brown & Tan & Gray Fat Clay w/ Gravel
Sampled on Site, B-2 3' to 7' deep
ASSUMED SPECIFIC GRAVITY: 2.7 + 40 Sieve
LL: PL: PI: Percent -200:
REMARKS: Diameter and Both Ends Trimmed + # 4 Sieve

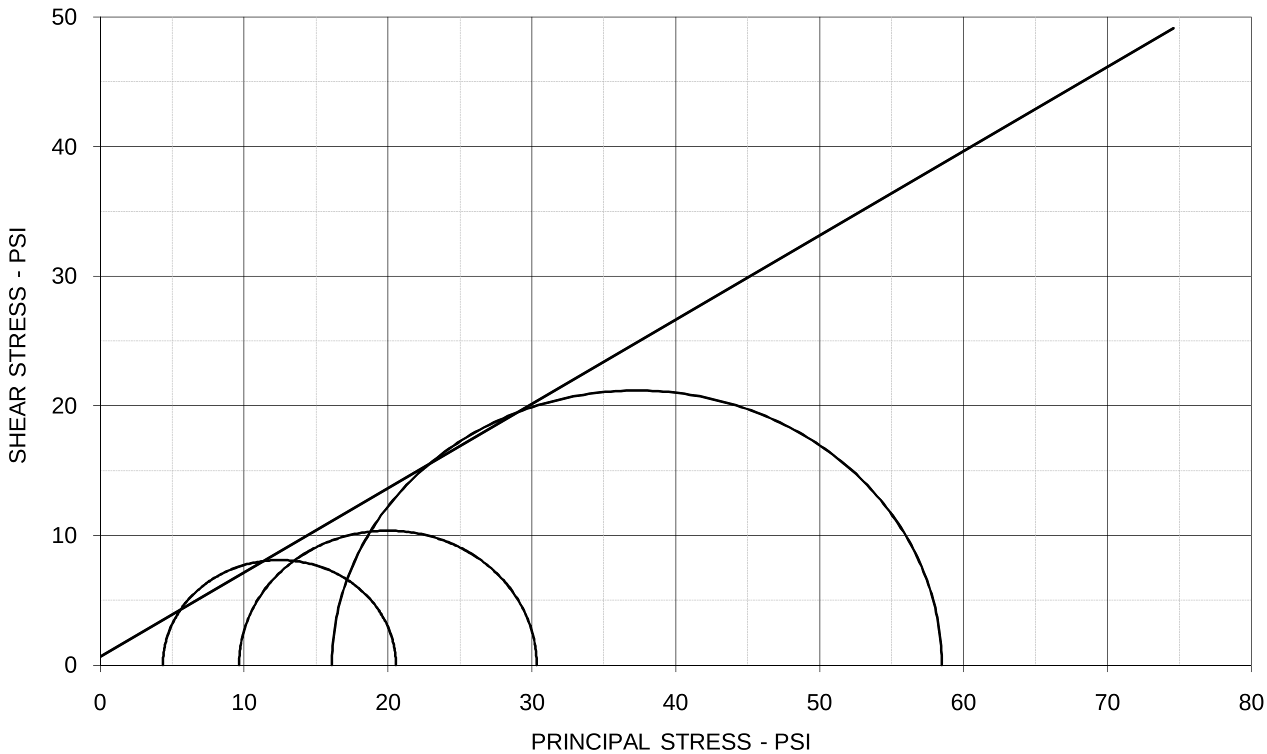
PROJECT INFORMATION

PROJECT: Flint Creek Power Plant
LOCATION: Centry, AR
PROJECT NO: G 3243 - 09
CLIENT: AEP
December 2009

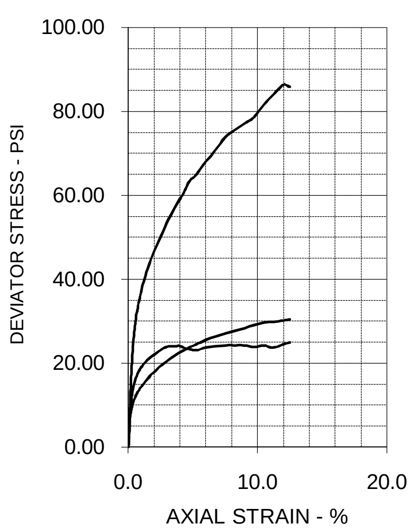
ETTL ENGINEERS & CONSULTANTS PLATE: B.3

Effective Stress Data - B-2, 23-25' Depth

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' = 33.0 deg	c' = 0.6 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	21.8	20.0	17.7	
Dry Density - pcf	103.5	109.2	114.4	
Diameter - inches	2.78	2.76	2.80	
Height - inches	5.68	5.67	5.69	
AT TEST				
Final Moisture - %	23.5	21.0	16.6	
Dry Density - pcf	103.8	110.3	117.0	
Calculated Diameter (in.)	2.77	2.74	2.78	
Height - inches	5.65	5.63	5.64	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	16.18	20.70	42.40	
Total Pore Pressure - psi	55.6	60.4	73.9	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	1.5	1.5	1.5	
σ_1' Failure - psi	20.55	30.34	58.49	
σ_3' Failure - psi	4.37	9.64	16.09	

TEST DESCRIPTION

TYPE OF TEST & NO: CU with PP

SAMPLE TYPE: Shelby Tube Sample

DESCRIPTION: Reddish Brown & Tan Lean Clay

Sampled on Site, B-2 23' to 35' deep

ASSUMED SPECIFIC GRAVITY: 2.7 + 40 Sieve

LL: PL: PI: Percent -200:

