PUBLIC

BEFORE THE NORTH CAROLINA UTILITIES COMMISSION

DOCKET NO. E-7, SUB 1146

In the Matter of		
Application of Duke Energy Carolinas, LLC, for Adjustment of Rates and Charges Applicable to Electric Utility Service in North Carolina	)))))	TESTIMONY OF L. BERNARD GARRETT PUBLIC STAFF – NORTH CAROLINA UTILITIES COMMISSION

BEFORE THE NORTH CAROLINA UTILITIES COMMISSION

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TESTIMONY OF L. BERNARD GARRETT
ON BEHALF OF THE PUBLIC STAFF
NORTH CAROLINA UTILITIES COMMISSION

JANUARY 23, 2018

Q. PLEASE STATE YOUR NAME, BUSINESS ADDRESS, AND PRESENT POSITION.

A. My name is Bernie Garrett. My business address is 1100 Crescent Green, Suite 208, Cary, North Carolina. I am the Secretary/Treasurer of Garrett and Moore, Inc.

Q. BRIEFLY STATE YOUR QUALIFICATIONS.

A. I am a licensed professional engineer with 28 years of experience engineering coal ash management projects, including the design and permitting of industrial landfills, the closure of coal ash impoundments, and the closure of coal ash landfills. Relevant projects include:

- Canadys Station (South Carolina Electric & Gas, or SCE&G) near Walterboro. South Carolina
  - Ash pond closure

- Cope Station (SCE&G) near Cope, South Carolina
  - Class Three landfill
  - Ash landfill closure
- Cross Station (Santee Cooper), near Pineville, South Carolina
  - Class Three Landfill
- McMeekin Station (SCE&G) near Columbia South Carolina
  - Ash pond closure
  - Ash landfill closure
- Urquhart Station (SCE&G), near Beech Island, South Carolina
  - Ash landfill closure
- Wateree Station (SCE&G) near Eastover, South Carolina
  - Ash pond closure
  - Class Three landfill
- Williams Station (SCE&G) near Charleston, South Carolina
  - Class Three landfill
  - Ash landfill closure

Additional qualifications are set forth in Appendix A.

Q. WHAT IS THE PURPOSE OF YOUR TESTIMONY?

A. The purpose of my testimony is to present the results of my investigation into the prudence and reasonableness of costs incurred by Duke Energy Carolinas, LLC ("DEC" or "Company") with respect to its coal ash management in South Carolina for which DEC is seeking cost recovery in this proceeding. In addition, I also present my perspective on the prudence and reasonableness of costs identified by DEC as part of its future regulatory obligations related to coal ash management in South Carolina.

Q. WHY DO YOU SAY "PRUDENCE AND REASONABLENESS"?

A. I am not an expert in utility regulation but have relied upon guidance from the Public Staff attorneys with respect to the legal standard for my investigation. Those attorneys inform me that under North Carolina General Statute 62-133, a utility's operating expenses must be "reasonable" to be included in the revenue requirement that is the basis for setting rates the utility may charge to consumers. Likewise, the cost of utility property allowed in the rate base, to which an authorized return may be applied, must also be "reasonable". Furthermore, I have been advised that management prudence is one aspect of this statutory reasonableness, and yet some costs or expenses can be prudent but still not reasonable for recovery as a component of the revenue requirement used for setting rates. For purposes of my testimony, I do not attempt to present the legal theory for a distinction between "prudence" and other "reasonableness"; rather, I just describe the facts that led us to conclude that a particular cost or expense is not reasonable for purposes of rate recovery.

Q. HOW DOES YOUR TESTIMONY DIFFER FROM THAT OF PUBLIC STAFF EMPLOYEES IN THIS CASE?

A. I understand that Public Staff witnesses Junis and Maness speak to disallowance for costs of environmental violations and the appropriate regulatory accounting treatment for coal ash-related costs. I do not address those issues. The testimony of Public Staff witness Vance Moore evaluated DEC's costs with respect to its coal ash management in North Carolina, and so our testimony together provides a combined perspective on the prudence and reasonableness of the coal ash closure costs for which DEC is seeking cost recovery in this proceeding.

Q. WHAT IS THE SCOPE OF YOUR INVESTIGATION INTO THE PRUDENCE AND REASONABLENESS OF DEC'S COAL ASH MANAGEMENT COSTS?

A. I reviewed the approach taken by DEC for each of DEC's CCR units – meaning each coal ash landfill, surface impoundment (basin), structural fill, or other means of disposing of coal ash located in South Carolina to evaluate whether the approach taken by DEC was the least cost method of achieving compliance with the laws and regulations governing coal ash management. To the extent the approach taken by DEC was not the least cost method of achieving compliance with the laws and regulations governing coal ash management, I compared the costs incurred by DEC from January 1, 2015, through November 30, 2017 to the estimated costs for the least cost method, and recommend that the Commission disallow the difference is these costs.

In some circumstances, DEC incurred costs associated with management of coal ash from CCR units that were not required under State or federal law. In those circumstances, I evaluated the specific facts and details surrounding those CCR units to determine whether I agreed management of those CCR units was reasonable and prudent. If management of those CCR units were reasonable and prudent, I reviewed DEC's actions and costs incurred to determine if I agreed with their decisions. To the extent I believed that DEC's actions and costs incurred were not reasonable nor prudent, I recommend that the Commission disallow these costs.

Q. PLEASE DESCRIBE THE RESOURCES UTILIZED IN CONDUCT OF YOUR INVESTIGATION.

A. In order to prepare this testimony, I reviewed the testimony and work papers of DEC witnesses Kerin, Wright, McManeus, and others. Through the Public Staff, I also submitted extensive discovery to DEC regarding its selection and analysis of CCR unit closure options, including the technical and financial basis for such decisions. I also participated in meetings, site visits, and conference calls with Duke personnel.

Q. PLEASE SUMMARIZE YOUR TESTIMONY.

A. My testimony is divided into two parts. First, I provide a brief overview of DEC's legal and regulatory obligations related to coal ash management. I review the costs incurred by DEC from January 1, 2015, through November

30, 2017, related to coal ash management and the technical basis for the expenditures to indicate my opinion on the reasonableness of those decisions, and how those comport with providing the lowest cost compliance options for customers.

The second part of my testimony focuses on the technical basis for the future compliance alternatives proposed by DEC as part of its recognition of future legal and regulatory obligations. While DEC does not propose to utilize these future costs in this rate case for the determination of future rates, they form the basis for the regulatory accounting treatment proposed by DEC. As such, they require analysis as to the reasonableness of the technical basis for including these costs. The adjustments that I recommend in my testimony are incorporated into the rates proposed by Public Staff witness Maness.

DEC'S CCR UNITS IN SOUTH CAROLINA

Q. PLEASE PROVIDE A SUMMARY OF DEC'S CCR UNITS LOCATED IN SOUTH CAROLINA.

A. The W.S. Lee facility is located in Belton, South Carolina. It was an operational coal ash facility from 1951 to 2014, with a generation capacity of approximately 370 megawatts. The CCR units at the facility include the Primary Ash Basin, which was constructed in 1974 and contains approximately 2.2 million tons of ash, the Secondary Basin, which was constructed in 1978 and contains approximately 30,000 tons of ash. In addition to the Primary Ash Basin and the Secondary Ash Basin, there are three other ash management areas, referred to by DEC as the Structural Fill Area, Inactive Ash Basin, and Old Ash Fill. Public Staff Garrett Exhibit 1 presents a site plan identifying all ash management areas at the W.S. Lee site.

DEC'S LEGAL AND REGULATORY OBLIGATIONS

Q. PLEASE PROVIDE A SUMMARY OF THE REGULATORY REQUIREMENTS FOR EACH OF DEC'S CCR UNITS LOCATED IN SOUTH CAROLINA.

A. Closure of the W.S. Lee impoundments must comply with federal regulations, specifically the "CCR Rule", which is the Hazardous and Solid Waste Management System: Disposal of Coal Combustion Residuals from Electric Utilities, promulgated by the United States Environmental Protection Agency ("EPA") and published Federal Register Vol. 80, No. 74, on April 17, 2015, and various South Carolina statutes and regulations.

Q. DO YOU AGREE WITH THE SUMMARY OF REQUIREMENTS REGARDING CCR AND CLOSURE OF COAL ASH IMPOUNDMENTS INCLUDED IN PAGES 22 THROUGH 36 OF DUKE WITNESS KERIN'S DIRECT TESTIMONY?

A. I have reviewed the discussion of regulatory requirements included in DEC witness Kerin's testimony and agree with his general characterization of the applicable federal and State regulations addressing the management and closure of CCR units in South Carolina. However, Kerin's testimony on applicable regulations focuses primarily on North Carolina facilities, since seven of the eight sites are located in North Carolina, and does not discuss the applicable South Carolina regulations in any detail.

Q. PLEASE PROVIDE ADDITIONAL EXPLANATION OF THE APPLICABLE SOUTH CAROLINA REGULATIONS.

A. The South Carolina Department of Health and Environmental Control (SCDHEC) regulates ash basins through two primary regulatory programs: (1) The National Pollutant Discharge Elimination System (NPDES) Wastewater Permitting Program and (2) the Dams and Reservoirs Safety Program. The NPDES Wastewater Permitting Program regulates the discharge of wastewater outfalls and requires groundwater monitoring associated with the permitted/regulated ash basins. The Dams and Reservoirs Safety Program regulates the structural integrity of dams associated with the permitted/regulated ash basins. The Primary Ash Basin and the Secondary Ash Basin at W.S. Lee are regulated under these two programs.

Q. ARE THERE ANY SCDHEC REGULATIONS THAT ARE SPECIFIC TO ASH BASINS?

No. South Carolina does not have a regulatory scheme similar to the North Carolina Coal Ash Management Act (comprised of Session Law 2014-122, Senate Bill 729; Session Law 2015-110, Senate Bill 716; and Session Law 2016-95, House Bill 630,; collectively referred to as "CAMA") guiding the closure of ash basins.

Q. ARE THERE ANY ASH BASINS REGULATED BY THE EPA CCR RULE?

A. Yes. As noted by DEP witness Kerin, the Primary and Secondary Ash Basins are regulated by the EPA CCR Rules, but the 1951/1959 inactive basin is not.

Q. WHAT IS DEC'S SELECTED CLOSURE METHOD FOR REGULATED BASINS AT THE W.S. LEE FACILITY?

A. For the Primary and Secondary Ash Basins, DEC has selected to close the basins by excavation with disposal in an on-site landfill. In response to discovery from the Public Staff, DEC indicated that: The WS Lee Primary and Secondary Ash Basins had historically experienced problems with dam slope stability, with recurring slumps and scarps. The Secondary basin was operated at a normal pool water level from a dam stability perspective even though it contained very little coal ash, only approximately 10,000 tons.

Based on the historic performance of these basins, the decision was made and communicated to SCDHEC in December 2014 to recommend excavation of the Primary and Secondary ash basins.

With the knowledge that the Secondary Basin was practically empty from a coal ash content perspective, it was readily identified as a potential site for a new on-site CCR landfill to

> contain the Primary Ash basin contents. The design of the landfill embankments could address slope stability issues.
>
> Design of the CCR landfill in the Secondary Ash Basin footprint is continuing.[1]

Q. WHAT IS YOUR OPINION REGARDING DEC'S SELECTED CLOSURE METHOD FOR REGULATED BASINS AT THE W.S. LEE FACILITY?

A. DEC did not provide any additional reports or alternatives analysis supporting the selected closure method, but I concur with DEC's plan to close the Primary and Secondary Ash Basins in an on-site CCR landfill.

My opinion is not based on any definitive closure alternatives analysis completed by DEC, but rather my own personal, extensive experience working with SCDHEC on coal ash management projects over the last 20 years. Of the 13 utility owned coal-fired power plants located in South Carolina, I have provided engineering and permitting services for coal ash management projects at nine of these plants.[2] Projects include ash basin closures, new ash landfills, landfill closures, and wastewater ponds.

Also, in South Carolina, ten of the 13 utility owned coal fired power plants have ash basins that either have closed, are in the process of closing, or closure plans have been announced. To the best of my knowledge, I

[1] DEC Response to Public Staff Coal Ash Data Request 5-5(b), July 10, 2017.

[2] SCE&G: Canadys, Cope, McMeekin, Wateree, Williams, Urquhart; Santee Cooper: Cross, Winyah; Savannah River Site: D-Area Powerhouse.

believe that at all ten of these plants, the basin closure designs approved by SCDHEC are excavation and disposal in a lined landfill.

Therefore, while cap in place may have been an acceptable alternative in meeting the EPA CCR rule, I do not believe that the alternative would have been approved by SCDHEC.

In addition, I concur with DEC's approach to utilize an on-site landfill, since this is consistent with Duke Energy's stated guiding principles[3] and provides lower cost solution as compared to an off-site landfill. Also, I concur with the idea of repurposing the Secondary Ash Basin area as the location for the on-site landfill.

Q. WITH REGARD TO THE OTHER ASH MANAGEMENT AREAS AT THE W.S. LEE SITE, HOW ARE THEY REGULATED?

A. The Structural Fill Area was developed between 2000 and 2007 and was developed in order to excavate ash from the primary basin to improve the operational efficiency of the wastewater treatment system. Approval was received from the SCDHEC NPDES Wastewater Permitting Program and included input from the SCDHEC Bureau of Land and Waste Management regarding the design of the engineered final cover system, which was

[3] Duke Energy: Guiding Principles for Ash Basin Closure. Online at:https://www.duke-energy.com/_/media/pdfs/our-company/ash-management/guiding-principles-for-closure-fact-sheet.pdf?la=en. Last accessed: January 12, 2018

installed once the project was completed. DEC reported that as of January 1, 2015, approximately 859,200 tons of ash are disposed in the Structural Fill Area. The Inactive Ash Basin and the Old Ash Fill, which were both developed in the 1950s and operated from 1951 to 1974, are both unregulated. DEC reported that as of January 1, 2015, approximately 1,178,338 tons of ash were contained in the Inactive Ash Basin and 378,989 tons of ash were contained in the Old Ash Fill.

Q. HAVE YOU REVIEWED THE ONGOING OR PLANNED CLOSURE ACTIVITIES ASSOCIATED WITH THESE UNREGULATED AREAS?

A. Yes.

Q. DO YOU AGREE WITH DEC'S APPROACH TO THE ONGOING OR PLANNED CLOSURE ACTIVITIES ASSOCIATED WITH THESE AREAS?

A. No. In my opinion, the actions taken and the costs incurred associated with closure activities at the Inactive Ash Basin and the Old Ash Fill were not reasonable or prudent.

DEC provided in response to Public Staff data requests that:

> Based on stability analysis, the 1951/1959 Inactive Ash Basin did not meet the required CCR Rule dam safety factors for maximum storage pool and liquefaction conditions. Additionally, there was historical evidence that the embankment along the Broad River was partially constructed with coal ash, and was also likely founded on coal ash.

> In September 2014, Duke Energy and SCDHEC entered into a Consent Agreement SCDHEC 14-13-HW that required Duke Energy to excavate the Inactive 1951/1959 Ash Basin. The coal ash has been excavated from this ash basin and trucked to the solid waste landfill operated by Waste Management at Homer, Georgia.[4]

The Public Staff asked for further engineering support for the statement that the 1951/1959 Inactive Ash Basin did not meet the required CCR Rule dam safety factors for maximum pool storage and liquefaction conditions, and DEC provided a report entitled "Phase 2 Reconstitution of Ash Pond Designs, Comprehensive Report, W.S. Lee Station, Anderson County, South Carolina for the W.S. Lee Primary Ash Pond (State Id D4887), W.S. Lee Secondary Ash Pond (State Id D4888), W.S. Lee Retired Ash Basin", prepared by AECOM. The AECOM report, dated January 4, 2018, references several earlier reports that identified geotechnical risk issues and possible risk mitigation alternatives known to DEC in 2014. The AECOM report further states, however, that each of the risk issues have since been mitigated by removal of interior ash deposits, and indicated that this work was completed in November 2017.

I also reviewed other reports submitted by DEC in response to Public Staff data requests related to this topic that indicated geotechnical risks existed at the site, including the September 12, 2014 report entitled "1951 Retired

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[4] DEC Response to Public Staff Coal Ash Data Request 5-5(b), July 10, 2017.

Ash Basin, Duke Energy Lee Steam Station" by S&ME, which is included as Public Staff Garrett Exhibit 2. Conclusions of the report are as follows:

> We recommend monitoring the performance of the dikes to observe changing conditions and/or performance issues. In the short-term, planned additions of rip-rap protection and/or armoring along the River at Sections C-C and B-B may help improve conditions against shallow surface sloughing locally at these levels previously caused by local loss of passive resistance as a result of erosion along the shoreline. If increasing factors of safety to industry-standards is desired or required, significant buttressing and/or reconstruction of the downstream embankment(s) to flatter slopes would be required.
>
> Additional data and surveying may be necessary to verify the existing slope (along the Saluda River) is as steep as recent surveys indicate and also define the topography further into the River. Also, additional data in the apparent "ash layer" would provide insight as to the soil composition of the region. Slope stability results did indicate surface sloughing along the apparent "ash layer" is possible (FS = 1.04). As previously mentioned, the apparent "ash" was modeled with an underlying ash embankment fill strata because data was un-retrievable along that portion of the slope embankment.

Based on my review of relevant documents related to the geotechnical issues associated with the Inactive Ash Basin, I concur with DEC that some action was necessary to mitigate risk associated with a potential geotechnical issue. However, it is my opinion that a more cost-effective approach was feasible than immediate excavation of the basin and transportation of the CCR material off-site.

Q. WHAT APPROACH SHOULD DEC HAVE PURSUED?

A. To help address the geotechnical issues, DEC should have undertaken a grading and slope stabilization project, whereby the overly steep sections of the perimeter berm are excavated and moved to the interior of the basin. The result of the project would have been a flattening of the perimeter berm slopes and mitigation of the geotechnical concerns. Attached Public Staff Garrett Exhibit 3 illustrates the concept.

Q. IF DEC HAD PERFORMED THE GRADING AND SLOPE STABILIZATION PROJECT AS YOU DESCRIBE, DO YOU BELIEVE THAT DEC'S DECISION TO EXCAVATE ASH FROM THE INACTIVE ASH BASIN WAS REASONABLE?

A. Yes. In some circumstances, it is appropriate for DEC to take some actions to mitigate environmental risks even if they are not compelled to by environmental regulations. Regardless of the regulatory status of the Inactive Ash Basin as unregulated, the proximity of the ash to the Saluda River and the potential for undocumented groundwater contamination at the site provide some reasonable basis for DEC to seek to mitigate a long-term environmental liability.