REMARKS: Diameter and Both Ends Trimmed + # 4 Sieve

G 3243-09, B-2 23'-35' Flint Creek

PROJECT INFORMATION

PROJECT: Flint Creek Power Plant

LOCATION: Centry, AR

PROJECT NO: G 3243 - 09

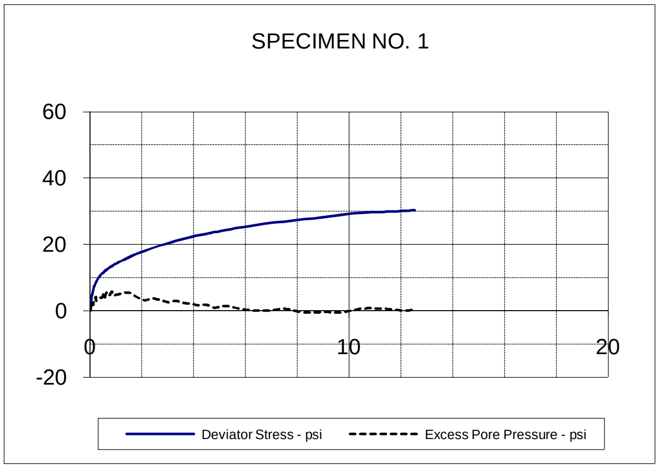
CLIENT: AEP

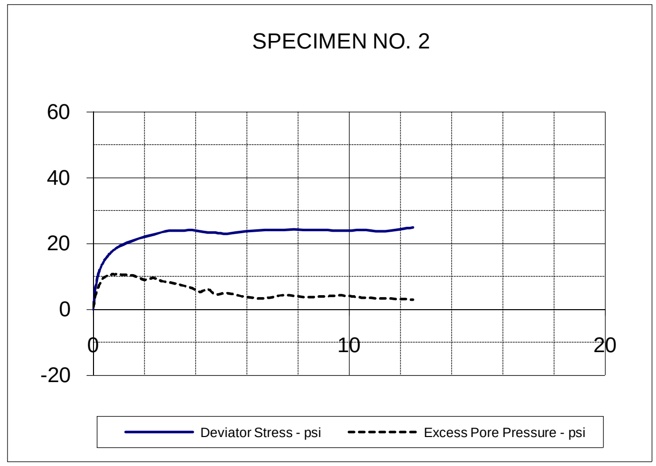
December 2009

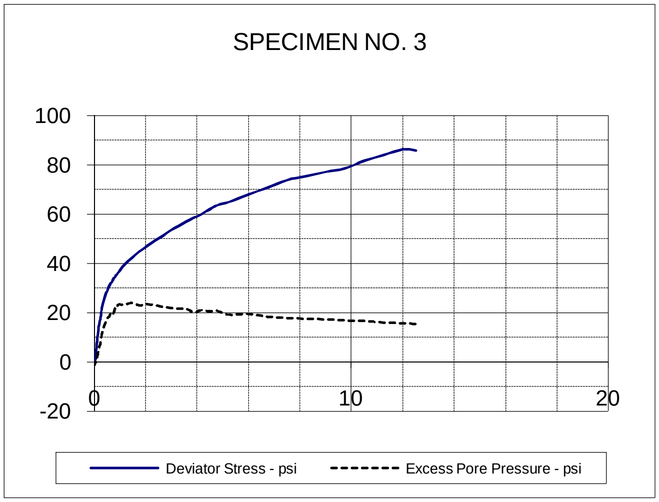
ETTL ENGINEERS & CONSULTANTS

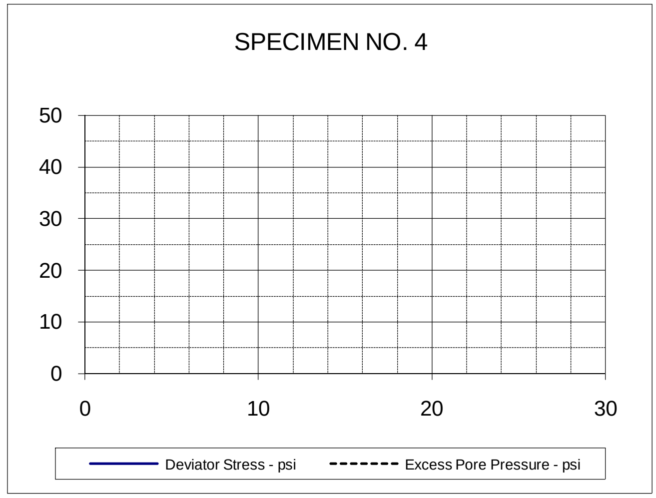
PLATE: B.1

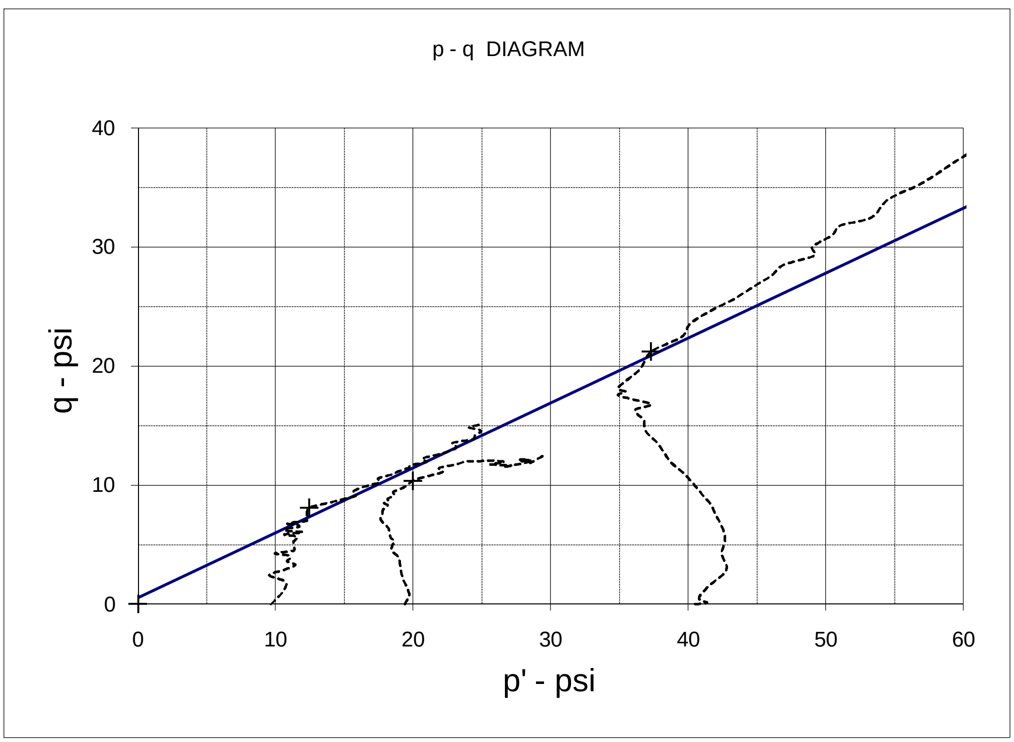
Effective Stress Data - B-2, 23-25' Depth









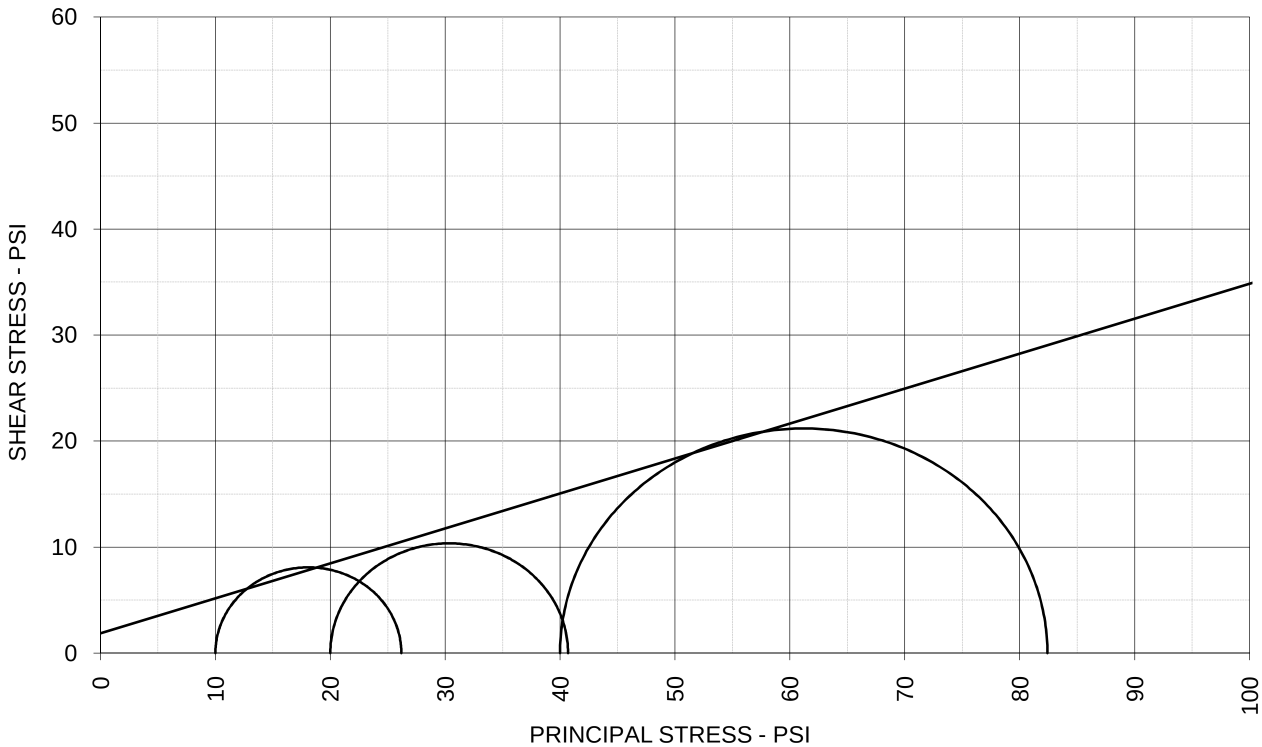


EFFECTIVE STRESS PARAMETERS	R^2 = 0.98	α (deg) = 28.6	a (psi) = 0.5
PROJECT: Flint Creek Power Plant		TYPE OF TEST & NO: CU with PP	
PROJECT NO: G 3243 - 09		**ETTL ENGINEERS & CONSULTANTS**	PLATE: B.2
DESCRIPTION: Reddish Brown & Tan Lean Clay			

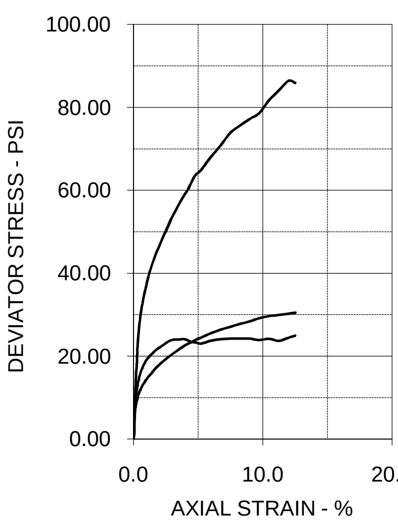
G 3243-09, B-2 23'-35' Flint Creek

Total Stress Data - B-2, 23-25' Depth

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 18.3 deg	c = 1.9 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	21.8	20.0	17.7	
Dry Density - pcf	103.5	109.2	114.4	
Diameter - inches	2.78	2.76	2.80	
Height - inches	5.68	5.67	5.69	
AT TEST				
Final Moisture - %	23.5	21.0	16.6	
Dry Density - pcf	103.8	110.3	117.0	
Calculated Diameter (in.)	2.77	2.74	2.78	
Height - inches	5.65	5.63	5.64	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	16.18	20.70	42.40	
Total Pore Pressure - psi	55.6	60.4	73.9	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	1.5	1.5	1.5	
σ_1 Failure - psi	26.18	40.70	82.40	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

TYPE OF TEST & NO: CU with PP

SAMPLE TYPE: Shelby Tube Sample

DESCRIPTION: Reddish Brown & Tan Lean Clay

Sampled on Site, B-2 23' to 35' deep

ASSUMED SPECIFIC GRAVITY: 2.7 + 40 Sieve

LL: PL: PI: Percent -200:

REMARKS: Diameter and Both Ends Trimmed + # 4 Sieve

PROJECT INFORMATION

PROJECT: Flint Creek Power Plant

LOCATION: Centry, AR

PROJECT NO: G 3243 - 09

CLIENT: AEP

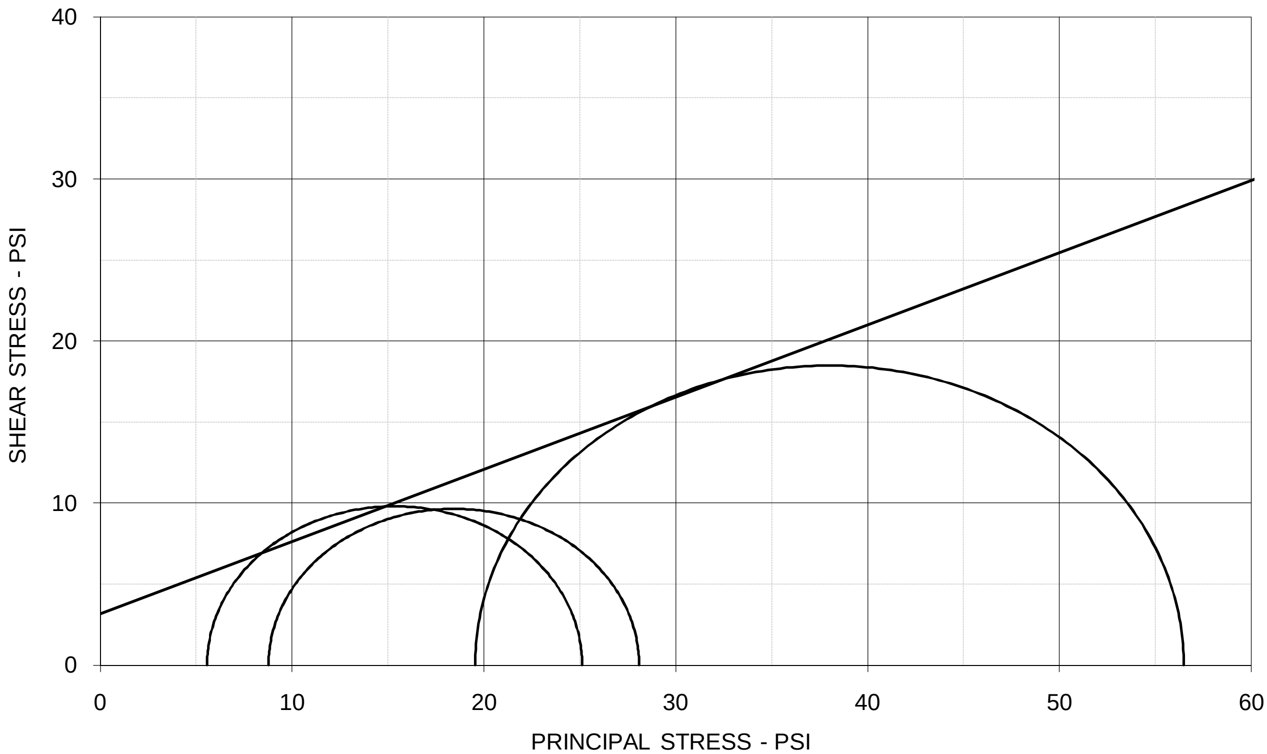
December 2009

ETTL ENGINEERS & CONSULTANTS

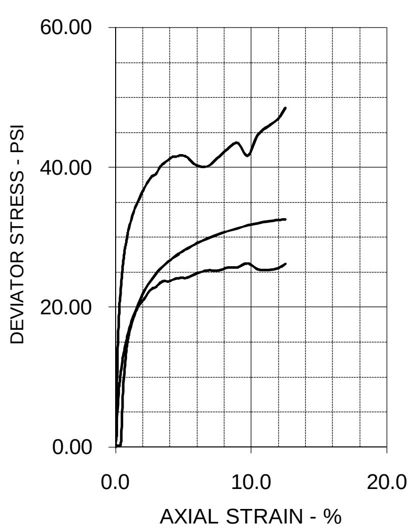
PLATE: B.3

Effective Stress Data - B-2, 23-25' Depth

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ ' = 24.0 deg		c' = 3.2 psi	
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	17.6	20.3	17.6	
Dry Density - pcf	107.9	106.2	107.7	
Diameter - inches	2.76	2.76	2.77	
Height - inches	5.68	5.68	5.68	
AT TEST				
Final Moisture - %	24.0	22.3	22.0	
Dry Density - pcf	108.5	107.0	109.8	
Calculated Diameter (in.)	2.76	2.75	2.75	
Height - inches	5.67	5.64	5.62	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	19.56	19.31	36.95	
Total Pore Pressure - psi	54.4	61.2	70.5	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	1.5	1.5	2.1	
σ_1' Failure - psi	25.12	28.08	56.49	
σ_3' Failure - psi	5.56	8.77	19.54	



TEST DESCRIPTION

TYPE OF TEST & NO: CU with PP

SAMPLE TYPE: Shelby Tube Sample

DESCRIPTION: Redd. Brown & Tan Sandy Lean Clay w/ Gravel

Sampled on Site, B-3 3' to 7' deep

ASSUMED SPECIFIC GRAVITY: 2.7 + 40 Sieve

LL: PL: PI: Percent -200:

REMARKS: Diameter and Both Ends Trimmed + # 4 Sieve

PROJECT INFORMATION

PROJECT: Flint Creek Power Plant

LOCATION: Centry, AR

PROJECT NO: G 3243 - 09

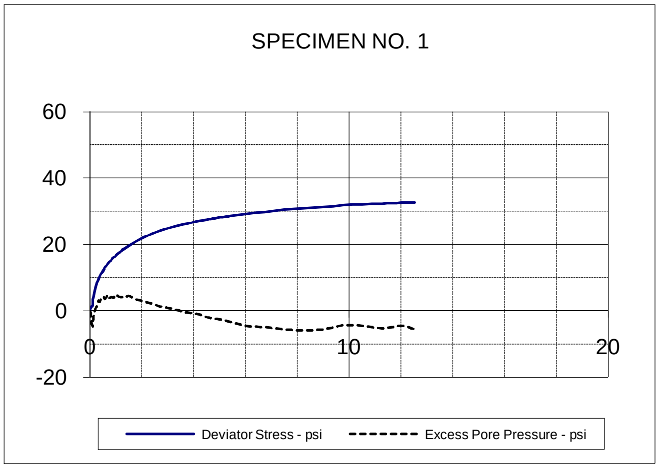
CLIENT: AEP

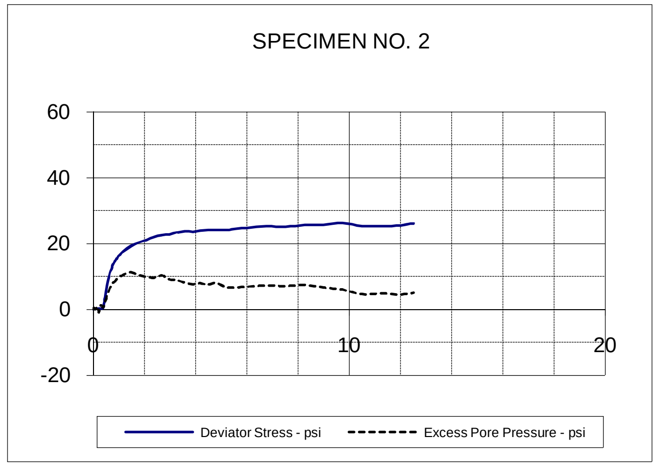
December 2009

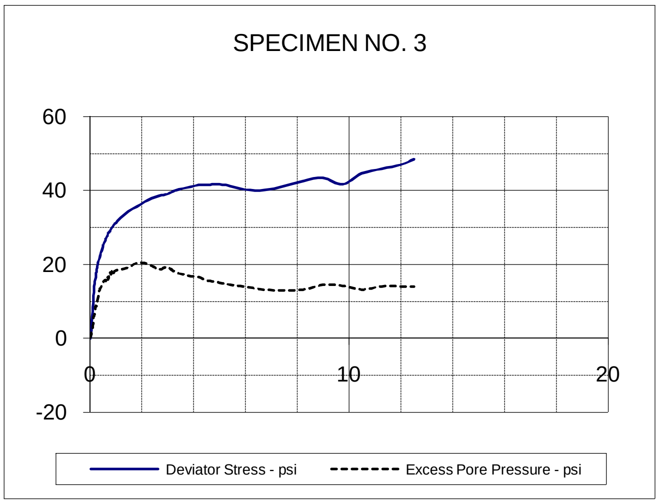
ETTL ENGINEERS & CONSULTANTS PLATE: B.1

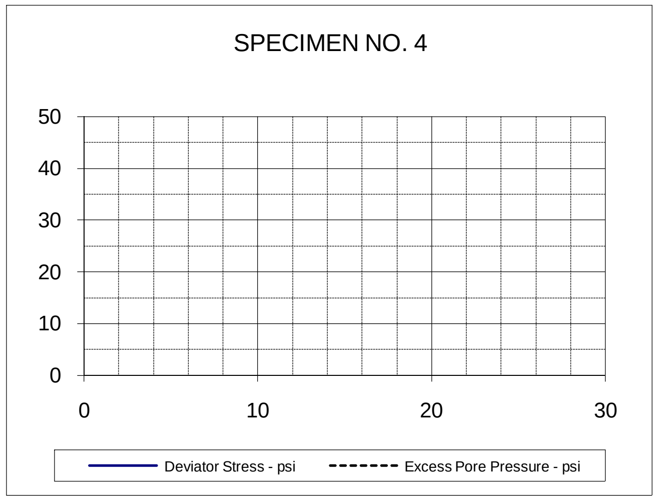
G 3243-09, B-3 5'-7' Flint Creek

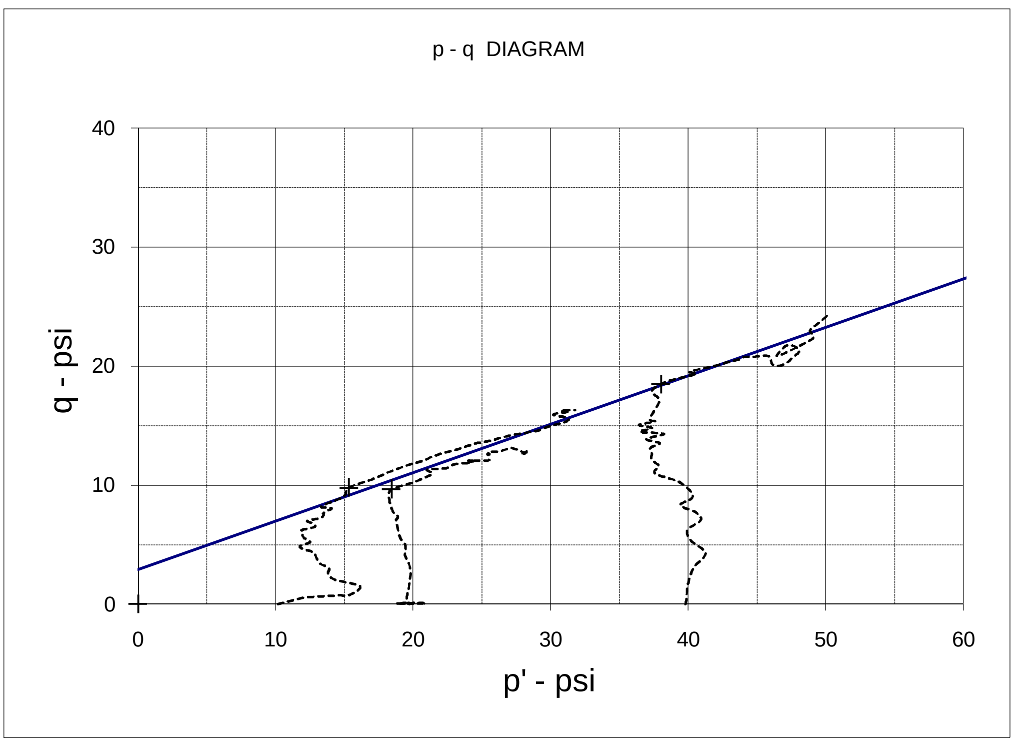
Effective Stress Data - B-2, 23-25' Depth