Q. DO YOU THEREFORE AGREE THAT THE ACTIONS TAKEN BY DEC TO EXCAVATE THE ASH FROM THE INACTIVE ASH BASIN WAS PRUDENT?

A. No, the timing of DEC's actions to excavate the ash from the Inactive Ash Basin and decision to transport the ash material off-site by truck to Homer, Georgia, was unreasonable. A two-phased approach, whereby DEC addressed the immediate geotechnical concerns followed by an excavation plan that allowed disposal of the ash in the on-site landfill once it is completed, would have been a more cost–effective approach. This is consistent with the approach being taken in North Carolina under CAMA, where the immediate dam stability concerns are addressed first, while options for closure of the basins are developed.

Q. IS YOUR POSITION THE SAME FOR THE OLD ASH FILL?

A. Yes. DEC should have waited for construction of the on-site landfill to be complete prior to excavation of the ash in this area. There were no geotechnical concerns or other imminent environmental problems posed by the Old Ash Fill that required immediate excavation of the area.

Q. ARE THERE OTHER FACTORS THAT HAVE POTENTIALLY IMPACTED DEC'S SELECTION OF CLOSURE OPTIONS FOR W.S. LEE?

A. Yes. As discussed by DEC witness Kerin and Public Staff witness Junis, DEC entered into a Consent Agreement with SCDHEC applicable to ash management at the WS Lee plant. The Consent Agreement requires ash excavation of the Inactive Ash Basin, the Old Ash Fill, and any other areas where ash may have potentially migrated from these sites.

Q. HOW IS THE CONSENT AGREEMENT BETWEEN DEC AND SCDHEC REFERENCED BY DEC WITNESS KERIN APPLICABLE TO YOUR POSITION?

A. I am not aware of any documented environmental compliance issues associated with the Inactive Ash Basin or the Old Ash Fill at the W.S. Lee plant prior to DEC and SCDHEC entering into the consent agreement. The Public Staff requested information on correspondence between DEC and SCDHEC leading up to establishment of the consent agreement and also contacted SCDHEC personnel to discuss the development of the consent agreement. It is my understanding that DEC initiated discussions with SCDHEC regarding the conditions (work required, time period) of the Consent Agreement. Since, SCDHEC's goals of enhanced environmental protection were met by entering into the Consent Agreement, SCDHEC agreed. By entering into the Consent Agreement, however, DEC committed to a method of closure, including excavation and transportation offsite, and disposal in an applicable disposal facility, that may not have otherwise been necessary under applicable state or federal law. The Consent Agreement states:

> Prior to 1974, CCR was placed in the Inactive Ash Basin, which is an unregulated basin located south of the power plant.

Since the Inactive Ash Basin was unregulated, the Consent Agreement should not be viewed as an enforcement mechanism. Instead, the Consent

Agreement provided the regulatory framework for SCDHEC to review and approve the actions initiated by DEC.

Q. DO YOU BELIEVE THE COST INCURRED BY DEC TO DISPOSE OF ASH THAT WAS TRANSPORTED TO THE OFF-SITE LANDFILL IN HOMER, GEORGIA WERE REASONABLE AND PRUDENT?

A. No. The two-step approach described above would have eliminated the need for disposal of the ash in an off-site landfill and would have resulted in a much lower cost for customers. Therefore, I am recommending the Commission consider certain disallowance of costs, primarily associated with the transportation of ash off-site.

Q. WILL DEC HAVE CAPACITY IN THE PLANNED ON-SITE LANDFILL TO MANAGE ALL THE ASH ON SITE?

A. Yes. DEC's Demonstration of Need submittal to SCDHEC states the planned on-site landfill proposed to be located in the Secondary Ash Basin, will encompass an area of approximately 35 acres, and store 2.9 million cubic yards (or 3.5 million tons) of ash, which would provide sufficient capacity to dispose of the ash contained in Primary Ash Basin, the Secondary Ash Basin, the Inactive Ash Basin, and the Old Ash Fill. In addition, once the Primary Ash Basin is "closed," additional land will be available for cost-effective, lateral expansion of the landfill for any "overruns" on ash quantities

Q. DO YOU AGREE WITH DEC'S PLANS REGARDING THE STRUCTURAL FILL PROJECT?

A. SCDHEC stated in a conference call with the Public Staff on Thursday, January 4, 2018, that the Structural Fill Area was developed in accordance with the applicable environmental regulations and that SCDHEC is not contemplating any future action to address the structural fill.

As previously discussed, the Structural Fill Area approval included design requirements for the engineered final cover system, which was installed once the Structural Fill project was completed. Public Staff Garrett Exhibit 4 provides an aerial image of the Structural Fill Area, which illustrates the engineered final cover system. In contrast to the Inactive Ash Basin, where a potential environmental risk existed, I am not aware of any environmental concerns associated with the Structural Fill Area. While there are no costs included in DEC's request for cost recovery in this proceeding related to excavation of the Structural Fill Area, I disagree with DEC's plans to excavate this area in the future.

<u>CONCLUSION</u>

Q. PLEASE PROVIDE A SUMMARY OF THE ADJUSTMENTS TO DEC'S REQUEST FOR COST RECOVERY THAT YOU RECOMMEND.

A. Public Staff Garrett Exhibit 5 provides a summary of my recommended adjustments, resulting in an adjustment of the current expenditure of

$27,275,192. Supporting details of the adjustment are provided in Confidential Public Staff Garrett Exhibit 6 for the Inactive Ash Basin and Confidential Public Staff Garrett Exhibit 7 for the Old Ash Fill. The amount is then included in the testimony of Public Staff witness Maness in his recommendations for the appropriate recovery of these costs.

Q. DOES THIS CONCLUDE YOUR TESTIMONY?

A. Yes, it does.

Appendix A

Qualifications of Garrett and Moore, Inc.

Garrett and Moore, Inc., specializes in engineering services for power and waste industries. We remain focused and specialized in these markets and are dedicated to continuing to advance the reputation of excellence our staff has established through the years. Our company has been responsible for the construction administration and Construction Quality Assurance for about $90 million worth of lined landfill, final cover system, and lined wastewater pond construction since 2007, with much of that work specific to CCR landfills and ash basins. We have familiarity with the federal CCR Rule and the North Carolina Coal Ash Management Act, and have tremendous experience with CCR disposal methods and their associated costs.

Vance Moore and Bernie Garrett have specialized expertise in the following areas:

Coal Combustion Residuals

Through our firm of Garrett and Moore, Inc., we have provided engineering and consulting services to support power companies in the management of coal combustion residuals (CCRs), including but not limited to the following:

- ☐ Groundwater Monitoring
- ☐ Groundwater Corrective Action
- ☐ Hydrogeological Investigations
- ☐ Site Characterization Studies
- ☐ Geotechnical Evaluations
- ☐ Stability and Liquefaction Analysis
- ☐ Ash Pond Closure Design
- ☐ FIN 47 Cost Liability Estimating
- ☐ Ash Pond Closure Construction
- ☐ Ash Pond to Landfill Conversion
- ☐ Source Remediation
- ☐ Dewatering Design
- ☐ Ash Landfill Siting & Design
- ☐ Ash Landfill Construction
- ☐ Landfill Closure & Post-Closure
- ☐ Federal CCR & CAMA Rule Guidance
- ☐ Regulatory Compliance
- ☐ Environmental / Permit Audits

Solid Waste Engineering

Through our firm of Garrett and Moore, Inc., we have provided full-service solid waste design and permitting services for municipal solid waste (MSW), construction and demolition debris (C&D), land clearing and inert debris (LCID), industrial waste, tire monofills, and coal combustion ash landfills. We have a very successful track record of overseeing landfill development projects from concept to operations. Our expertise in solid waste engineering includes the following:

- ☐ Facility Siting Studies
- ☐ Engineering Design
- ☐ USEPA HELP Modeling
- ☐ Slope Stability & Liquefaction Analysis

- ☐ Settlement and Bearing Capacity
- ☐ Alternative Liner Analysis
- ☐ Stormwater Management & Design
- ☐ Equivalency Determinations
- ☐ Recyclables Program Management
- ☐ Landfill Closure & Post-Closure
- ☐ Convenience Center Planning / Design
- ☐ Waste Treatment & Processing
- ☐ Landfill Gas Remediation Plans
- ☐ Leachate Management System Design
- ☐ Landfill Gas Planning and Design
- ☐ Operations Planning
- ☐ Life of Site Analysis
- ☐ Alternate Final Cover Evaluations
- ☐ Transfer Stations
- ☐ Compost Systems
- ☐ Special Waste Permitting
- ☐ Operations & Maintenance

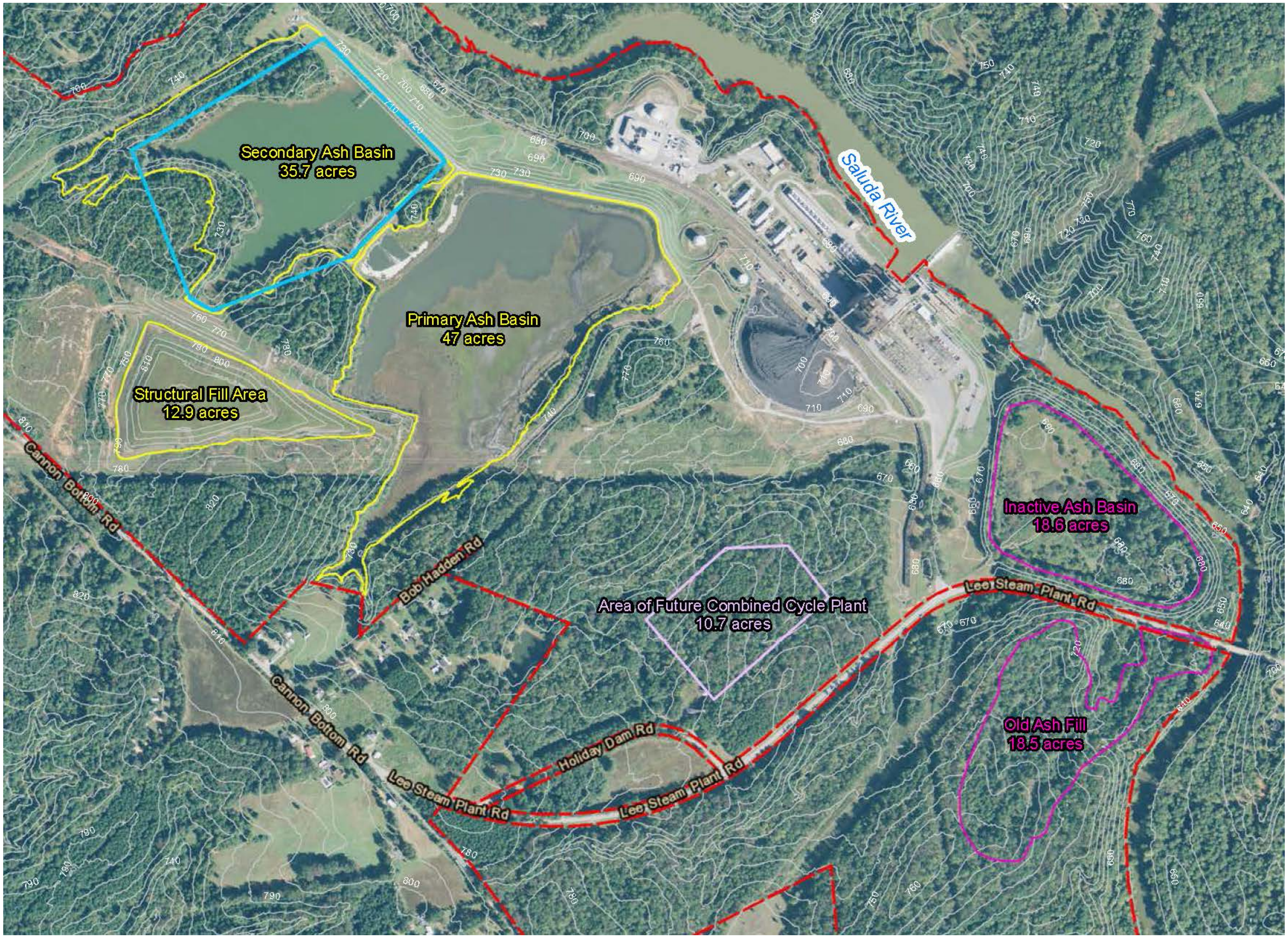
Bernie Garrett and Vance Moore have been providing engineering services for CCR management projects continuously since 1995. Over the last 10 years, we have performed all engineering associated with CCR management projects at all six of SCE&G's coal fired power plants, as well as facilities owned and operated by Santee Cooper. Our credentials include the following:

■ **Vance F. Moore, P.E**

Mr. Moore is a principal and founding member of Garrett & Moore.
Mr. Moore has 27 years of experience providing environmental engineering and consulting services to the power and waste industries. He has provided design, permitting, construction quality assurance, and operations support for numerous RCRA Subtitle D landfill projects, ash landfill projects, ash landfill closure projects, and ash pond closures in North and South Carolina.
Registrations: Professional Engineer – Georgia, North Carolina, South Carolina
Education: B.S., Civil Engineering, North Carolina State University, 1989
Associations: North Carolina SWANA Chapter - Technical Committee.
South Carolina SWANA Chapter

■ **Bernie Garrett, P.E.**

Mr. Garrett is a principal and founding member of Garrett & Moore.
Mr. Garrett has 27 years of experience providing environmental engineering and consulting services to the power and waste industries. His experience and professional responsibilities have progressed from project engineer with a major national engineering firm, project manager on solid waste landfill projects with a regional engineering firm, to client/project manager responsible for comprehensive engineering and consulting at Garrett & Moore, Inc.
Mr. Garrett has been working on coal ash management projects continuously since 1999. He has provided design, permitting, and construction quality assurance and operations support for ash pond closures, ash landfill projects, and ash landfill closure projects.
Registrations: Professional Engineer - Georgia, North Carolina, South Carolina, Virginia.
Education: B.S. Civil Engineering, Virginia Tech (1989);
M.S. Environmental Engineering, Old Dominion University (1996)
Associations: PENC Central Carolina Chapter Board of Directors
ACEC/PENC Solid and Hazardous Waste Subcommittee


Secondary Ash Basin
35.7 acres
Primary Ash Basin
47 acres
Structural Fill Area
12.9 acres
Saluda River
Inactive Ash Basin
18.6 acres
Area of Future Combined Cycle Plant
10.7 acres
Old Ash Fill
18.5 acres
Cannon Bottom Rd
Bob Hadden Rd
Lee Steam Plant Rd
Holiday Dam Rd

1951 RETIRED ASH BASIN
DUKE ENERGY
LEE STEAM STATION
BELTON, SOUTH CAROLINA
S&ME Project No. 7126-14-005

Prepared for:

Duke Energy Carolinas, LLC
526 South Church Street
Charlotte, North Carolina 28202

SOUTH CAROLINA
S&ME, INC.
NO. C00473
CERTIFICATE OF AUTHORIZATION

Prepared by:

S&ME
301 Zima Park Drive
Spartanburg, South Carolina 29301

SOUTH CAROLINA
PROFESSIONAL ENGINEER
REGISTERED
9-12-14
NO. 19495
MICHAEL G. REVIS

September 12, 2014

OFFICIAL COPY

Jan 23 2018

September 12, 2014

Duke Energy Carolinas, LLC
Mail Code EC11J
526 South Church Street
Charlotte, North Carolina 28202

Attention: Mr. Timothy M. Russell, P.E.
Senior Engineer, Program Engineering

Reference: Existing Basin Dike Stability Evaluation and
Liquefaction Potential Study
1951 Retired Ash Basin
Duke Energy Lee Steam Station
Belton, South Carolina
S&ME Project No. 7126-14-005

Dear Mr. Russell:

As requested, S&ME, Inc. has completed our evaluation of the slope stability and liquefaction potential of the existing 1951 retired ash basin dam at the Lee Steam Station in Belton, South Carolina. Our work was performed in accordance with Proposal No. 71-14-00036 **Revision 1** dated April 30, 2014. The purposes of the evaluations were to review existing available data, perform additional exploration and laboratory testing, and evaluate the stability of the existing dikes and liquefaction potential of embankment/foundation materials beneath the existing dikes. The following report presents a brief description of the background information, the evaluation procedures and results, and our recommendations regarding dike slope stability and liquefaction potential.

S&ME appreciates the opportunity to offer our engineering assistance to this project. If you have any questions concerning the information presented or if we can be of further assistance, please feel free to contact us.

Sincerely,
S&ME, Inc.

Frank Morris, E.I.
Project Professional
fmorris@smeinc.com

Michael Revis, PE
Senior Engineer
mrevis@smeinc.com

Jason Reeves, PE
Senior Project Manager
jreeves@smeinc.com

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TABLE OF CONTENTS

TABLE OF CONTENTS

TABLES LIST

FIGURES LIST (Report)

FIGURES

APPENDIX ATTACHMENTS

1.0 OBJECTIVE

The objectives for this evaluation include the following:

- Review previous subsurface, laboratory and slope stability analysis data from the 2007/2008 SCS Engineers (SCS) evaluation.
- Present the results of the geotechnical exploration program performed as part of this analysis, which consisted of geo-probe borings (continuous sampling) with soil sleeves, cone penetrometer (CPT) soundings with shear wave velocity measurements, dilatometer modulus testing (DMT) with shear strength correlation at depths of interest, soil test borings (STB) and sampling, and laboratory testing.
- Obtain survey data to define overall dike/basin geometry in preparation of this report.
- Review water level measurements in the existing piezometers taken/provided by Duke Energy Carolinas LLC (Duke).
- Perform slope stability analyses for various configurations/sections of the existing ash basin dike.
- Evaluate the liquefaction potential of the subsurface materials within and beneath the existing ash basin dike.

2.0 REFERENCE INFORMATION

In preparation of this report, the following documents were reviewed, used and/or incorporated in the analyses.

2.1 USGS's Custom Hazard Map generator (2014). http://geohazards.usgs.gov/hazards/apps/cmaps/

2.2 Lee Steam Station Piezometer Data Excel Spreadsheet, April 2014 – July 2014.

2.3 South Carolina Department of Health and Environmental Control (DHEC) R.61-71, South Carolina Well Standards (April 26, 2002).

2.4 United States Army Corps of Engineers (2003). Engineering and Design – Slope Stability, USACE Publication EM 1110-2-1902.

2.5 J.M. Duncan and S.G. Wright. Soil Strength and Slope Stability, John Wiley and Sons, Inc., 2005.

2.6 SCS Engineers (2008). Landfill Siting Study – Coal Combustion Products Landfill, Duke Energy Lee Steam Station, October 16, 2008.

2.7 GEO-Slope International Ltd. Slope/W GeoStudio 2012, version 8.0.10.6504.

2.8 WPC (2008). New Ash Landfill for Lee Steam Station (Draft), July 10, 2008.

2.9 Youd, et. al. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, April, 2001.

2.10 GeoLogismiki. CLiq Software, version 1.7.1.14

2.11 I. M. Idriss and R. W. Boulanger. Soil Liquefaction During Earthquakes, 2008.

2.12 Horton Jr., J. Wright and Victor A. Zullo. The Geology of the Carolinas, 1991.

2.13 Duke Energy (2012). Slope Stability Evaluation for Primary and Secondary Ash Pond Dams, Calculation Number LC-Units 1-3-0151-000, August 20, 2012.

2.14 Olsen, R.S. and Mitchell, J.K. (1995). "CPT stress normalization and prediction of soil classification." Proc., Int. Symp. on Cone Penetration Testing, CPT 95, Linkoping, Sweden, 257-262.

2.15 Olson, S.M. and T.D. Stark, "Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories," Canadian Geotechnical Journal, Vol. 39, No. 3, June, 2002, pp. 629-647.

2.16 FEMA Federal Guidelines for Dam Safety: Earthquake Analyses and Design of Dams, printed May 2005.

3.0 FIELD EXPLORATION AND TESTING

3.1 Geoprobe Boring with Soil Sleeves

The field exploration initially included twenty one (21) geoprobe ("direct push") borings with 5 ft. continuous soil sleeves pushed at the following locations: L-1 through L-13, L-6A through L-6D, and L-7A through L-7D. (Refer to the Figure 1 – Test Location Plan for approximate test locations.) Geoprobe borings L-1 through L-10 were performed on the crest of the dike, and geoprobe borings L-11 through L-13 were performed close to the toe of the slope near the Saluda River. In addition, four (4) offset geoprobe borings were performed at locations L-6 and L-7 to help delineate the starting and stopping depths of the ash fill within the basin and define the internal construction geometry of the embankments. Offset locations are suffixed in this manner: A, B, C, and D. Offset locations were spaced approximately 15-ft. on center from the original boring location in a southwest direction. Geoprobe borings, performed on the crest of the dike, extended to depths of 40 to 50 ft. (el. 647.4 to 637.3 ft.) below existing site grades. Geoprobe borings within the ash pond extended to depths of 25 to 40 ft. (el. 661.6 to 642.0 ft.) below existing site grades. Geoprobe borings near the Saluda River extended to depths of 13 to 15 ft. (el. 633.8 ft. to 637.5 ft.) below existing site grades.

Initially, geoprobe borings were planned at locations L-14 through L-16, but these locations were deemed unsafe for the geoprobe drill rig/crew because of steep terrain. This area is known as the apparent "ash layer", based on existing topography and historical information. Hand auger borings were attempted at these locations, but refused at depths between 1 and 1.5 ft. from existing grades. Large crushed stone and/or cobbles within the upper surface of the "ash layer" limited hand augering to shallow depths.

Geoprobe boring logs and photographs are included in Appendix C – Direct Push (Geoprobe) Logs and Photographs.

3.2 Cone Penetrometer Soundings/Testing (CPT)

After completion of the Geoprobe borings, S&ME performed thirteen (13) cone penetration soundings at locations L-1 through L-13 directly adjacent to the Geoprobe locations. The soundings were performed with a track-mounted Cone Penetration Test

(CPT) rig hydraulically pushing an electronically instrumented cone to depths ranging from 16.6 to 53.9 ft. (el. 672.3 to 633.5 ft.) below the existing ground surface (corresponding to CPT refusal) along the dike crest. At the soundings located near the Saluda River, the cone was hydraulically pushed (to CPT refusal) to depths ranging from 13.8 to 14.9 ft. (el. 636.5 to 633.9 ft.) below existing site grades. During penetration, the tip resistance, sleeve friction, and pore water pressure were measured and recorded in accordance with ASTM D-5778. This method produces a nearly continuous record of soil data that is useful in characterizing stratified soils and identifying thin layers.

Cone soundings were terminated (CPT refusal) when the cone tip could not be advanced using a tip stress of approximately 500 tons per square foot. Experience at others sites and correlation to soil test borings performed at this site determined the refusal material to be partially weathered rock.

Using the CPT rig, downhole shear wave velocity measurements were performed at approximate one-meter intervals in CPT soundings L-3, L-6, L-9 and L-10. The travel time of a shear wave generated at the ground surface to a shear wave geophone mounted near the cone tip was measured. For each measurement, the travel time of the first shear wave arrival is determined and corrected for the horizontal offset of the shear wave source. Interval shear wave velocities are calculated by dividing the difference in travel times by the distance between adjacent depths. Shear wave measurements were made to the maximum depth the cone could be hydraulically advanced.

CPT sounding logs and shear wave velocity results are presented in Appendix B – CPT (Cone Penetration) Logs, DMT (Flat Plate Dilatometer) Test Results, and Shear Wave Velocity Results.

3.3 Dilatometer Modulus Testing (DMT)

S&ME performed Dilatometer Modulus Testing (DMT) at six (6) locations; L-3, L-6, L-7, L-11, L-12, and L-13. Termination depths for locations on the crest of the dike ranged from 18 to 34 ft. (el. 670.0 ft. to 653.3 ft.) below existing site grades, and termination depths for locations along the Saluda River ranged from 6 to 14 ft. (el. 636.5 to 635.8 ft.) below existing site grades. Pressure measurements were recorded vertically every 2 ft. on-center from existing grades.

The modulus testing was performed with the track-mounted CPT rig using a standard Marchetti blade that is 15-mm. thick and 96-mm. wide. A 60-mm. diameter stainless steel membrane is seated in the middle of the plate. The plate is pushed into the ground, and the membrane is inflated with nitrogen gas. Pressures to inflate and deflate the membrane are recorded at select intervals. The data reduction for Flate Plate Dilatometer testing yields the following for soils: Drained Elastic Modulus, Undrained Shear Strength, Tangent Vertical Constrained Modulus, Drained Friction Angle, Coefficient of Lateral Earth Pressure at Rest, Overconsolidation Ratio, and Preconsolidation Pressure. Overall, DMT data helps in estimating soil parameters used for analysis.

3.4 Soil Test Borings

A soil test boring regime was established once the CPT, DMT and Geoprobe (Direct Push) field data was retrieved. This included performance of ten (10) of the thirteen (13) total borings extended to depths ranging from 51 to 60.9 ft. (el. 636.3 to 627.1 ft.) below existing grades on the crest of the dike. Boring extension depths ranged from 12.0 to 13.8 ft. (el. 638.5 to 635.0 ft.) below existing grades near the Saluda River. All borings were extended to Partially Weathered Rock (PWR) or auger refusal.

The soil test borings were performed with a truck-mounted drill rig (CME 750), equipped with an automatic hammer. The soil test borings were advanced using auger flight techniques; with Standard Penetration Test (SPT) split-spoon soil sampling at standard intervals, combined with continuous sampling at intervals where additional data was necessary. (See Table 3.1 below for the soil test boring sampling scheme.) Continuous sampling was accomplished in five (5) borings on the crest of the dike to aid in capturing the transition from dike/embankment fill, alluvium and foundation materials.

In addition to the disturbed sampling, a total of sixteen (16) undisturbed (thin-walled) samples were extracted from the borings. (Note that split-spoon samples were driven just above and below undisturbed sampling depths.)

Table 3.1 Soil Test Boring Sampling Scheme

Boring (STB)	Sample Depths (SPT)	UD (Shelby Tube) Depths (ft.)	Special Considerations
L-2	2.5’ centers in the upper 10’ & 5’ centers thereafter	5-7’, 13-15’	
L-3	2.5’ centers in the upper 10’ & 5’ centers thereafter	8-10’, 35-37’, 42-44’	
L-4	2.5’ centers in the upper 10’ & 5’ centers thereafter		Continuous SS from 38.5 - bottom
L-5	2.5’ centers in the upper 10’ & 5’ centers thereafter		Continuous SS from 20 – bottom
L-6	2.5’ centers in the upper 10’ & 5’ centers thereafter	10-12’, 45.5-47’	Continuous SS from 30 - bottom
L-7	2.5’ centers in the upper 10’ & 5’ centers thereafter	30-32’, 35-37’	Continuous SS from 38.5 - bottom
L-9	2.5’ centers in the upper 10’ & 5’ centers thereafter	5-7’, 32-34’	Continuous SS from 40 - bottom
L-11	2.5’ centers in the upper 10’ & 5’ centers thereafter	10-11.5’	
L-12	2.5’ centers in the upper 10’ & 5’ centers thereafter	6.5-8.5’, 10.5-12.5’	
L-13	2.5’ centers in the upper 10’ & 5’ centers thereafter	5-7’, 11-13’	

Soil strata depths were based on visual field classification by an S&ME geotechnical engineer in general accordance with the Unified Soil Classification System (USCS). The resulting soil classifications are presented on the Boring Logs in Appendix A – Soil Test Boring Logs. Similar soils were grouped into representative strata on the logs. The strata contact lines represent approximate boundaries between soil types. The actual transitions between soil types in the field are likely more gradual in both the vertical and horizontal directions than those which are indicated on the logs.

Auger cuttings were placed in an on-site container and removed from the site by a waste disposal subcontractor. The manifest and disposal documentation is attached in Appendix G – Supplemental Information. Following ground water measurements (see Section 4.4), the test locations were abandoned using a bentonite grout in accordance with SCDHEC regulations (Reference 2.3).

3.5 Laboratory Testing

S&ME soil laboratory technicians performed quantitative ASTM-standardized laboratory tests on selected samples obtained from various depths and locations to help classify the soils and formulate our conclusions and recommendations. The laboratory tests performed included the following:

- 9 Consolidated-Undrained Triaxial Shear Tests (ASTM D 4767)
- 11 Grain Size Distribution (ASTM D 422)
- 25 Fines Content (200 Wash Only, ASTM D 1140)
- 19 Soil Plasticity (Atterberg Limits) (ASTM D 4318)
- 19 Natural Moisture Content (ASTM D 2216)

The laboratory data sheets for the above listed tests are attached to this report in Appendix D – Laboratory Test Data. Please note that some changes were made to the number and/or type of laboratory test performed based upon judgments made by the geotechnical engineer at the time of exploration, due to the soil conditions observed.

4.0 SITE CONDITIONS

4.1 Dam Geometry

Based on the furnished project drawings from Davis & Floyd, the ash basin dikes are generally arranged in a 3-sides configuration or tri-oval. The tri-oval can be further divided into one section (referred to herein as the “tall embankment” section) that extends parallel to the Saluda River (making up the northeast portion of the tri-oval, including the north and east radii), and one section (referred to herein as the “short embankment” section) that extends parallel to Lee Steam Plant Road and the plant entrance road making up the south and northwest portions of the tri-oval, including the southwest radius). The short and tall geometry is further discussed in Sections 4.1.1 and 4.1.2, respectively.

4.1.1 Dike Bordering Plant Entrance Road ("Short Embankment")

Based on the furnished project drawings from Davis & Floyd, the dike that borders the plant entrance road and Lee Steam Plant Road are oriented in a general north-south and east-west direction, respectively. The overall length of this span is approximately 1,980 ft., with a maximum structural height on the order of 17 ft. The crest of the dike is near elevation 688 ft. and is approximately 10 to 20 ft. wide. The embankment slopes generally have inclinations of about 1.3 to 2:1 (horizontal to vertical). The steepest section, section A-A, was selected for conservative slope stability analysis. Section A-A represents conditions along the "short embankment" sections generally located parallel to the plant entrance road. These analysis sections are depicted in the Figure 2.

4.1.2 Dike Bordering the Saluda River ("Tall Embankment")

Based on the furnished project drawings from Davis & Floyd, the dike that borders the Saluda River is oriented in a general northwest-southeast direction. The overall length of this span, which is parallel to the river, is approximately 1,950 ft. An apparent "ash layer" is a feature of the northwestern half of the dike. S&ME borings and historical information indicate that a layer of ash was integrated into the dike construction. The apparent "ash layer" exists just south of an existing 60-inch diameter CMP pipe (extending in a northeast-southwest direction beneath the basin) and seems to have been part of original dam construction (prior to pond expansion). The maximum structural height for this section of the Retired Ash Basin Dam is on the order of 45 ft. The crest of the dike is near elevation 688 ft. and is approximately 10 to 20 ft. wide. The embankment slopes generally have inclinations of about 1.8 to 2:1 (horizontal to vertical) based on the selected cross-sections and topographic data. Sections B-B, C-C, D-D, and E-E were selected to represent conditions along the "tall embankment" sections generally located parallel to the Saluda River. These analysis sections are depicted in the Figures 3, 4, 5 and 6.

4.2 Area Geology

4.2.1 Residuum

The Lee Steam Station is located in northeast-central Anderson County along the Saluda River. The site is located near the western boundary of the Paris Mountain Thrust Sheet which is situated within the Inner Piedmont Belt (Reference 2.12). The western limits of the Paris Mountain Thrust Sheet is bounded by an unnamed fault. The primary rock type of the Paris Mountain Thrust Sheet is sillimanite-mica schist interlayered with amphibolite and quartzite with intrusive granite gneiss common. The age of the rocks are typically Cambrian for the metamorphics and Ordovician to Devonian for the instrusives.

The major portion of the bedrock in the Piedmont is covered with a varying thickness of residual soil, which has been derived by chemical decomposition and physical weathering of the underlying rock. The residual soils developed during the weathering of this bedrock consist predominately of micaceous silty sands and sandy silts which can grade to micaceous clayey silts and silty clays with nearness to the ground surface.

The boundary between the residual soil and the underlying bedrock is not sharply defined. Generally, a transition zone consisting of very hard soil and soft rock appropriately classified as “partially weathered rock” is found. Within the transition zone, large boulders or lenses of relatively fresh rock often exist, which are generally much harder than the surrounding material. The irregular bedrock surface is basically a consequence of differential weathering of the various minerals and joint patterns of the rock mass.

No known published references exist which document liquefaction or sand boil features associated with historic or prehistoric earthquake activity in the South Carolina Piedmont.

4.2.2 Alluvium

Alluvial soils (or alluvium) should be expected below and near the natural drainage features on site, especially in the features with flowing water. Alluvial soils are typically found near rivers and streams and are usually loose, unconsolidated, soil sediments which have been eroded, deposited, and reshaped by water in some form. Alluvium is typically made up of a variety of materials, including fine particles of silt and clay and larger particles of sand and gravel.

4.2.3 Existing Fill/Ash

It should be noted that the natural geological profile at the site has been modified by past grading activities that have resulted in the placement of fill and ash. Existing fill can vary in composition and consistency, and the engineering characteristics of existing fill can be difficult to predict. The majority of the ash was hydraulically placed using sluicing techniques. The engineering characteristics of ash are difficult to predict and its strength and compressibility is highly dependent on the depth of water within the basin.

4.3 Subsurface Conditions

Borings L-1 through L-13 represents the subsurface conditions that comprise the embankment and foundation materials. S&ME generated generalized subsurface profiles, which were used for analysis. The subsurface conditions generally consist of compacted fill in the existing embankment areas underlain by alluvial deposits. Alluvial deposits are then underlain by residuum/partially weathered rock layers. Some ash fill is present within the embankment profile in the north-cental portion of the tall embankment (Section B-B).

4.3.1 Embankment Fill

Based on soil borings located on the dike crest, embankment fills depths range from 34.5 to 45.5 ft. (el. 651.8 ft. to 641.8 ft.) from existing site grades. The sampled embankment fill composition consists primarily of sandy elastic silt (USCS Classification MH), silty sand (SM), clayey sand (SC), sandy silt (ML) and well-graded sand with silt (SW-SM). Standard penetration (“N”) values ranged from 5 to 60 blows per foot (bpf), indicating a low to high degree of compaction. Note that the embankment fill contained varied amounts of crushed stone, artificially elevating some of standard penetration (“N”) values.

The sampled fill primarily contained only trace amounts of organic matter, as would be expected for structural fill. However, the embankment fill did contain varied amounts of ash and unburnt coal (as discussed below).

Ash fills exist within the "tall embankment", located south of the 60-inch CMP. Ash fill exists intermittently throughout the embankment. Borings L-5 and L-6 encountered ash fill beginning at depths of 23 and 24.5 ft. (approximately el. 664 ft.) and ending at depths of 32.5 and 33 ft. (approximately el. 655 ft.) below existing site grades. Ash fill composition is primarily silty fine to coarse sand (SM). This stratum is referred to as the apparent "ash layer" because it extends laterally to the embankment face. Slight amounts of bottom ash and unburnt coal are typically present throughout the borings located along the Saluda River (borings L-4, L-5, L-6, L-7, L-8, and L-9). The sampled ash fill was generally dry at the time of drilling.

4.3.2 Fill at the Toe of the Embankment

Based on soil borings located at the toe of the embankment (boring L-11, L-12 and L-13), fill depths range from 5 to 10 ft. (el. 644.8 ft. to 640.5 ft.) below existing grades. The embankment fill composition ranges from silty sand (SM) to elastic silt (MH) to silt with sand (ML). Boring L-11 encountered a tree root/stump during sampling directly above refusal. The sampled fill at Borings L-12 and L-13 contained trace organics, rock fragments and unburnt coal. Standard penetration ("N") values ranged from 4 to 17 bpf. This soil layer is generally characterized as having a low to moderate degree of compaction.

4.3.3 Alluvial Deposits

Due to the proximity of the dike to the Saluda River, alluvial soils were encountered in all the soil borings, with the exception of Borings L-1, L-2 and L-11. Alluvial deposits range from bottom depths of 39 to 54 ft. (el. 647.8 ft. to 634.0 ft.) relative to existing crest of embankment site grades. Alluvial deposits range from bottom depths of 13 to 13.5 ft. (el. 636.8 ft. to 635.3 ft.) relative to existing toe of embankment site grades. The sampled alluvial deposit composition consists of organic silt (OL), silty sand (SM), poorly-graded sand (SP), well-graded sand with silt (SW-SM), clayey sand (SC), silt with sand (ML), sandy elastic silt (MH), poorly graded sand with clay (SP-SC) and sandy lean clay (CL). Standard penetration ("N") values ranged from WOH ("Weight of Hammer") to 47 bpf. The alluvial deposit did contain varied amounts of large cobbles, artificially elevating some of the standard penetration values ("N"). Generally, this soil layer was characterized as having a loose relative density or soft consistency. Boring L-8 was terminated in alluvium at a depth of 50 ft. (el. 638.7 ft.).