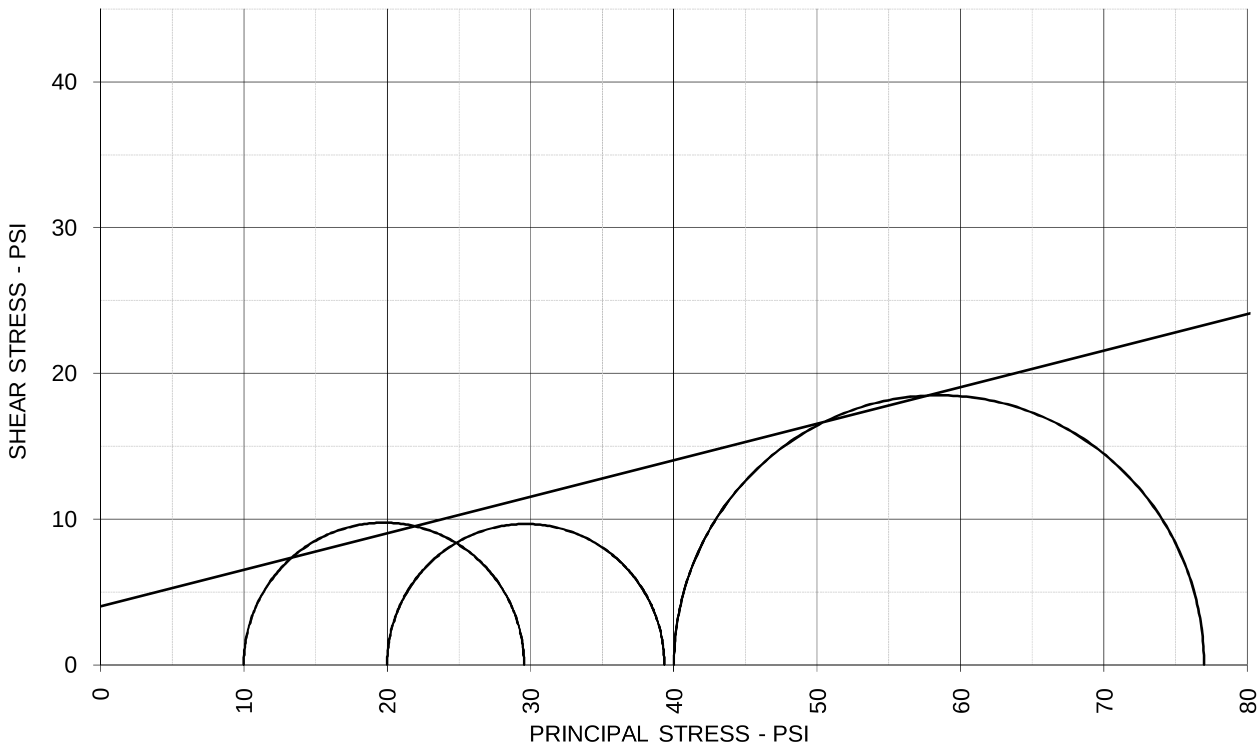


EFFECTIVE STRESS PARAMETERS	R^2 = 0.98	α (deg) = 22.2	a (psi) = 2.9
PROJECT: Flint Creek Power Plant		TYPE OF TEST & NO: CU with PP	
PROJECT NO: G 3243 - 09		**ETTL ENGINEERS & CONSULTANTS**	PLATE: B.2
DESCRIPTION: Redd. Brown & Tan Sandy Lean Clay w/ Gravel			

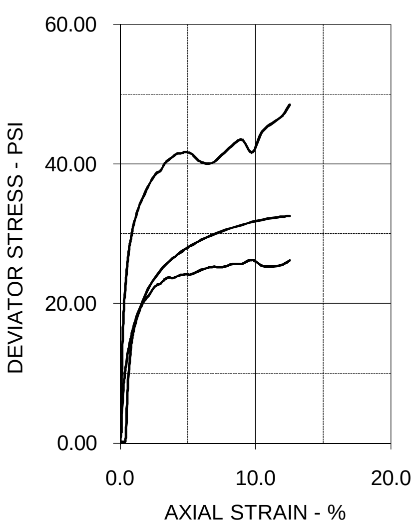
G 3243-09, B-3 5'-7' Flint Creek

Total Stress Data - B-2, 23-25' Depth

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 14.1 deg		c = 4.0 psi	
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	17.6	20.3	17.6	
Dry Density - pcf	107.9	106.2	107.7	
Diameter - inches	2.76	2.76	2.77	
Height - inches	5.68	5.68	5.68	
AT TEST				
Final Moisture - %	24.0	22.3	22.0	
Dry Density - pcf	108.5	107.0	109.8	
Calculated Diameter (in.)	2.76	2.75	2.75	
Height - inches	5.67	5.64	5.62	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	19.56	19.31	36.95	
Total Pore Pressure - psi	54.4	61.2	70.5	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	1.5	1.5	2.1	
σ_1 Failure - psi	29.56	39.31	76.95	
σ_3 Failure - psi	10.00	20.00	40.00	



TEST DESCRIPTION

TYPE OF TEST & NO: CU with PP
SAMPLE TYPE: Shelby Tube Sample
DESCRIPTION: Redd. Brown & Tan Sandy Lean Clay w/ Gravel
Sampled on Site, B-3 3' to 7' deep
ASSUMED SPECIFIC GRAVITY: 2.7 + 40 Sieve
LL: PL: PI: Percent -200:
REMARKS: Diameter and Both Ends Trimmed + # 4 Sieve

PROJECT INFORMATION

PROJECT: Flint Creek Power Plant
LOCATION: Centry, AR
PROJECT NO: G 3243 - 09
CLIENT: AEP
December 2009

ETTL ENGINEERS & CONSULTANTS

PLATE: B.3

T

Holtz & Kovacs Reference

PRENTICE-HALL CIVIL ENGINEERING AND ENGINEERING MECHANICS SERIES

N. M. Newmark and W. J. Hall, Editors

An Introduction to Geotechnical Engineering

ROBERT D. HOLTZ, PH.D., P.E.
University of Washington

WILLIAM D. KOVACS, PH.D., P.E.
University of Rhode Island

Holtz & Kovacs Reference

failure depends on the field loading conditions one wishes to model. Four common field conditions and the laboratory stress paths which model them are shown in Fig. 10.22. Note that these stress paths are for *drained* loading (discussed in the next chapter) in which there is *no* excess pore water pressure; therefore total stresses equal effective stresses and the total stress path (TSP) for a given loading is identical to the effective stress path (ESP).

As suggested by Eq. 10-20, we are often interested in conditions at failure, and it is useful to know the relationship between the K_f line and the Mohr-Coulomb failure envelope. Consider the two Mohr circles shown in Fig. 10.23. The circle on the left, drawn for illustrative purposes only, represents failure in terms of the p-q diagram. The identical circle on the right is the same failure circle on the Mohr τ-σ diagram. To establish the slopes of the two lines and their intercepts, several Mohr circles and stress paths, determined over a range of stresses, were used. The equation of the K_f line is

$$q_f = a + p_f \tan\psi \tag{10-23}$$

where a = the intercept on the q-axis, in stress units, and
ψ = the angle of the K_f line with respect to the horizontal, in degrees.

The equation of the Mohr-Coulomb failure envelope is

$$\tau_{ff} = c + \sigma_{ff} \tan\phi \tag{10-9}$$

From the geometries of the two circles, it can be shown that

$$\sin\phi = \tan\psi \tag{10-24}$$

and

$$c = \frac{a}{\cos\phi} \tag{10-25}$$

So, from a p-q diagram the shear strength parameters ϕ and c may readily be computed.

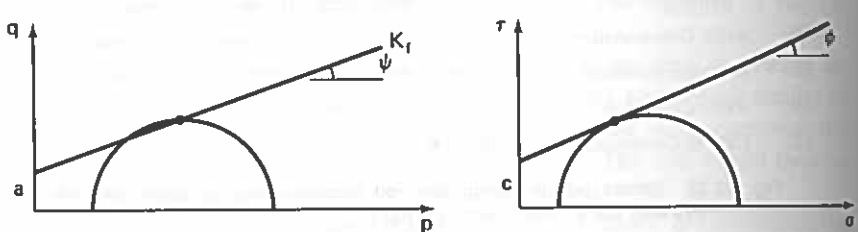


Fig. 10.23 Relationship between the K_f line and the Mohr-Coulomb failure envelope.

Another useful aspect of the p-q diagram is that it may be used to show both total and effective stress paths on the same diagram. We said before that for drained loading, the total stress path (TSP) and the effective stress path (ESP) were identical. This is because the pore water pressure induced by loading was approximately equal to zero at all times during shear. However, in general, during *undrained* loading the TSP is not equal to the ESP because excess pore water pressure develops. For axial compression (AC) loading of a normally consolidated clay ($K_o < 1$), a *positive* excess pore water pressure Δu develops. Therefore the ESP lies to the *left* of the TSP because $\sigma' = \sigma - \Delta u$. At any point during the loading, the pore water pressure Δu may be scaled off any horizontal line between the TSP and ESP, as shown in Fig. 10.24.

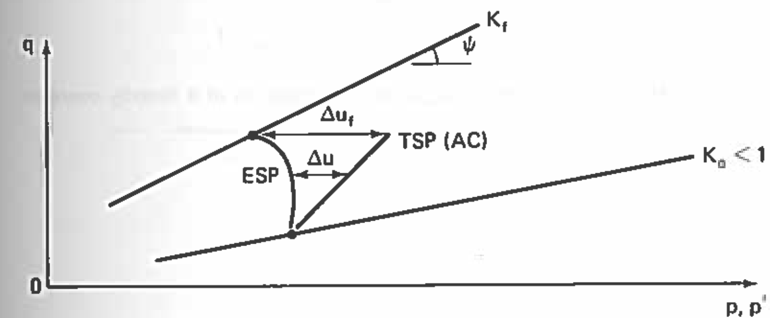


Fig. 10.24 Stress paths during undrained axial compression loading of a normally consolidated clay.

If a clay is overconsolidated ($K_o > 1$), then *negative* pore water pressure ($-\Delta u$) develops because the clay *tends* to expand during shear, but it can't. (Remember: we are talking about undrained loading in which no volume change is allowed.) For AC loading on an overconsolidated clay, stress paths like those shown in Fig. 10.25 will develop. Similarly, we can plot total and effective stress paths for other types of loadings and unloadings, for both normally and overconsolidated soils, and we shall show some of these in Chapter 11.

In most practical situations in geotechnical engineering, there exists a static ground water table; thus an initial pore water pressure u_o, is acting on the element in question. So there are really three stress paths we should consider, the ESP, the TSP, and the $(T - u_o)$SP. These three paths are shown in Fig. 10.26 for a normally consolidated clay with an initial pore water pressure u_o undergoing AC loading. Note that as long as the ground water table remains at the same elevation, u_o does not affect either the ESP or the conditions at failure.

Appendix B

Pseudostatic Coefficient Reference Material

Project Name:	Flint Creek Power Station, Existing Ash Storage Ponds Embankment		
Project Number:	60437225		
Client:			
Description:	Site Classifications		
By:	MF	**Checked By:**	JD
Date:	1-Sep-15	**Date:**	1-Sep-15

Task:

Evaluate the site classification based on the average blow count, Ñ, in the upper 100 feet of the soil profile.

Reference:

ASCE (2013). Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10)

Site Class Definitions:

Chapter 20 Site Classficationm Procedure for Seismic Design; Table 20.3-1

Average Blow Count, Ñ	Average Soil Shear Wave Velocity, V_s (feet/sec)	Site Class	Soil Profile Name
N/A	$V_s > 5000$	A	Hard rock
N/A	$2500 < Vs \leq 5000$	B	Rock
Ñ > 50	$1200 < Vs \leq 2500$	C	Very dense soil and soft rock
15 ≤ Ñ ≤ 50	$600 < Vs \leq 1200$	D	Stiff soil profile
Ñ < 15	$Vs < 600$	E	Soft soil profile

General Site Data from Boring Logs:

Reference: SPT data from B-1 through B-7

Selected most critical soil profile where fill layer is the thickest

Soil Type	Average Layer Thickness (ft)	Average Blow Count
Fill	20	19
Native Soil	20	28
Weathered Limestone	60	50
	= 100	

Evaluation of Average Blow Count, Ñ:

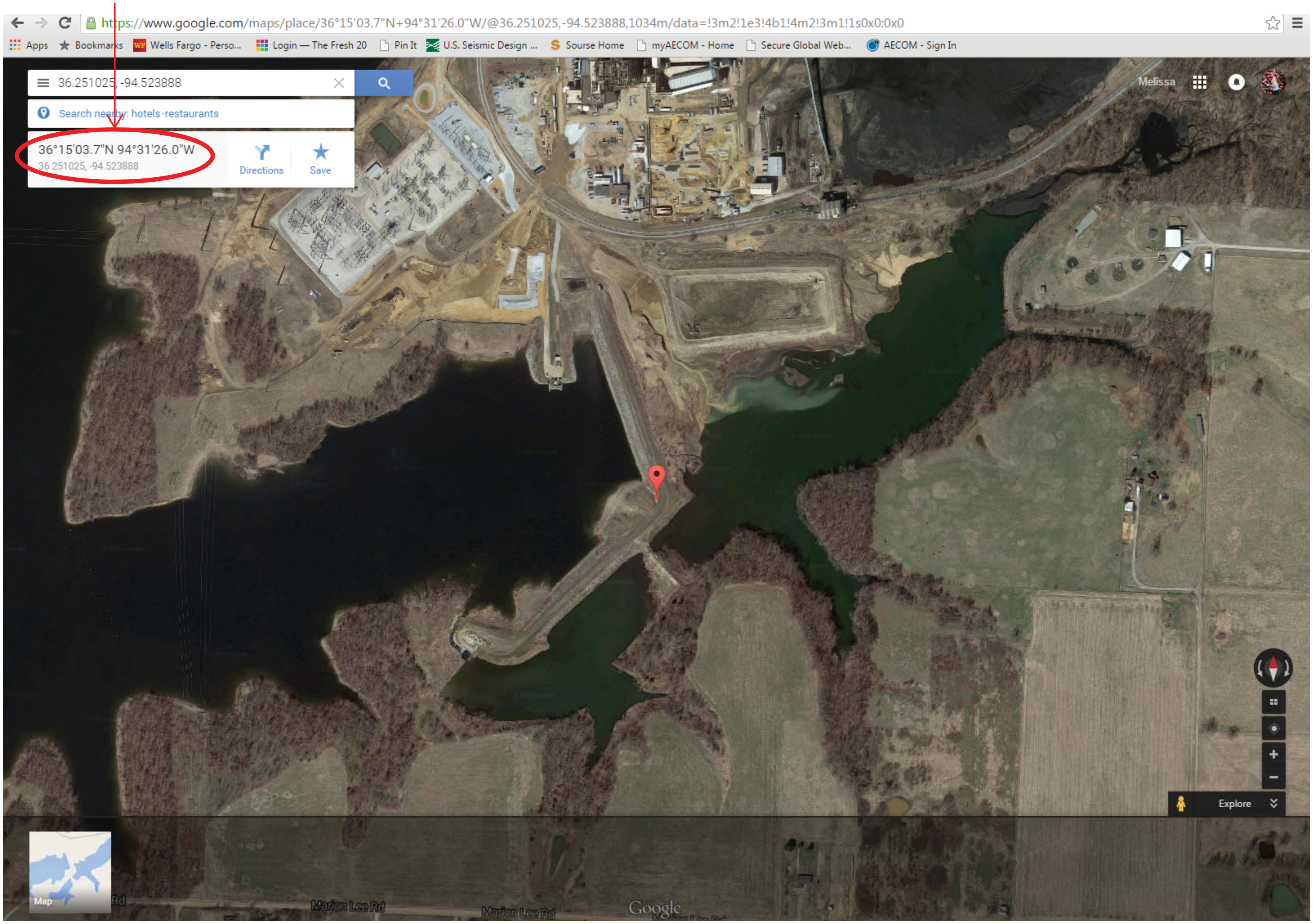
$$\tilde{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Ñ =	39