4.3.4 Residuum

Residual soils of the type common to the Belton area were encountered below the fill in Borings L-1 and L-2 at a depth of 37 ft. and 38 ft., respectively, and beneath alluvium in Borings L-6, L-7, L-10 and L-11 to bottom depths of 11.5 to 60.5 ft. The residuum consisted of elastic silt (MH), sandy silt (ML) and silty sand (SM). The N-values recorded in the residual soils ranged from 12 to 55 blows per foot, indicating a stiff to hard consistency for silt, and a medium dense relative density for sand. Borings L-1 and

L-10 were terminated in residual soils at a depth of 40 feet below the existing ground surface (el. 647.4 to 646.8 ft.).

4.3.5 Partially Weathered Rock (PWR)

Partially Weathered Rock (PWR) underlies alluvial or fill deposits in borings L-4, L-5, L-9, L-12 and L-13, and residuum in borings L-2, L-6 and L-7. The PWR layer bottom depth ranged from depths of 52.8 ft. to 60.9 ft. (el. 633.1 ft. to 627.1 ft.) relative to existing site grades on the crest of the dike, and 13.1 ft. to 13.8 ft. (el. 636.7 ft. to 635 ft.) relative to existing site grades near the Saluda River. PWR composition consists of elastic silt (MH), silty sand (SM), silt (ML) and poorly graded sand (SP). The poorly graded sand classification likely comes from the pulverization of PWR/rock fragments through SPT sampling. Partially weathered rock is defined as a transitional material between very hard soil and rock that has a Standard Penetration Resistance value of at least of 50 blows per 6 inches.

Borings L-2, L-4, L-5, L-6, L-7, L-9, L-12 and L-13 were terminated in PWR at depths of 13.1 to 60.9 ft. below the ground surface.

4.4 Groundwater Conditions

Groundwater elevations used for this analysis are based on S&ME boring data and review of the provided water level data (Reference 2.2) from April 25, 2014 through August 4, 2014. Groundwater elevations were taken from piezometers P-1 through P-18 by Duke Energy personnel. Piezometers are located at various locations along the embankment, as well as within the ash basin. The piezometer locations are depicted on Figure 1, while average groundwater elevations in each piezometer are presented in Table 4.1 and included in Appendix G – Supplemental Information. Twenty-four (24) hour water level readings were logged for each soil test boring and are shown in Table 4.2.

Table 4.1: Piezometer Groundwater (G/W) Elevations

Piezometer ID	Average G/W Elevation (ft.)
P-1	645.4
P-2	655.8
P-3	661.0
P-4	646.0
P-5	641.9
P-6R	653.0
P-7	658.0
P-8	651.7
P-9	648.5
P-10	652.2
P-11	647.7
P-12	644.7
P-13	644.9
P-14	643.0
P-15	639.2
P-16	639.0
P-17	637.7
P-18	638.6

Table 4.2: 24-hr. Groundwater (G/W) Elevations in S&ME Borings

Boring (STB)	24-hr. G/W Elevation (ft.)
L-2	650.1
L-3	643.5
L-4	643.3
L-5	645.9
L-6	643.2
L-7	643.3
L-9	644.1
L-11	639.5
L-12	639.1
L-13	639.3

Groundwater levels will fluctuate due to seasonal variations, rainfall, plant operations, River level and construction activity.

4.5 Laboratory Test Results

As previously discussed, selected samples from the field exploration program were subjected to laboratory tests for general classification and for shear strength parameters. The laboratory results are discussed in the following sections, and presented on the Summary of Laboratory Test Data and individual data sheets in Appendix D –Laboratory Test Data.

4.5.1 Classification Test Results

The USCS classifications of the embankment fill soils based on Percent Finer than a No. 200 sieve and Atterberg Limits testing are MH, ML, SM, SC, and SW-SM. The embankment fills soils have from 12.8 to 72.3 percent fines (material passing a No. 200 sieve). The minus No. 40 sieve portion of this soil has a Liquid Limit (LL) ranging from 37 to 67 with Plasticity Index (PI) values of 3 to 32 percent.

The alluvial soils tested have USCS classifications of SP, SC, SP-SC, SM, ML, MH, and CL. These soils have between 2.6 and 88 percent soil fines. The LL of the alluvial stratum ranged from NP (Non-Plastic) to 67 percent, with PI values from non-plastic (NP) to 31 percent.

The residual soil tested has a USCS classification of ML. The soil sample contained 72.8 percent fines, with a LL of 46 percent and PI of 17 percent.

4.5.2 CU Triaxial Shear Strength Tests

S&ME evaluated shear strength parameters of relatively undisturbed soil samples obtained at representative locations in the crest of the dike and toe of slope, performing a total of nine (9) Consolidated Undrained (CU) triaxial shear tests at various confining stresses and a total of twenty-six (26) individual soil specimens. Triaxial tests included samples of the following materials: embankment fill, alluvium, and residuum.

<u>Effective Stress Shear Strength</u> - Confining stresses for individual tests were assigned based on consideration of the typical stress range for the anticipated failure surfaces, approximately 500 psf to 4,000 psf, and also considered in-situ stresses of the samples. A

linear relationship for the effective shear strength, τ, relating to the embankment fill (Figure 4.1), alluvial soils (Figure 4.2), and residual soils (Figure 4.3) were interpreted from the CU triaxial tests were typically based on a failure criteria defined by the maximum principal stress ratio for effective stress parameters. CU triaxial test results were compiled to obtain an overall estimate of effective strength parameters using a linear relationship as shown in the following figures.

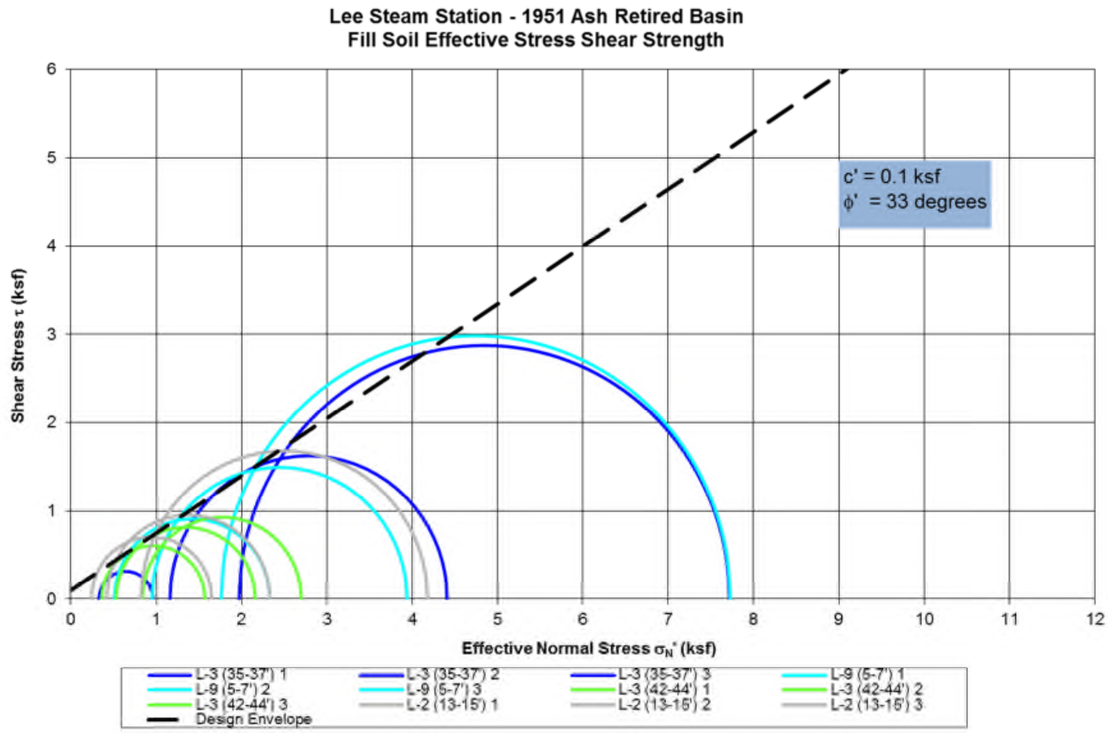


Figure 4.1: Fill Soil Effective Stress Shear Strength

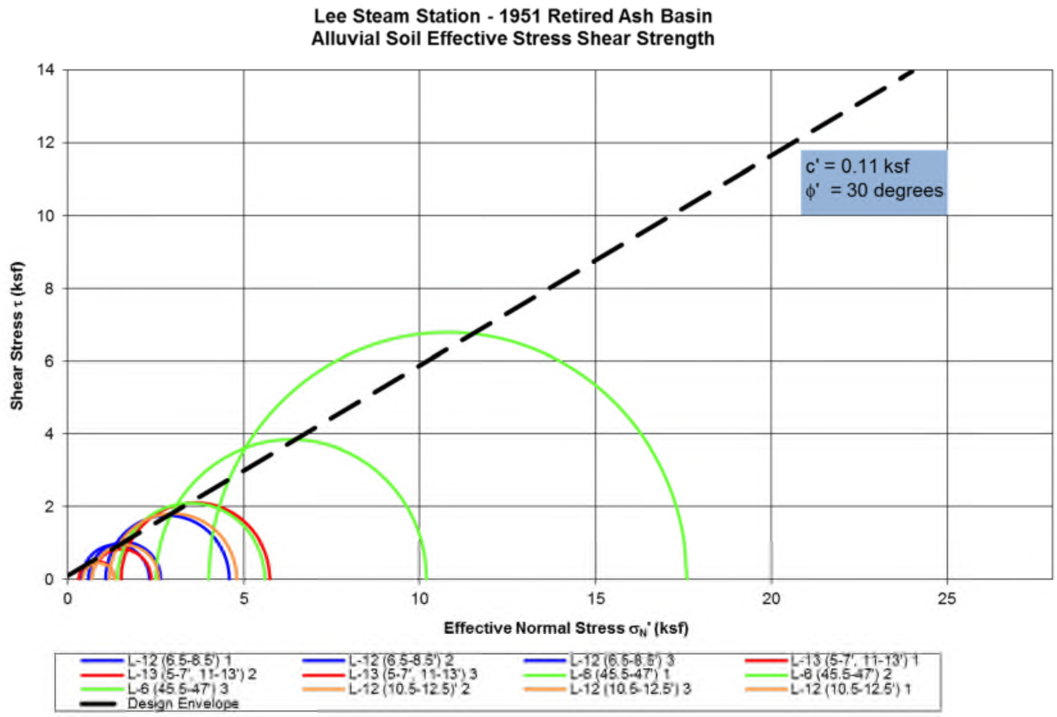


Figure 4.2: Alluvial Soil Effective Stress Shear Strength

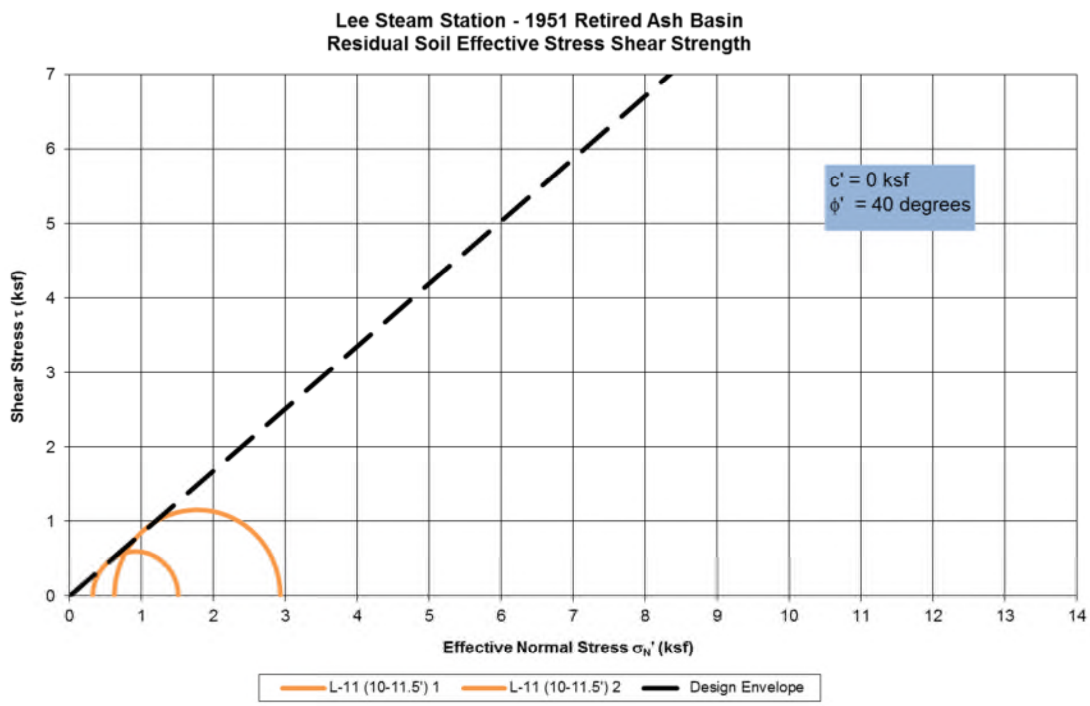


Figure 4.3: Residual Soil Effective Stress Shear Strength

Total Stress Shear Strength - Linear strength envelopes were estimated for the total stress shear strengths for the embankment fill (Figure 4.4), alluvial soil (Figure 4.5), and residuum soil (Figure 4.6). The total stress Mohr circles for each soil type were merged to develop a composite set of strength data. The total stress strength parameters, C_R and ϕ_R were generally defined based on total stress strengths interpreted at maximum deviator stress, generally at 15 percent axial strain.

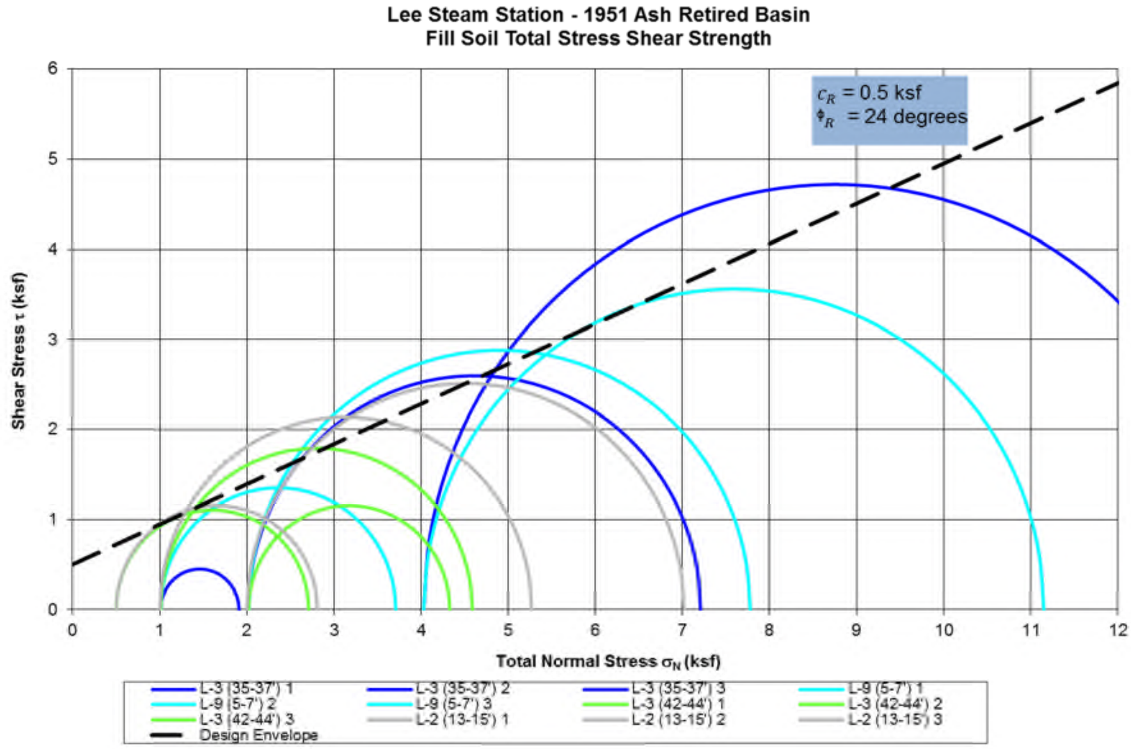


Figure 4.4: Fill Soil Total Stress Shear Strength

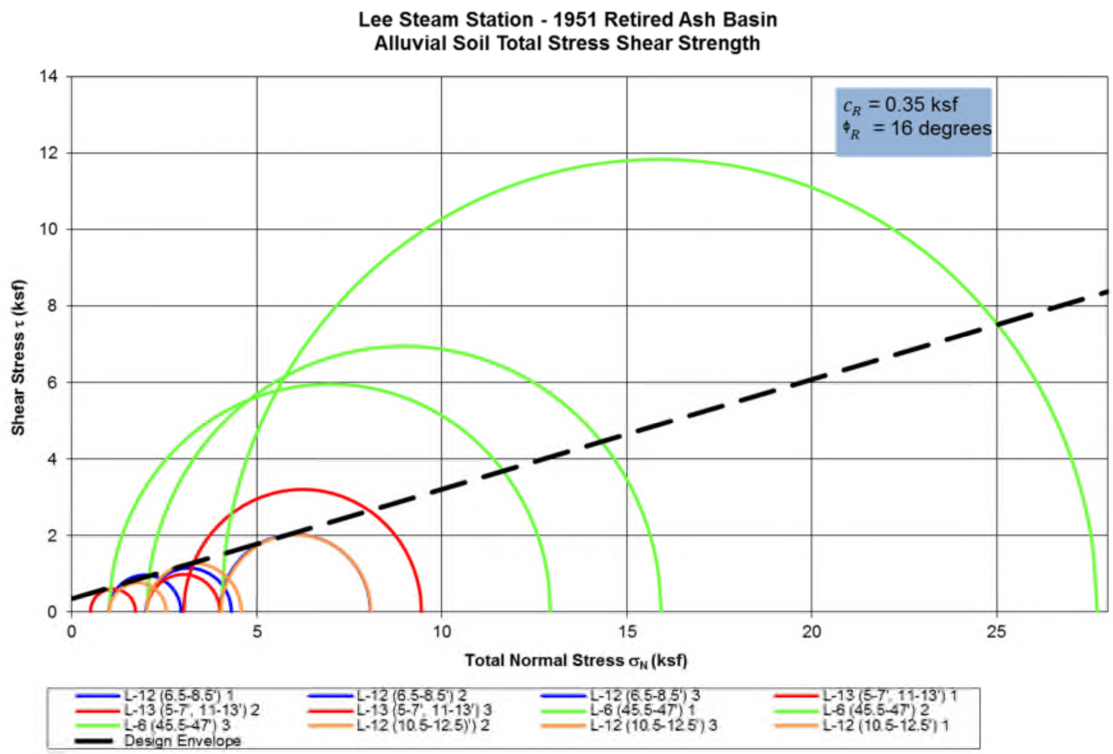


Figure 4.5: Alluvial Soil Total Stress Shear Strength

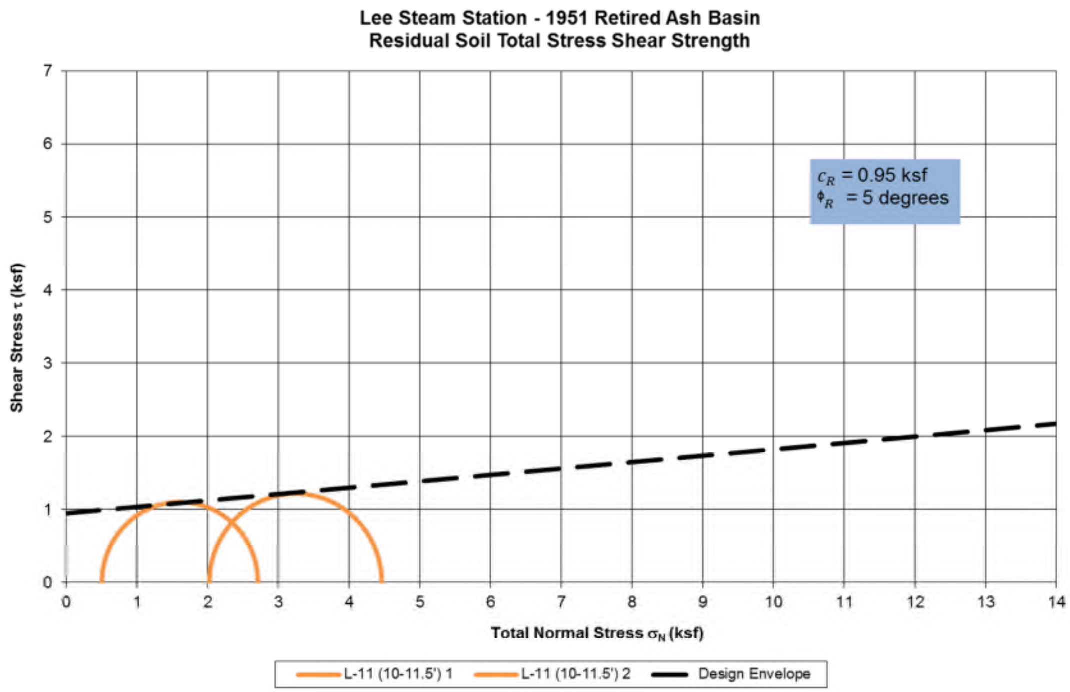


Figure 4.6: Residual Soil Total Stress Shear Strength

After merging effective strength envelopes for each soil type and averaging the initial specimen moist unit weights, effective strength parameters were estimated. The effective strength parameters used for analysis are presented in Table 5.1.

Similarly, the total strength envelopes for each soil type are merged and the mean of the initial specimen moist unit weights to tabulate total strength parameters. The total stress cohesion, C_R and friction angle, ϕ_R values are also presented in Table 5.1.

4.5.3 CU Triaxial Shear Strength Data for Ash Fill

S&ME plotted triaxial shear strength data for existing ash fill within the basin, which was established by WPC in 2008 (Reference 2.8). WPC sampled ash fill in various locations throughout the existing basin by excavating test pits. Remolded specimens were then compacted and molded in the laboratory; therefore, undisturbed data is not available at this time. Because the specimens were remolded via compaction techniques, the lower bound strength envelope was used for parameter correlation. S&ME derived shear strength relationships as a function of effective normal stress based on the five (5) tests that were performed by WPC in 2008. The effective shear strength and total shear strength data is presented in Figures 4.7 and 4.8, respectively.

A linear relationship for the effective shear strength, τ, relating to the ash fill (Figure 4.7), was interpreted from the CU triaxial tests based on WPC failure criteria (unknown). CU triaxial test results were compiled to obtain an overall estimate of effective strength parameters using a linear relationship as shown in the following figures.

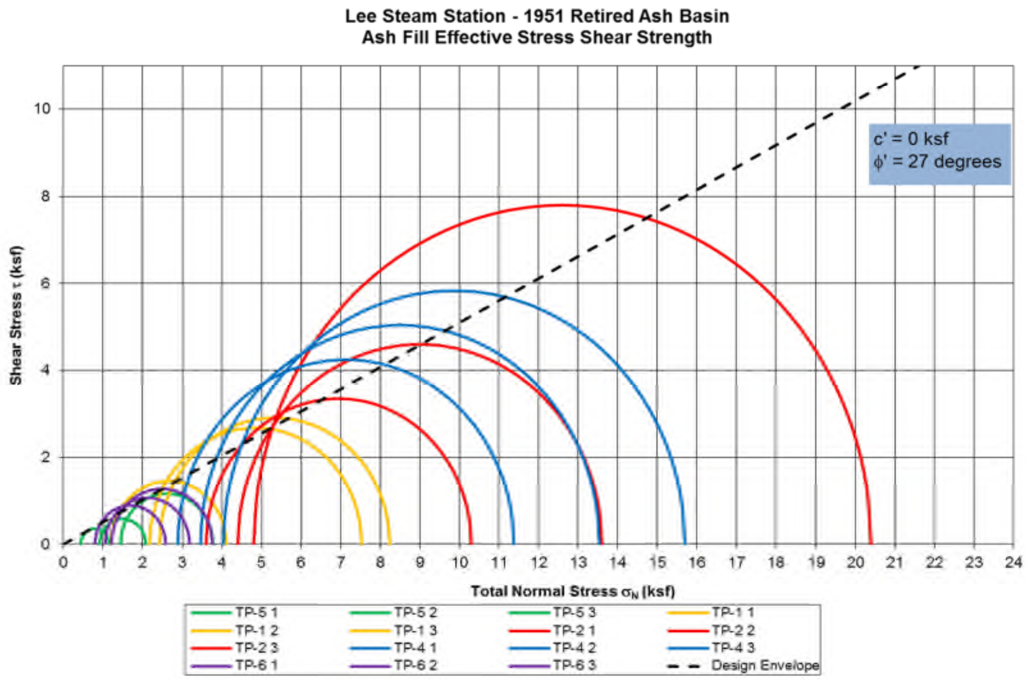


Figure 4.7: Ash Fill Soil Effective Stress Shear Strength

A Linear strength envelope was estimated for the total stress shear strength of the ash fill (Figure 4.8). The total stress Mohr's circles for each test were merged to develop a composite set of strength data. The total stress strength parameters, C_R and ϕ_R were generally defined based on total stress strengths interpreted at maximum deviator stress, generally at 15 percent vertical strain.

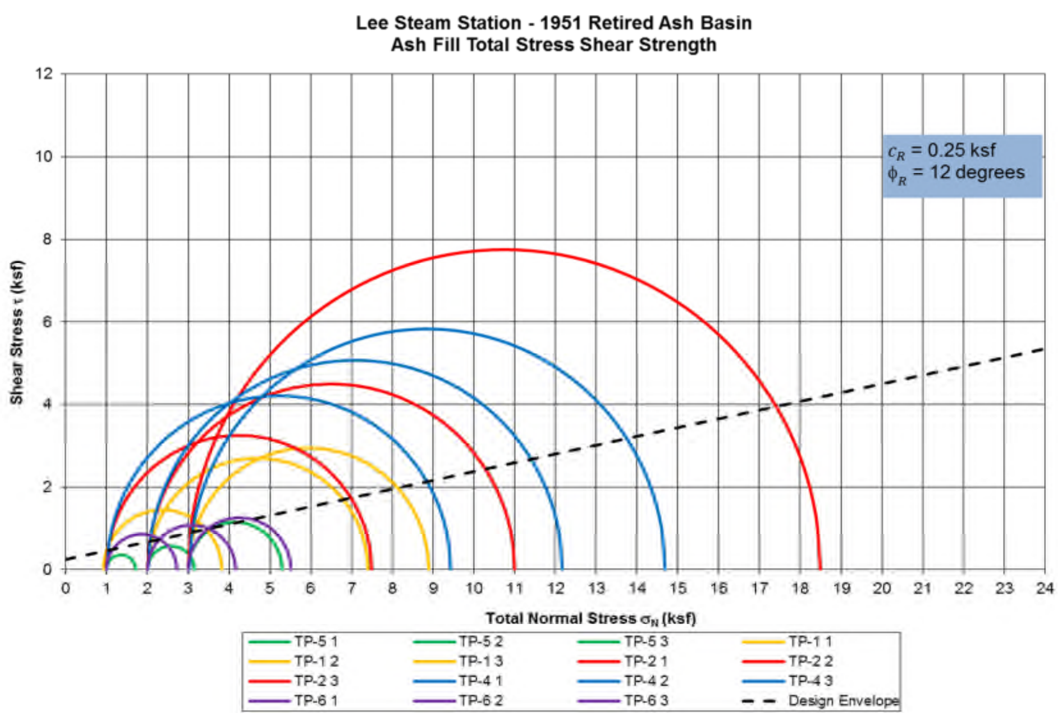


Figure 4.8: Ash Fill Soil Total Stress Shear Strength

The lower bound effective and total strength parameters used for analysis are presented in Table 5.1.

4.6 Seismic Conditions

Based on the USGS National Earthquake Information Center, the site is in an area with a low potential for seismic activity. Using the USGS Seismic Hazard Maps (Reference 2.1), the site has a 2% chance of experiencing a peak ground acceleration (PGA) in rock of approximately 0.14g in a 50-year period. The USGS Custom Hazard Map is depicted on Figure 7.

We have considered the site seismic conditions based on the International Building Code, 2012 Edition and Chapter 20, ASCE 7-10 Standard. Based on our interpretation, the divider dike area will have a Seismic Site Classification of D in accordance with IBC Section 1613.3.2 for the average properties within the upper 100 feet. Scaling the peak ground acceleration to account for the seismic site class results in a design PGA of 0.217g.

Accordingly to the USGS National Earthquake Information Center, the design event for the site has a moment magnitude (M_w or M) of 7.3.

5.0 SLOPE STABILITY ANALYSIS

The stability of the existing dike embankments was evaluated using the SLOPE/W computer program (Reference 2.7). Critical failure surfaces were determined using the Spencer Method, because it satisfies horizontal and vertical force equilibrium, as well as overall and individual slice moment equilibrium. Soil slopes often appear to fail on circular slip surfaces and in this particular instance we considered it reasonable to analyze slope stability using randomly generated circular slip surfaces using a search algorithm contained as a subroutine within the program. An optimization feature within the program that generates non-circular slip surface was also utilized to conservatively estimate the critical slip failure surfaces.

5.1 Loading Conditions

Slope stability analyses were performed for two (2) loading conditions. The analyzed loading conditions are as follows:

1. Steady State Seepage – Current Groundwater Level
2. Pseudo-Static Seismic Loading – Current Groundwater Level

Toe stability was performed on sections with steep toe slopes.

5.2 Dam Embankment Soil Parameters

Soil parameters incorporated into the slope stability analyses were based on S&ME soil laboratory data for embankment fill, alluvium and residual soils, and the WPC report for ash fill (Reference 2.8). Table 5.1 summarizes the soil shear strength using linear Mohr-Coulomb relationships for dam embankment materials.

Table 5.1: Soil Parameters (Linear Mohr-Coulomb)

Analysis Soil Type	Unit Wt., pcf	Saturated Unit Wt., pcf	Total Stress[1]		Effective Stress	
			ϕ_R, deg.	c_R, psf	ϕ', deg.	c', psf
Embankment Fill Soils	117.7	N/A	24	500	33	100
Alluvial Soils[2]	106	109	16	350	30	100
Residual Soil	102.3	105.3	5	950	40	0
Ash Fills	94.6	97.6	12	250	27	0

Notes: [1.] 80% of undrained (total) strengths (Reference 2.5) were used for loading case 2.
[2.] Cohesion was neglected for poorly graded alluvial sand.

5.3 Pseudo-Static Parameters

For a pseudo-static analysis, the cyclic loading or shaking generated by an earthquake is represented by a horizontal force applied to each slice of the potential failure mass. A factor of safety for the slope stability is calculated as with the static analysis with the inclusion of this lateral force acting on the slope material. A seismic coefficient, k, that is expressed as a fraction or percentage of gravity is used in the slope stability analysis to calculate the horizontal force that is applied. The seismic coefficient is determined based on a reference ground acceleration that is chosen based on the design earthquake (Section 4.4) and the method used for the pseudo-static analyses.

A horizontal seismic coefficient (k_h) of 0.10 was used for modeling during pseudostatic analyses based on previous Duke analysis for the existing primary/secondary ash basin dikes (Reference 2.13). This is slightly conservative when compared to the Hynes-Griffin Franklin method (Reference 2.5) that suggests using a coefficient equal to one-half of the base rock motion, or 0.07 g.

5.4 Acceptance Criteria

The minimum factors of safety (FS) for critical failure surfaces are presented in Table 5.2.

Table 5.2: Minimum Factor of Safety

Loading Condition	Minimum FS
1. Steady State Seepage – Current Groundwater Level	1.5[1]
2. Pseudo-Static Seismic Surcharge – Current Groundwater Level	1.0[2]

[1] – US Army Corps of Engineers, Slope Stability Engineer Manual, 2003.
[2] – Hynes-Griffin and Franklin, Rationalizing the Seismic Coefficient Method, 1984.

5.5 Assumptions

1. Two-dimensional limit equilibrium analysis.
2. Hydrostatic groundwater conditions based on S&ME boring and Duke Energy piezometer data. Seepage analysis was not performed.
3. Phreatic surface in steady state condition.
4. Drained (effective) strengths were used for Load case 1.
5. 80% undrained (total) strengths (Reference 2.5) were used for Loading Case 2. Critical failure surfaces generated are greater than 1 ft. in depth.

5.6 Results

The calculated factors of safety (FS) for the critical surfaces for each loading condition are presented in Table 5.3 and 5.4. SLOPE/W output data is included in Appendix E – SLOPE/W Output.

Table 5.3: Calculated Factors of Safety (Loading Condition 1)

Section	Analysis Region	Minimum FS	Calculated FS
A-A	Global Stability	1.5	2.5
B-B	Global Stability	1.5	1.5
B-B	Toe Stability (Lower Toe Area)	1.5	1.1
B-B	Toe Stability (Intermediate Terrace)	1.5	1.0
B-B	Toe Stability (Upper Terrace)	1.5	2.5
C-C	Global Stability	1.5	1.6
C-C	Toe Stability	1.5	1.1
D-D	Global Stability	1.5	1.7
E-E	Global Stability	1.5	2.1

Table 5.4: Calculated Factors of Safety (Loading Condition 2)

Section	Analysis Region	Minimum FS	Calculated FS
A-A	Global Stability	1.0	2.0
B-B	Global Stability	1.0	1.1
B-B	Toe Stability (Lower Toe Area)	1.0	1.3
B-B	Toe Stability (Intermediate Terrace)	1.0	1.4
B-B	Toe Stability (Upper Terrace)	1.0	1.5
C-C	Global Stability	1.0	1.1
C-C	Toe Stability	1.0	1.4
D-D	Global Stability	1.0	1.5
E-E	Global Stability	1.0	1.0

5.7 Discussion

5.7.1 Global Stability (Loading Case 1)

The global failure surfaces generated for all sections under static loading indicates a factor of safety of 1.5 or greater, which meets or exceeds the industry minimum.

5.7.2 Toe Stability (Loading Case 1)

Because of the configuration/geometry of the existing dikes at section B-B and C-C, additional analyses were performed to evaluate the stability at the toe. In the case of section B-B, this included stability in the lower toe, intermediate terrace and upper terrace. The upstream failure surfaces generated for each toe section indicates factors of safety of 1.04 to 1.11, which is below the industry minimum of 1.5. Note that the Intermediate Terrace at section B-B also fails to meet industry standard minimum factors of safety for the same reason as mentioned above. Because the dam embankment soil was modeled with a very small amount of cohesion (which is consistent with the laboratory strength data), existing ash and steep slope inclinations, shallow "surface sloughing" failures near the slope face or through the ash result in low factors of safety. Shallow surface sloughing, if promptly repaired, does not significantly affect the overall stability of the dam.

5.7.3 Seismic Stability (Loading Case 2)

The factors of safety for the seismic loading cases met or exceeded industry minimum standards.

5.8 Conclusions

In summarizing the stability evaluation results, the locations and conditions where factors of safety are less than industry-recommended standards are as follows:

- Section B-B (Lower Toe and Intermediate Terrace)
- Section C-C (Toe Area)

Generally speaking, these below-standard factors of safety are exhibited on shallower, sloughing type failures, with deeper-seated, more global-level failure surfaces exhibiting higher factors of safety. It should be noted that the industry-recommended standard is referenced to the US Army Corps of Engineers Engineering Manual 1110-2-1902 (Reference 2.4) for new earth and rock-fill dams. Section 3-3 of the Manual addresses existing embankment dams and the emphasis not being placed solely on slope stability analysis, but rather historical behavior/performance of the dam. Lower factors of safety for slope stability of existing dams can be acceptable based on past slope performance.

The lower factors of safety for the shallower, sloughing type failures are consistent geotechnical expectations, given the upstream embankment slope(s) inclinations, composition and modeling of the soil's effective shear strength with primarily a frictional component only (i.e., little cohesion). Shallower surface sloughs are generally not detrimental to the overall integrity of the dam, provided they are promptly repaired, as they have been on the subject dam(s). However, since toe stability near the River are below industry standards, modifications should be considered to increase stability.

We would like to point out that the existing embankment is wooded with moderate to large deciduous trees and light underbrush (in some areas). While existing vegetation is likely providing some shallow stabilization in areas where root systems penetrate the embankment soils, this is difficult to quantify in actual slope stability analysis and is more of a qualitative indication of slope stability improvement.

6.0 LIQUEFACTION SCREENING ANALYSIS

The method used to evaluate liquefaction potential is in general accordance with that proposed by Youd et al. 2001 (Reference 2.9). To evaluate the liquefaction potential of existing subgrade materials, the cyclic stress ratio induced by a seismic event (CSR_{EQ}) was compared to the cyclic resistance ratio (CRR) of subgrade soils, as developed from CPT data. The CLiq computer program (Reference 2.9) was used to evaluate liquefaction potential for the three CPT locations. The following outlines the general Youd et al. procedure.