Soil Classification Recommendation:

D	Stiff Soil Profile

Approximate site coordinates



Design Maps Detailed Report

ASCE 7-10 Standard (36.25103°N, 94.52389°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 0.150 g
From Figure 22-2 [2]	S_1 = 0.085 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

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

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.150 g, F_a = 1.600

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.085 g, F_v = 2.400

Equation (11.4–1): $S_{MS} = F_aS_S = 1.600 \times 0.150 = 0.240$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 2.400 \times 0.085 = 0.205$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

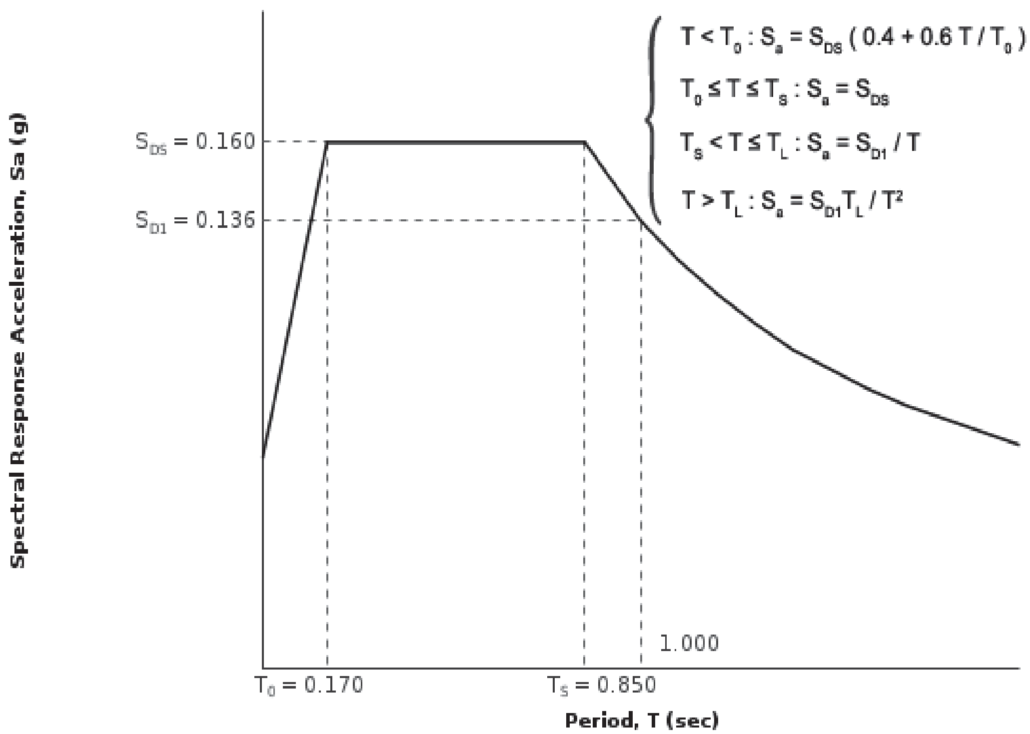
Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 0.240 = 0.160$ g

Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.205 = 0.136$ g

Section 11.4.5 — Design Response Spectrum

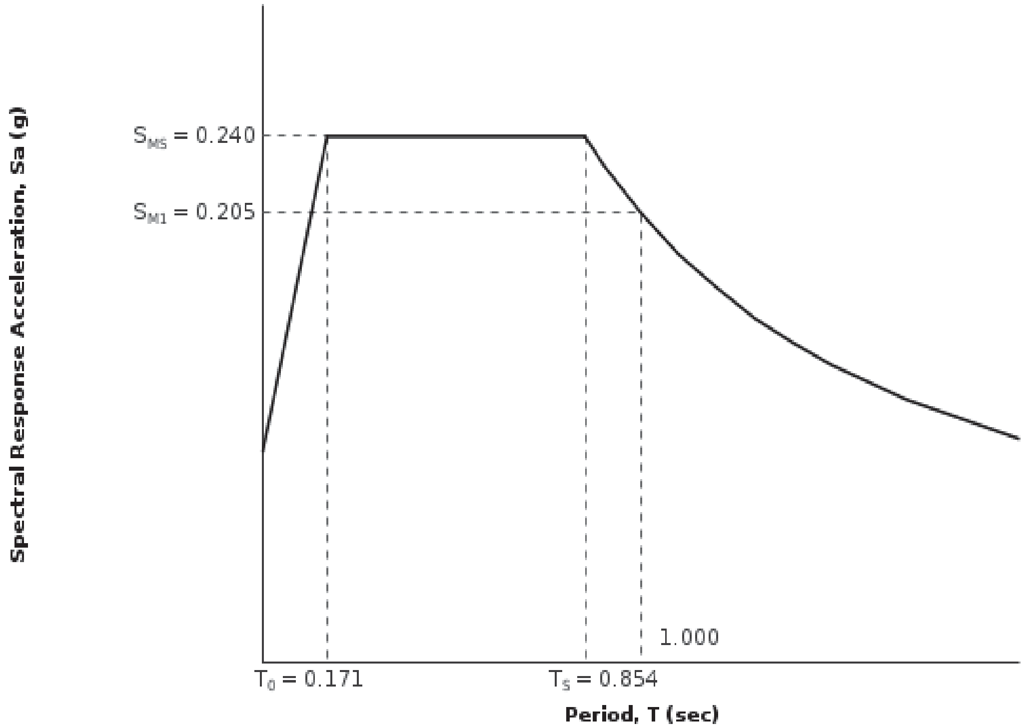
From Figure 22-12 [3] $T_L = 12$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.072

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.600 \times 0.072 = 0.115$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.072 g, F_{PGA} = 1.600

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.872

From Figure 22-18 [6] C_{R1} = 0.841

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.160 g, Seismic Design Category = A

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.136 g, Seismic Design Category = C

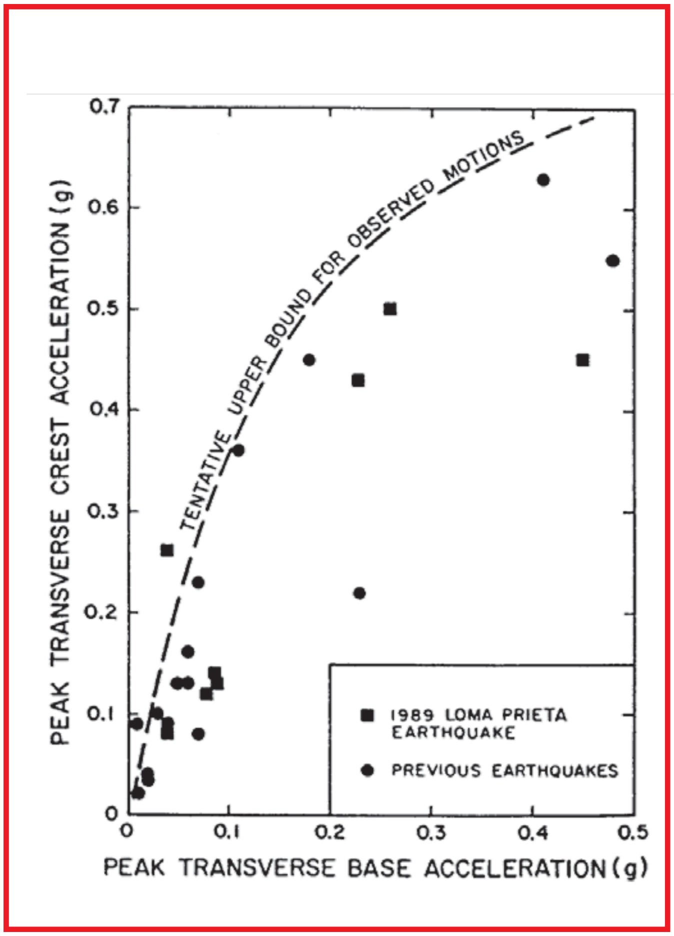
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = C

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf



FHWA, (2011). *LRFD Seismic Analysis and Design Transportation Geotechnical Features and Structural Foundations - Reference Manual,* NHI Course No. 130094, FHWA-NHI-11-032, GEC No. 3, August (Rev. 1).