6.1 Calculations

The factor of safety against liquefaction was estimated using Equation 1 as described by Youd et al., 2001 (Reference 2.9).

$$FS_{Liquefaction} = \left(\frac{CRR_{7.5}}{CSR_{EQ}}\right) * MSF * K_{\sigma} * K_{\alpha} \qquad \text{(Equation 1)}$$

Where:

$FS_{Liquefaction}$ = factor of safety against liquefaction triggering;
$CRR_{7.5}$ = cyclic resistance ratio developed from CPT data;
CSR_{EQ} = cyclic stress ratio induced by the design seismic event;
MSF = magnitude scaling factor;
K_{σ} = overburden correction factor; and
K_{α} = correction factor for sloping ground, assumed to be 1.0 for this analysis.

Describe the overburden correction factor (K_{σ})

The overburden correction factor was estimated using Equation 2 as described by Youd et al., 2001 (Reference 2.9).

$$K_{\sigma} = \left(\frac{\sigma_{vo}'}{P_a}\right)^{(f-1)} \qquad \text{(Equation 2)}$$

Where:

σ_{vo}'= effective vertical stress (kPa);
P_a = atmospheric pressure (kPa); and,
f = ranges from 0.7-0.8 for relative densities ranging from 40% to 60% and from 0.6 to 0.7 for relative densities ranging from 60% to 80%, respectively.

Describe the magnitude scaling factor (MSF)

The cyclic resistance ratio equations are applicable for magnitude 7.5 seismic events. The magnitude scaling factor (MSF) is a correction for seismic events of magnitude other than 7.5. Based on Section 4.6, a seismic event with a moment magnitude of 7.3 was chosen for design based on the 1886 Charleston, South Carolina event. A lower-bound MSF relationship was estimated using Equation 3 as described by Youd et al., 2001 (Reference 2.9).

$$MSF = \left(\frac{10^{2.24}}{M_w^{\ 2.56}}\right) \qquad \text{(Equation 3)}$$

Where:

M_w = Moment magnitude.

Describe the cyclic stress ratio (CSR_{EQ})

The seismic demand on a soil layer is expressed in terms of the cyclic stress ratio (CSR_{EQ}) and was estimated using Equation 4 as described by Youd et al., 2001 (Reference 2.9).

$$CSR_{EQ} = 0.65 * \frac{a_{max}}{g} * r_d * \left(\frac{\sigma_{vo}}{\sigma_{vo}'} \right) \qquad \text{(Equation 4)}$$

Where:

a_{max} = peak ground surface, or free-field, acceleration (m/s^2);
g = acceleration of gravity (m/s^2);
r_d = stress reduction coefficient;
σ_{vo} = total vertical stress (kPa); and
σ_{vo}'= effective vertical stress (kPa).

The stress reduction coefficient accounts for flexibility of the soil profiles and was estimated using Equation 5 as described by Youd et al., 2001 (Reference 2.9).

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \qquad \text{(Equation 5)}$$

Where:

z = Depth (m).

Describe the cyclic resistance ratio (CRR7.5)

The cyclic resistance ratio represents the capacity of the soil to resist liquefaction for a seismic event with a moment magnitude of 7.5. The magnitude scaling factor (MSF) will modify this value for the site-specific seismic event, as previously discussed in this calculation. The cyclic resistance ratio was estimated using Equations 6a and 6b as described by Youd et al., 2001 (Reference 2.9).

$$(q_{c1N})_{CS} < 50 \rightarrow CRR_{7.5} = 0.833\left[\frac{(q_{c1N})_{CS}}{1000}\right] + 0.05 \qquad \text{(Equation 6a)}$$

$$50 \le (q_{c1N})_{CS} < 160 \rightarrow CRR_{7.5} = 93\left[\frac{(q_{c1N})_{CS}}{1000}\right]^3 + 0.08 \qquad \text{(Equation 6b)}$$

Where:

$(q_{c1N})_{CS}$ = clean-sand cone penetration resistance normalized to atmospheric pressure.

The normalized clean-sand cone penetration resistance $(q_{c1N})_{CS}$ was estimated using Equation 7 as described by Youd et al., 2001 (Reference 2.9).

$$(q_{c1N})_{CS} = K_C q_{C1N} \qquad \text{(Equation 7)}$$

Where:

K_C = correction factor for grain characteristics; and
q_{c1N} = the normalized CPT tip resistance.

The correction factor for grain characteristics transforms the normalized CPT tip resistance (q_{c1N}) into a clean-sand equivalent value ($(q_{c1N})_{CS}$), and was estimated using Equations 8a and 8b as described by Youd et al., 2001 (Reference 2.9).

$$I_c \leq 1.64 \rightarrow K_c = 1.0 \qquad \text{(Equation 8a)}$$

$$I_c > 1.64 \rightarrow K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \qquad \text{(Equation 8b)}$$

Where:

I_c = soil behavior type index.

The soil behavior type index is calculated in a step-wise approach using Equations 9, 10, and 11 as described by Youd et al., 2001 (Reference 2.9).

$$I_c = \left[(3.47 - \log Q)^2 + (1.22 + \log F)^2\right]^{0.5} \qquad \text{(Equation 9)}$$

$$Q = \left[\frac{q_c - \sigma_{v0}}{P_a}\right]\left[\left(\frac{P_a}{\sigma'_{v0}}\right)^n\right] \qquad \text{(Equation 10)}$$

$$F = 100\left(\frac{f_s}{q_c - \sigma_{v0}}\right) \qquad \text{(Equation 11)}$$

Where:

q_c = cone tip resistance;
f_s = sleeve resistance;
σ_{vo} = total vertical stress (kPa);
σ_{vo}'= effective vertical stress (kPa);
P_a = atmospheric pressure; and
n = exponent based on soil type (1.0 for clays, 0.5 for granular soils, and 0.7 for silts).

The soil behavior type index is calculated using Steps 1 through 3 as outlined below:

Step 1: Assume a clayey soil (n = 1.0) and calculate I_c. If $I_c > 2.6$, the soil is classified as too clay-rich to liquefy and the calculation is complete. If $I_c < 2.6$, proceed to Step 2.

Step 2: Assume a granular soil (n = 0.5) and calculate Ic. If Ic < 2.6, the soil is classified as granular and this Ic value is used during liquefaction analyses. If Ic > 2.6 the soil is likely silty, and Ic is calculated in Step 3.

Step 3: Assume a silty soil (n = 0.7) and calculate Ic, this Ic value is used during liquefaction analyses.

The normalized CPT tip resistance is calculated using Equations 12 and 13 as described by Youd et al., 2001 (Reference 2.9).

$$q_{c1N} = C_Q\left(\frac{q_c}{P_a}\right) \qquad \text{(Equation 12)}$$

$$C_Q = \left(\frac{P_a}{\sigma_{vo}'}\right)^n \qquad \text{(Equation 13)}$$

Where:

C_Q = normalizing factor;
q_c = field tip resistance value;
P_a = atmospheric pressure;
σ_{vo}'= effective vertical stress (kPa); and
n = exponent based on soil type as calculated in section 5.1.2 of this calculation.

In addition to the Youd et. al procedure, additional screening criteria is used to help evaluate liquefaction potential. Particle size/distribution, age of the deposit and plasticity can help determine liquefaction potential. As previously stated, no known liquefaction has been documented in residual deposits. Table 1 presented in Reference 2.11 described saturated soils within Pleistocene and Pre-Pleistocene geologic periods have a "very low" susceptibility of liquefaction, while compacted fill has a "low" likelihood. For fine grained soils, if the plasticity index is 7 or greater, then the soils can reasonably be expected to behave like a clay (Reference 2.11) and would be more resistant to liquefaction.

6.2 Acceptance Criteria

A factor of safety against liquefaction ($FS_{Liquefaction}$) of ≥ 1.2 (Reference 2.11) or I_c greater than 2.6 was considered acceptable (Reference 2.9). If $FS_{Liquefaction}$ values are less than 1.2 are generated, then further evaluation using the deposit age, published literature, SPT and/or laboratory data, and I_c value was performed to help further evaluate liquefaction potential.

6.3 Assumptions

The thirteen (13) CPT soundings were evaluated for liquefaction potential for the existing conditions case, assuming no surcharge or water drawdown. It was assumed that small zones of liquefiable material may not be thick enough to cause a significant decrease in soil strength; therefore, materials represented by a CPT sounding were identified as not liquefiable as long as liquefiable zones were generally isolated and less than one foot in thickness. In addition, it was assumed that because of the relatively short dam height (approximately 25 to 45 ft.) that the ground acceleration is representative of accelerations in the embankment.

6.4 Results

Each of the CPT locations was evaluated for liquefaction susceptibility for a design seismic event having a magnitude of M 7.3 and a pga of 0.217g (Section 4.6) for the existing loading conditions using the CLiq software. The results of the analyses are summarized in Table 6.1, with the full analysis results contained in Appendix F – Liquefaction Analysis Report.

Table 6.1: Liquefaction Screening Summary

Boring ID	Approximate Depth (ft.)	Factor of Safety FS_{liq}	STBn Index I_c	USCS Classification / Symbol	Percent Finer #200 Sieve	Liquefaction Potential	Reasoning
L-1	34 - 34.5	< 1	1.75	FILL - Silty SAND with ash (SM)	-	Unlikely	Layer thickness < 1 ft.
	35.1 - 35.2	< 1.2	2.6	FILL - Silty SAND with ash (SM)	-	Unlikely	Layer thickness < 1 ft./Ic=2.6
L-2	-	-	-	-	-	None	
L-3	48 - 50	< 1	2.0	ALLUVIUM - Poorly Graded SAND (SP)	2.6	Possible	FS < 1
L-4	44.2 - 44.8	1.15	2.5	ALLUVIUM - Silty SAND (SM)	-	Unlikely	Layer thickness < 1 ft.
	46 - 47.5	1.1	2.4	ALLUVIUM - Silty SAND (SM)	-	None	PI > 7
	48.9 - 49	1.2	2.3	ALLUVIUM - Silty SAND (SM)	18.6	None	FS ≥ 1.2
L-5	41.4 - 43.5	> 1.2	2.5	ALLUVIUM - Poorly Graded SAND (SP)	-	None	FS ≥ 1.2
	44 - 46.2	> 1.2	2.2	ALLUVIUM - Poorly Graded SAND (SP)	10.8	None	FS ≥ 1.2
L-6	-	-	-	-	-	None	
L-7	44.9 - 47.2	> 1.2	2.2	ALLUVIUM - Silty SAND (SM)	-	None	FS ≥ 1.2
	47.2 - 47.8	< 1.2	2.15	ALLUVIUM - Silty SAND (SM)		Unlikely	Layer thickness < 1 ft.
	47.8 - 48.2	> 1.2	2.1	ALLUVIUM - Silty SAND (SM)		None	FS ≥ 1.2
	48.2 - 48.5	< 1.2	2.2	ALLUVIUM - Silty SAND (SM)	32.3	Unlikely	Layer thickness < 1 ft.
L-8	-	-	-	-	-	None	
L-9	-	-	-	-	-	None	
L-10	-	-	-	-	-	None	
L-11	12.8 - 13.2	> 1.2	4	PWR	-	None	FS ≥ 1.2
L-12	11.3 - 12.6	1.2	1.75	ALLUVIUM - Clayey SAND (SC)	36.7	None	FS ≥ 1.2
	13 - 13.6	1.5	1.9	ALLUVIUM - Poorly Graded SAND (SP)	3.8	None	FS ≥ 1.2
L-13	12.5 - 13	< 1	2.3	ALLUVIUM - Sandy Lean CLAY (CL) Plasticity Index (PI) = 16	67.8	None	PI > 7

6.5 Discussion

Based on the results of the liquefaction screening, liquefiable layers ($FS_{Liquefaction} \leq 1.2$) were identified in CPT soundings L-1, L-3, L-4, L-5, L-7, L-11, L-12 and L-12, generally within existing alluvium near to and just below the water table. After reviewing the liquefaction data, eight (8) layers that were flagged as potentially liquefiable layers were excluded and deemed not liquefiable, since the factor of safety met the minimum industry standard. Five (5) remaining zones were listed as having an "unlikely" liquefaction potential based on "thin" layer thickness and/or I_c value of 2.6 or greater, and two (2) are listed as "none" because plasticity index was 7 or greater (Reference 2.11). The remaining zone encountered in L-3 was listed as having a "possible" liquefaction potential. Based on the CPT data from sounding L-3, there appears to be an approximately 2-ft. thick layer of "clean" alluvial sand that has a factor of safety less than 1.

6.6 Conclusions

Based on the CPT sounding data and methodologies described herein, relatively thin layers of soils considered to have a potential to liquefy during a seismic event were identified within the existing dike foundation materials. With the exception of sounding L-3, the zones of liquefiable material do not appear to be thick enough to cause a significant decrease in soil strength and would be more resistant to liquefaction. The layer in boring L-3 from 48 to 50 ft. is thicker and liquefaction cannot be ruled out. Accordingly, a post-seismic analysis was performed for Section D-D using reduces shear strengths. The procedure and results of those analyses are discussed in Section 6.7.

6.7 Post-Seismic Analysis

A post-seismic stability analysis was performed for Section D-D to evaluate the static slope stability of the embankment following a seismic event. During liquefaction, the shear strengths within the liquefied zone are reduced. Assuming a seismic event occurs and liquefies the lower alluvial layer in Boring L-3 (48 to 50 ft. per liquefaction results), the liquefied shear strength parameters are estimated and input into SLOPE/W as a function of vertical effective stress. Accordingly, the ratio of liquefied shear strength to prefailure effective vertical stress ($\frac{\tau}{\sigma_v'}$) was input as the soil strength parameter for the liquefied layer.

Using Olsen and Mitchell's relationship (Reference 2.14), the normalized tip resistance, $q_{c,1}$, must first be calculated based on CPT data to determine $\frac{\tau}{\sigma_v'}$:

$$q_{c,1} = C_q q_c,$$

where $C_q = \left(\frac{P_a}{\sigma_v'}\right)^c$ and is known as the normalization factor (Reference 2.14). Note that P_a and σ_v' are atmospheric and overburden pressure, respectively. The constant, c, is known as the normalizing exponent and is extrapolated based on the friction ratio. The friction ratio is calculated as:

$$R_f = \left(\frac{f_s}{q_c}\right) * 100.$$

The variable, f_s, is the raw sleeve resistance, when averaged is approximately 1,000 psf at the 48 to 50 ft. depth (Refer to CPT Logs Appendix B). The raw average of the tip resistance within the 48 to 50 ft. depth is 100,000 psf. Therefore, R_f is calculated as:

$$R_f = \left(\frac{f_s}{q_c}\right) * 100 = \left(\frac{1{,}000\ psf}{100{,}000\ psf}\right) * 100 = 1\%.$$

The normalizing exponent, c, is found using the normalizing exponent contours in Figure 6.1 (Reference 2.14).

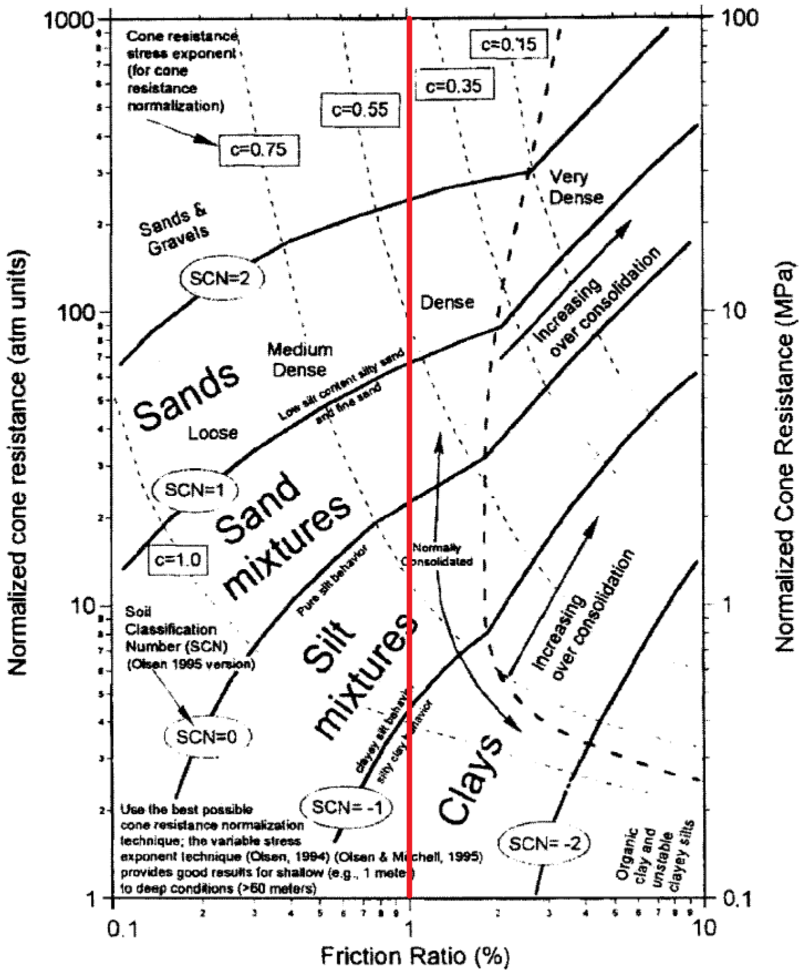


Figure 6.1: Variable CPT Normalization (Reference 2.14)

The normalizing exponent, c, is extrapolated to be 0.40 from the normalizing exponent contours. Next, the normalization factor is tabulated:

$$C_q = \left(\frac{P_a}{\sigma_v'}\right)^c = \left(\frac{2{,}000\,psf}{5{,}808\,psf}\right)^{.40} = 0.653.$$

Finally, the normalized tip resistance, $q_{c,1}$, is calculated as:

$$q_{c,1} = C_q q_c = 0.653 * 100{,}000\,psf = 65{,}300\,psf = 3.1\,MPa.$$

Figure 6.2 is used to back-calculate the liquefied strength ratio, $\frac{\tau}{\sigma_v'}$ (Reference 2.5).

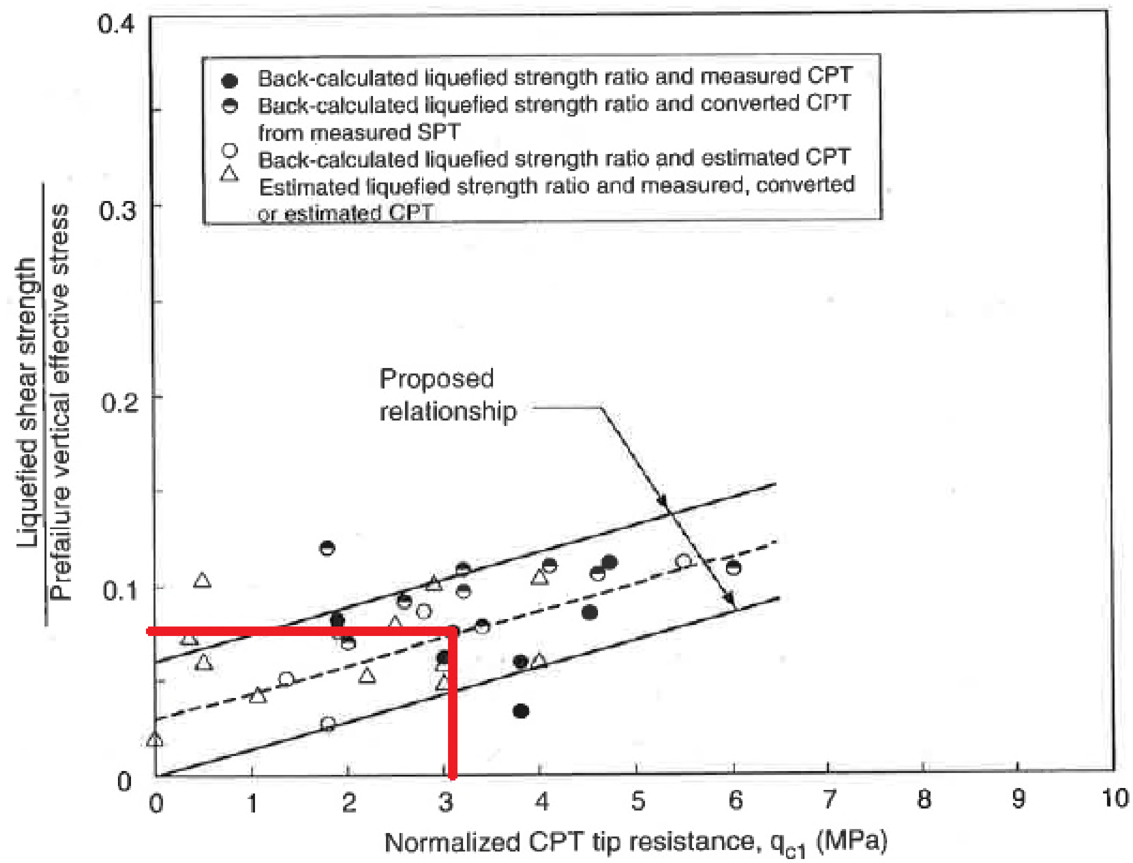


Figure 6.2: Liquefied Strength Ratio Relationship Based on Normalized CPT Tip Resistance

The constant function $\frac{\tau}{\sigma_v'} = 0.075$ (as generated from Figure 6.2) is input into SLOPE/W for post-seismic stability analysis for the liquefiable soil layer. Although the layer identified by the liquefaction screening was between 48 and 50 ft., for the SLOPE/W analysis the entire alluvial layer from 47 to 51 ft. (as identified in the soil test boring) was evaluated.

Using the reduced shear strengths calculated by the previous procedure, the section was analyzed using the SLOPE/W program. The calculated factors of safety (FS) for the critical surfaces for the post-seismic event are presented in Table 6.2. SLOPE/W output data is included in Appendix E – SLOPE/W Output.

Table 6.2: Calculated Factors of Safety (Post-Seismic Loading)

Section	Analysis Region	Minimum FS	Calculated FS
D-D	Global Stability (Effective Parameters)	1.0	1.2
D-D	Global Stability (Total Parameters)	1.0	1.2

Note: 80% undrained (total) strengths (Reference 2.5) were used.

Based on the post-seismic analysis, the global failure surfaces generated for section D-D under static loading indicates a factor of safety of 1.2, which exceeds the industry minimum of 1.0 for post seismic instability (Reference 2.16). Post seismic vertical settlement is calculated to be approximately 0.06 in., which is considered negligible.

7.0 RECOMMENDATIONS

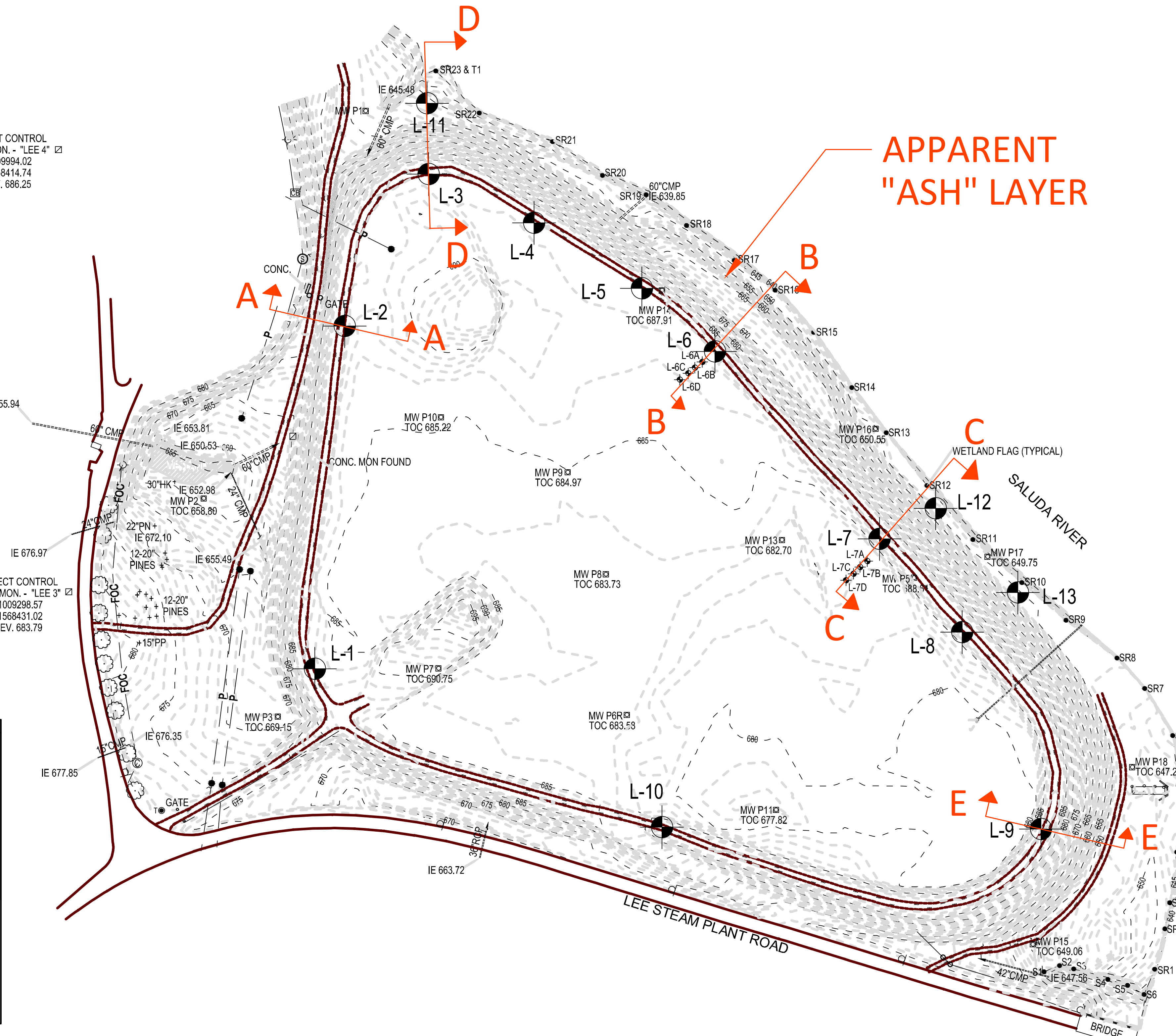
We recommend monitoring the performance of the dikes to observe changing conditions and/or performance issues. In the short-term, planned additions of rip-rap protection and/or armoring along the River at Sections C-C and B-B may help improve conditions against shallow surface sloughing locally at these levels previously caused by local loss of passive resistance as a result of erosion along the shoreline. If increasing factors of safety to industry-standards is desired or required, significant buttressing and/or re-construction of the downstream embankment(s) to flatter slopes would be required.

Additional data and surveying may be necessary to verify the existing slope (along the Saluda River) is as steep as recent surveys indicate and also define the topography further into the River. Also, additional data in the apparent “ash layer” would provide insight as to the soil composition of the region. Slope stability results did indicate surface sloughing along the apparent “ash layer” is possible (FS = 1.04). As previously mentioned, the apparent “ash” was modeled with an underlying ash embankment fill strata because data was un-retrievable along that portion of the slope embankment.

8.0 LIMITATIONS OF REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

FIGURES



LEGEND

- ASPHALT PAVEMENT
- CHAIN LINK FENCE WITH GATE POST
- CONCRETE MONUMENT
- 684 EXISTING CONTOURS (1' INTERVAL)
- FIBER OPTIC BOX
- FIBER OPTIC LINE
- FIRE HYDRANT
- GRAVEL DRIVE
- LIGHT POLE
- MW P8 TOC 683.73 MONITORING WELL WITH TOP OF CONCRETE ELEVATION
- OVERHEAD POWER LINE WITH UTILITY POLE AND GUY WIRE CONNECTION
- SIGN
- SOIL BORING
- TELEPHONE PEDESTAL
- TREE
- TREELINE
- WATER VALVE
- WETLAND
- WOODEN FENCE
- MONITORING WELL
- BORING LOCATION
- SLOPE STABILITY ANALYSIS SECTION

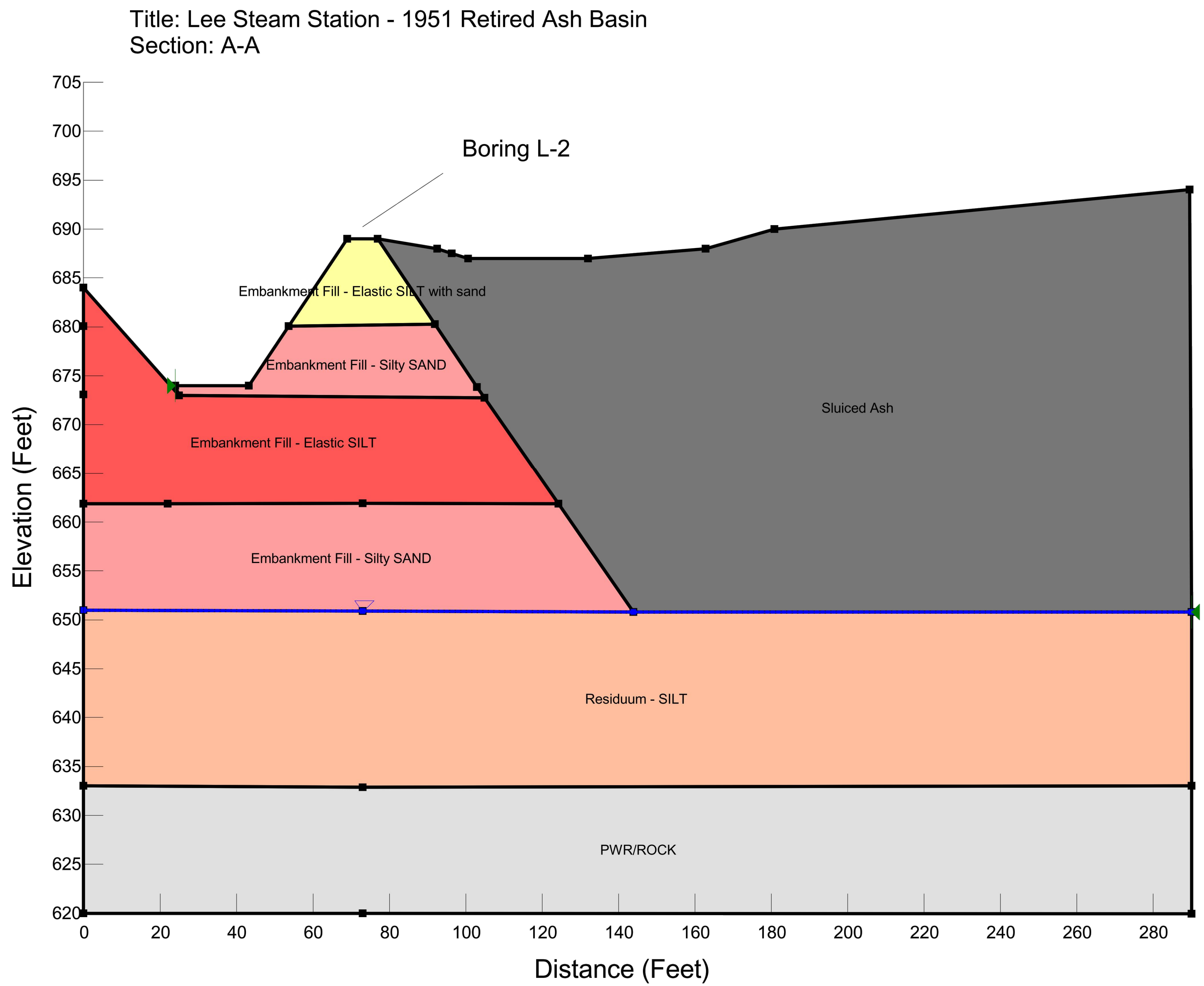
SOIL BORING TABLE

ID #	NORTHING	EASTING	ELEVATION
L-1	1009177.53	1568818.13	687.4
L-2	1009716.45	1568865.17	688.9
L-3	1009955.37	1568997.31	687.3
L-4	1009878.79	1569163.23	687.5
L-5	1009775.66	1569332.93	687.3
L-6	1009676.49	1569447.77	688.0
L-6A	1009659.99	1569428.08	686.6
L-6B	1009651.01	1569416.38	686.6
L-6C	1009642.07	1569405.92	686.4
L-6D	1009632.08	1569392.56	685.8
L-7	1009381.69	1569706.79	688.2
L-7A	1009346.69	1569687.70	682.0
L-7B	1009337.52	1569676.35	682.0
L-7C	1009326.92	1569666.03	682.0
L-7D	1009316.66	1569654.11	682.0
L-8	1009235.14	1569836.85	688.7
L-9	1008925.37	1569960.79	687.9
L-10	1008928.55	1569364.70	686.8
L-11	1010066.71	1568994.80	650.5
L-12	1009429.67	1569795.21	648.8
L-13	1009297.51	1569924.63	649.8

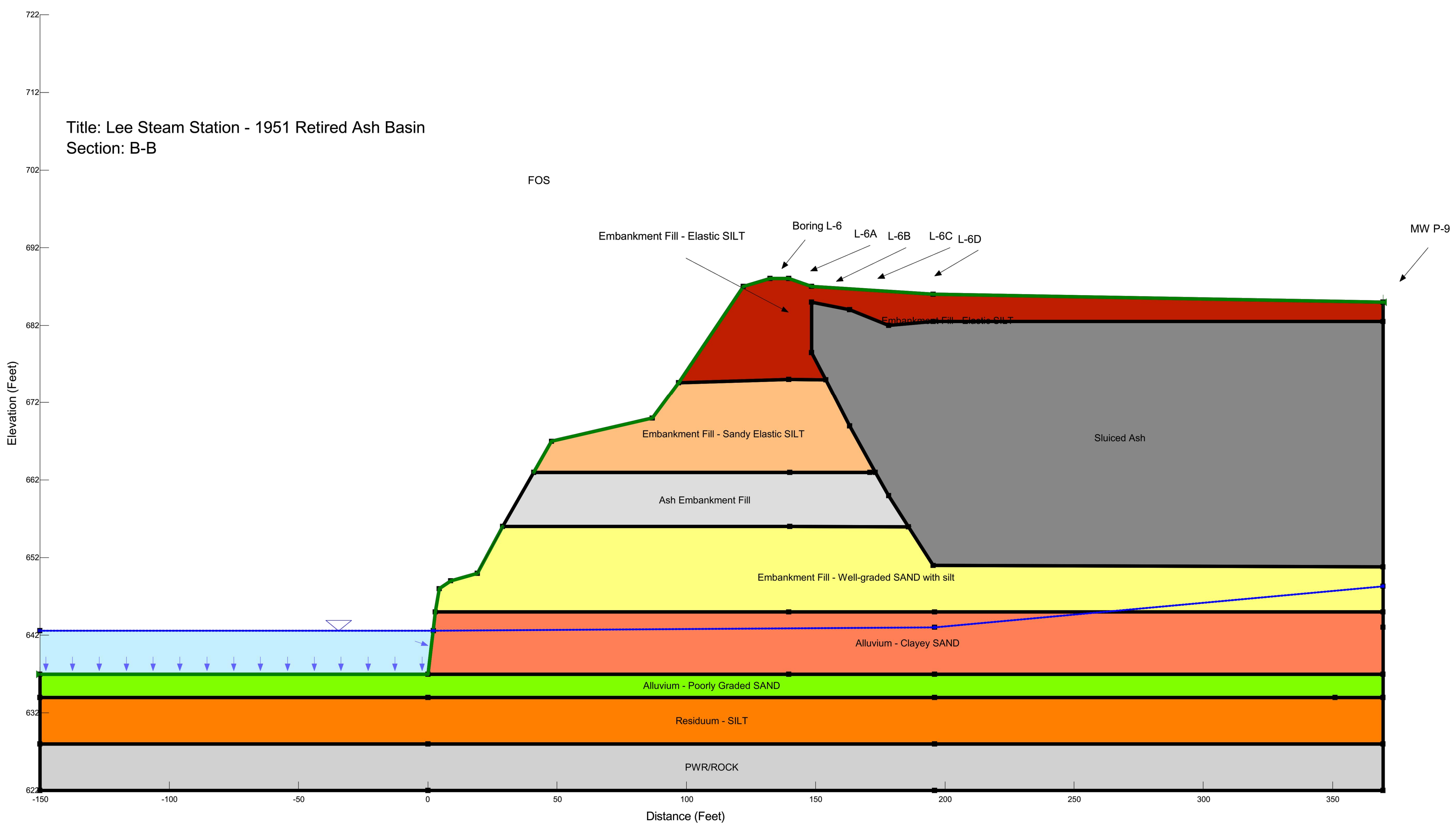
NOTES:

1.) THIS SURVEY REPRESENTS THE EXISTING PLANIMETRIC AND TOPOGRAPHIC FEATURES ONLY. IT DOES NOT INCLUDE EASEMENTS, RIGHTS OF WAY OR PROPERTY LINES.
2.) UTILITIES AS SHOWN WERE OBVIOUS AND APPARENT AT THE TIME OF THE SURVEY BUT THERE MAY BE OTHER UTILITIES PRESENT ON THE SITE THAT ARE NOT SHOWN.
3.) COORDINATES AND ELEVATIONS ARE BASED ON USGS MONUMENT "SR 2" USING THE NAD83(2011) HORIZONTAL DATUM AND THE NAVD88 VERTICAL DATUM.