Figure 5-5 Base and Crest Peak Accelerations Recorded at the Earth Dams (Harder, 1991)

The free field amplification curves presented in Figure 5-3 and Figure 5-4 may be used in a simplified three-step site response analysis procedure to account for the influence of local soil conditions on the peak ground acceleration from a conventional seismic hazard analysis (i.e. a seismic hazard analysis for Site Class B ground conditions) for PGA values less than or equal to 0.5. The observational data presented in Figure 5-5 may be used in a fourth step to account for the influence of an embankment on the transverse peak acceleration at the crest of the embankment. The procedure is as follows:

Step 1: *Evaluate the free field bedrock acceleration at the site for NEHRP/AASHTO Site Class B.* Determine the PGA from a conventional seismic hazard analysis for NEHRP/ASHTO Site Class B.

Step 2: *Classify the site according to the NEHRP/AASHTO site classification system.* Using Table 3-5, classify the site on the basis of the average shear wave velocity for the top 100 ft (30 meters) of soil, Vs_{30}.

Kramer, S.L. (1996). Geotechnical Earthquake Engineering, Prentice Hall Upper Saddle River, NJ

Resisting moment:

Section	Length (ft)	c (lb/ft^2)	Force (kips)	Moment Arm (ft)	Moment (kip-ft/ft)
A	11.5	600	6.9	78	538.2
B	129.3	1000	129.3	78	10,085.4
					10,623.6

Factor of safety:

$$\text{Static FS} = \frac{\text{resisting moment}}{\text{static overturning moment}} = \frac{10{,}623.6}{5925.5} = 1.79$$

$$\text{Pseudostatic FS} = \frac{\text{resisting moment}}{\text{static + pseudostatic overturning moments}}$$

$$= \frac{10{,}623.6}{8281.1} = 1.28$$

Selection of Pseudostatic Coefficient. The results of pseudostatic analyses are critically dependent on the value of the seismic coefficient, k_h. Selection of an appropriate pseudostatic coefficient is the most important, and most difficult, aspect of a pseudostatic stability analysis. The seismic coefficient controls the pseudostatic force on the failure mass, so its value should be related to some measure of the amplitude of the inertial force induced in the potentially unstable material. If the slope material was rigid, the inertial force induced on a potential slide would be equal to the product of the actual horizontal acceleration and the mass of the unstable material. This inertial force would reach its maximum value when the horizontal acceleration reached its maximum value. In recognition of the fact that actual slopes are not rigid and that the peak acceleration exists for only a very short time, the pseudostatic coefficients used in practice generally correspond to acceleration values well below a_{max}. Terzaghi (1950) originally suggested the use of $k_h = 0.1$ for "severe" earthquakes (Rossi-Forel IX), $k_h = 0.2$ for "violent, destructive" earthquakes (Rossi-Forel X), and $k_h = 0.5$ for "catastrophic" earthquakes. Seed (1979) listed pseudostatic design criteria for 14 dams in 10 seismically active countries; 12 required minimum factors of safety of 1.0 to 1.5 with pseudostatic coefficients of 0.10 to 0.12. Marcuson (1981) suggested that appropriate pseudostatic coefficients for dams should correspond to one-third to one-half of the maximum acceleration, including amplification or deamplification effects, to which the dam is subjected. Using shear beam models, Seed and Martin (1966) and Dakoulas and Gazetas (1986) showed that the inertial force on a potentially unstable slope in an earth dam depends on the response of the dam and that the average seismic coefficient for a deep failure surface is substantially smaller than that of a failure surface that does not extend far below the crest. Seed (1979) also indicated that deformations of earth dams constructed of ductile soils (defined as those that do not generate high pore pressures or show more than 15% strength loss upon cyclic loading) with crest accelerations less than $0.75g$ would be acceptably small for pseudostatic factors of safety of at least 1.15 with $k_h = 0.10$ ($M = 6.5$) to $k_h = 0.15$ ($M = 8.25$). This criteria would allow the use of pseudostatic accelerations as small as 13 to 20% of the peak crest acceleration. Hynes-Griffin and Franklin (1984) applied the Newmark sliding block analysis described in the following section to over 350 accelerograms and concluded that earth dams with pseudostatic factors of safety greater than 1.0 using $k_h = 0.5a_{max}/g$ would not develop "dangerously large" deformations.

As the preceding discussion indicates, there are no hard and fast rules for selection of a pseudostatic coefficient for design. It seems clear, however, that the pseudostatic coefficient should be based on the actual anticipated level of acceleration in the failure mass (including any amplification or deamplification effects) and that it should correspond to some fraction of the anticipated peak acceleration. Although engineering judgment is required for all cases, the criteria of Hynes-Griffin and Franklin (1984) should be appropriate for most slopes.

Limitations of the Pseudostatic Approach. Representation of the complex, transient, dynamic effects of earthquake shaking by a single constant unidirectional pseudostatic acceleration is obviously quite crude. Even in its infancy, the limitations of the pseudostatic approach were clearly recognized. Terzaghi (1950) stated that "the concept it conveys of earthquake effects on slopes is very inaccurate, to say the least," and that a slope could be unstable even if the computed pseudostatic factor of safety was greater than 1. Detailed analyses of historical and recent earthquake-induced landslides (e.g., Seed et al., 1969, 1975; Marcuson et al., 1979) have illustrated significant shortcomings of the pseudostatic approach. Experience has clearly shown, for example, that pseudostatic analyses can be unreliable for soils that build up large pore pressures or show more than about 15% degradation of strength due to earthquake shaking. As illustrated in Table 10-4, pseudostatic analyses produced factors of safety well above 1 for a number of dams that later failed during earthquakes. These cases illustrate the inability of the pseudostatic method to reliably evaluate the stability of slopes susceptible to weakening instability. Nevertheless, the pseudostatic approach can provide at least a crude index of relative, if not absolute, stability.

Discussion. The pseudostatic approach has a number of attractive features. The analysis is relatively simple and straightforward; indeed, its similarity to the static limit equilibrium analyses routinely conducted by geotechnical engineers makes its computations easy to understand and perform. It produces a scalar index of stability (the factor of safety) that is analogous to that produced by static stability analyses. It must always be recognized, however, that the accuracy of the pseudostatic approach is governed by the accuracy with which the simple pseudostatic inertial forces represent the complex dynamic inertial forces that actually exist in an earthquake. Difficulty in the assignment of appropriate pseudostatic coefficients and in interpretation of pseudostatic factors of safety, coupled with the development of more realistic methods of analysis, have reduced the use of the pseudostatic approach for seismic slope stability analyses. Methods based on evaluation of permanent slope deformation, such as those described in the following sections, are being used increasingly for seismic slope stability analysis.

Table 10-4 Results of Pseudostatic Analyses of Earth Dams That Failed during Earthquakes

Dam	k_h	FS	Effect of Earthquake
Sheffield Dam	0.10	1.2	Complete failure
Lower San Fernando Dam	0.15	1.3	Upstream slope failure
Upper San Fernando Dam	0.15	~2–2.5	Downstream shell, including crest slipped about 6 ft downstream
Tailings dam (Japan)	0.20	~1.3	Failure of dam with release of tailings

Source: After Seed (1979).