SCALE: AS SHOWN	DATE: 08-28-14	S&ME 301 ZIMA PARK DRIVE 29301 PH. (864) 574-2360 FAX. (864) 576-8730 WWW.SMEINC.COM	LEE STEAM STATION – 1951 RETIRED ASH BASIN STABILITY ANALYSIS SECTION AND TEST LOCATION PLAN BELTON, S.C.
PROJECT NO. 7126-14-005	DRAWN BY: F. MORRIS		FIGURE NO. 1
	CHECKED BY: M. REVIS		



SCALE: AS SHOWN	DATE: 08-28-14	S&ME	301 ZIMA PARK DRIVE 29301 PH. (864) 574-2360 FAX. (864) 576-8730 WWW.SMEINC.COM	LEE STEAM STATION – 1951 RETIRED ASH BASIN STABILITY ANALYSIS SECTION A-A BELTON, S.C.
PROJECT NO. 7126-14-005	DRAWN BY: F. MORRIS			FIGURE NO. 2
	CHECKED BY: M. REVIS			



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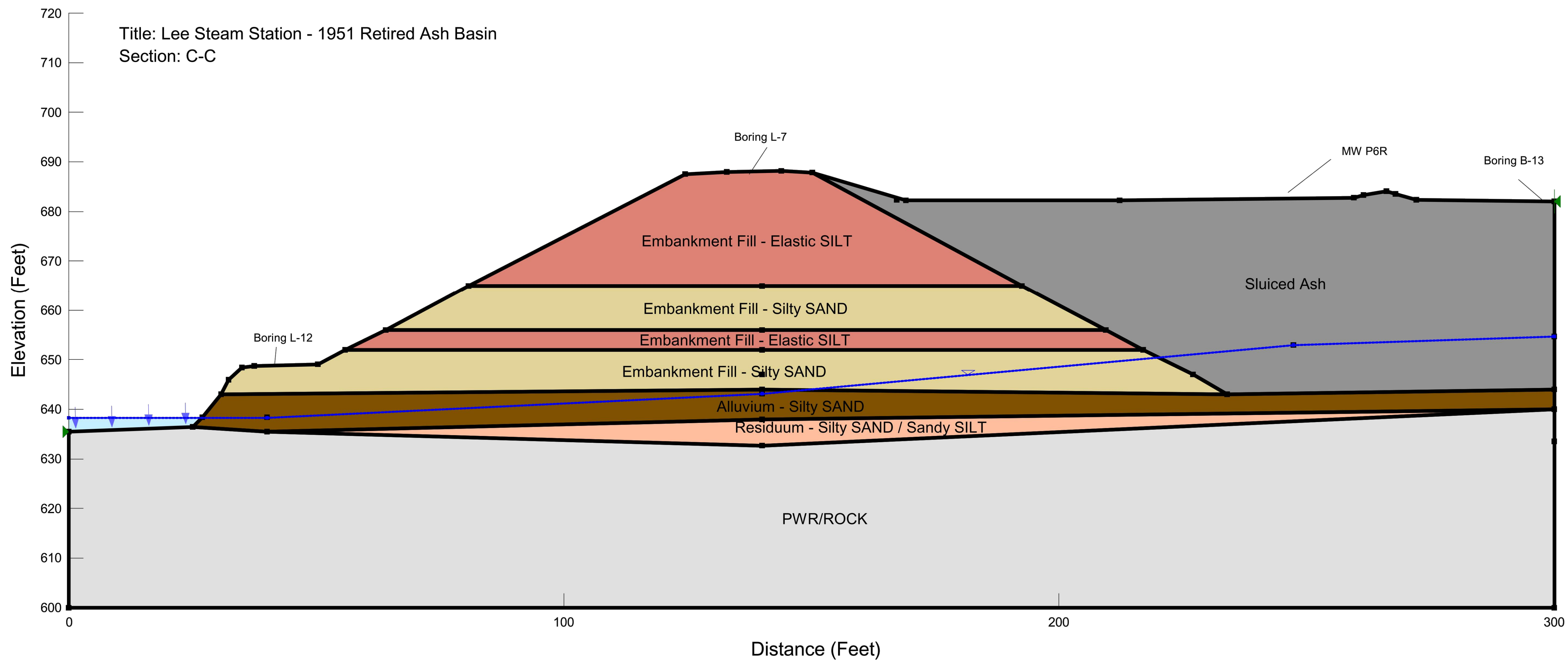
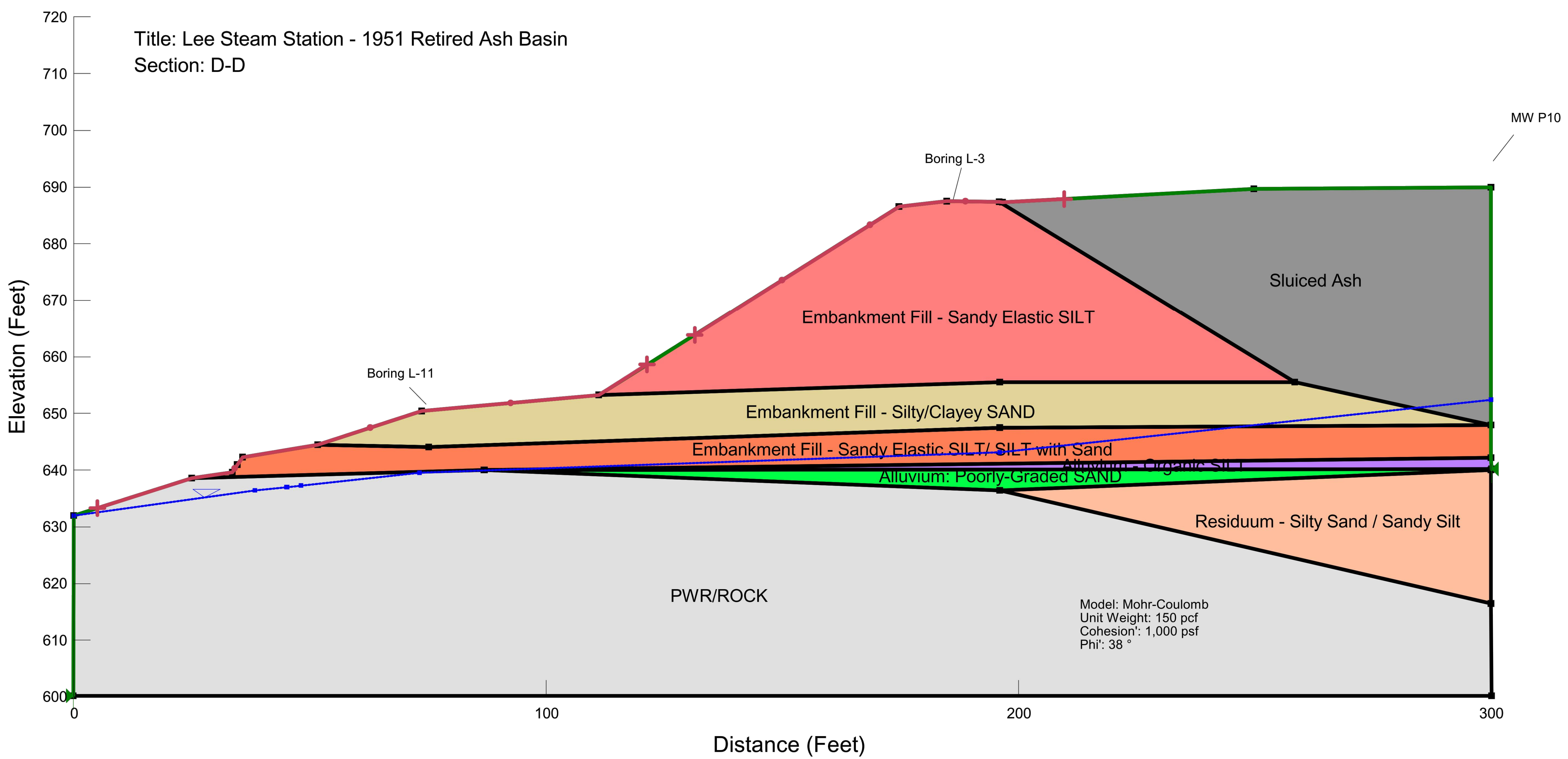
Jan 23 2018

SCALE: AS SHOWN	DATE: 08-28-14	S&ME	301 ZIMA PARK DRIVE 29301 PH. (864) 574-2360 FAX. (864) 576-8730 WWW.SMEINC.COM	FIGURE NO. 3
PROJECT NO. 7126-14-005	DRAWN BY: F. MORRIS			
	CHECKED BY: M. REVIS			

LEE STEAM STATION – 1951 RETIRED ASH BASIN
STABILITY ANALYSIS SECTION B-B
BELTON, S.C.



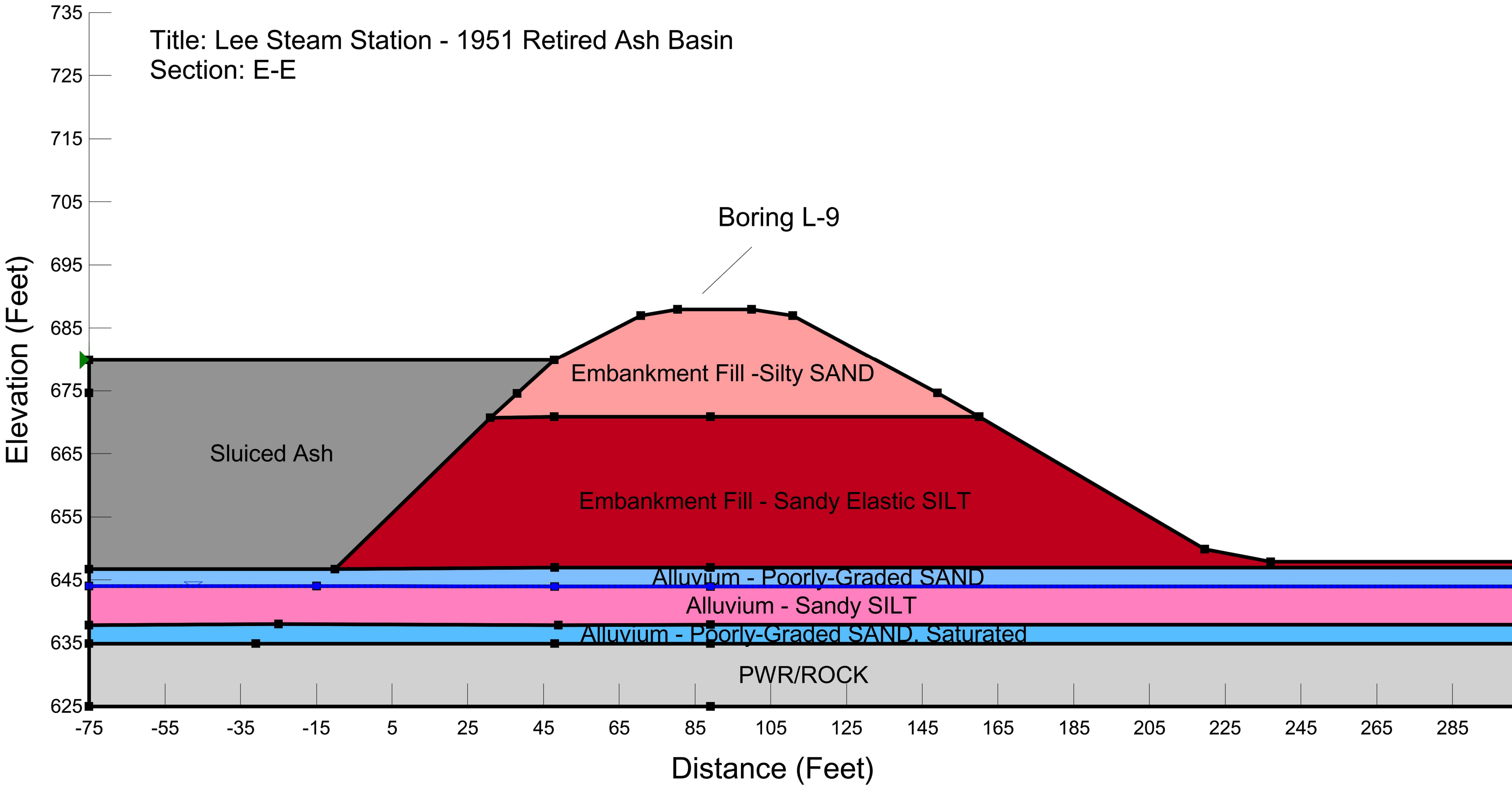
SCALE: AS SHOWN	DATE: 08-28-14	S&ME	301 ZIMA PARK DRIVE 29301 PH. (864) 574-2360 FAX. (864) 576-8730 WWW.SMEINC.COM	LEE STEAM STATION – 1951 RETIRED ASH BASIN STABILITY ANALYSIS SECTION C-C BELTON, S.C.	FIGURE NO. 4
PROJECT NO. 7126-14-005	DRAWN BY: F. MORRIS				
	CHECKED BY: M. REVIS				



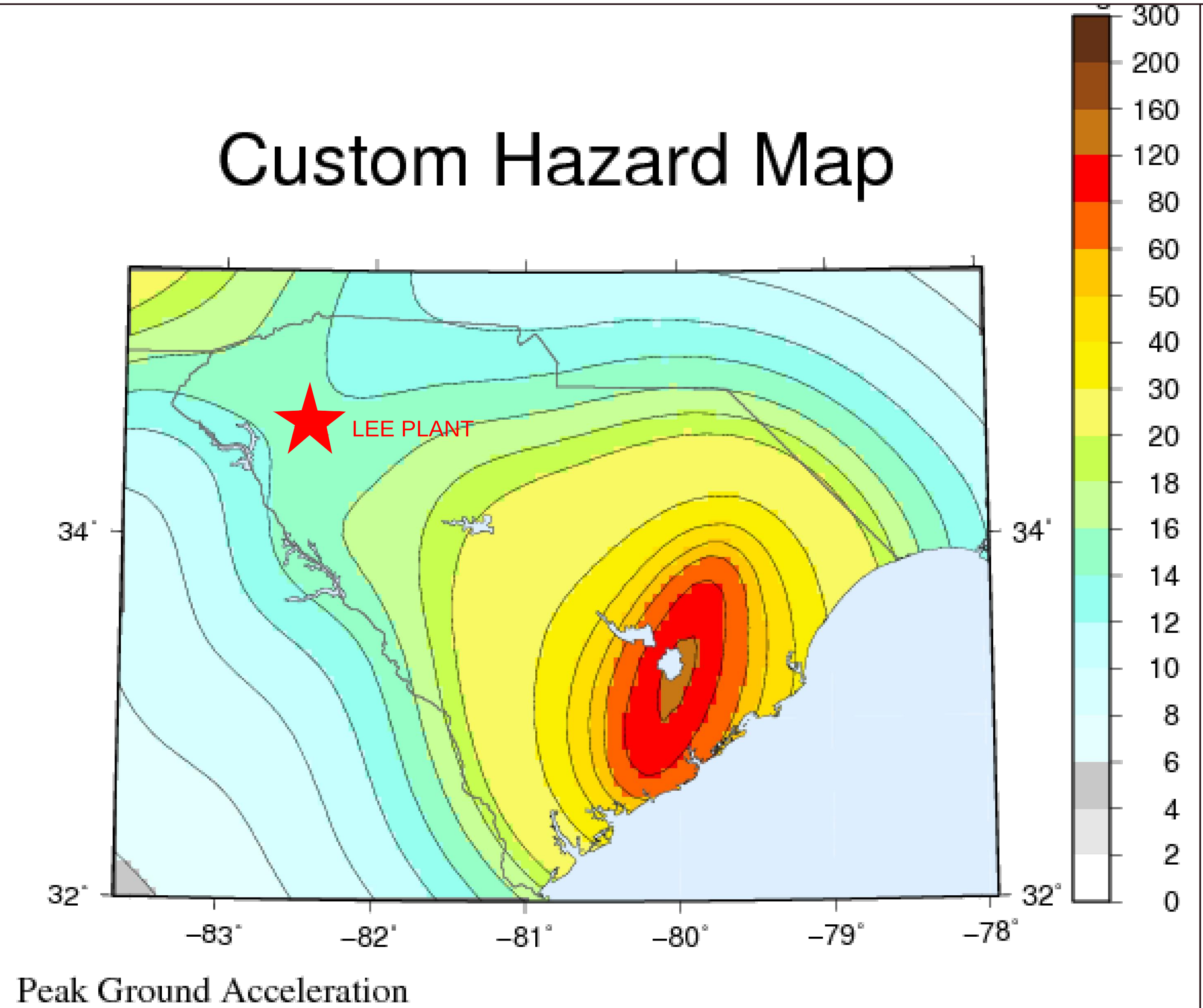
OFFICIAL COPY

Jan 23 2018

SCALE: AS SHOWN	DATE: 08-28-14	S&ME	301 ZIMA PARK DRIVE 29301 PH. (864) 574-2360 FAX. (864) 576-8730 WWW.SMEINC.COM	LEE STEAM STATION – 1951 RETIRED ASH BASIN STABILITY ANALYSIS SECTION D-D BELTON, S.C.	FIGURE NO. 5
PROJECT NO. 7126-14-005	DRAWN BY: F. MORRIS				
	CHECKED BY: M. REVIS				



SCALE: AS SHOWN	DATE: 08-28-14	S&ME	301 ZIMA PARK DRIVE 29301 PH. (864) 574-2360 FAX. (864) 576-8730 WWW.SMEINC.COM	LEE STEAM STATION – 1951 RETIRED ASH BASIN STABILITY ANALYSIS SECTION E-E BELTON, S.C.	FIGURE NO. 6
PROJECT NO. 7126-14-005	DRAWN BY: F. MORRIS				
	CHECKED BY: M. REVIS				



SOURCE:
USGS CUSTOM HAZARD MAP GENERATOR
(http://geohazards.usgs.gov/hazards/apps/cmaps/).

SCALE:	NOT TO SCALE
CHECKED BY:	JR
DRAWN BY:	MR
DATE:	8-28-14

S&ME
GREENVILLE, SC 864.574.2360
SPARTANBURG, SC 864.232.8987

USGS SEISMIC HAZARD MAP
1951 RETIRED ASH BASIN
DUKE ENERGY LEE STEAM STATION
BELTON, SOUTH CAROLINA

JOB NO: 7126-14-005

FIGURE NO: 7

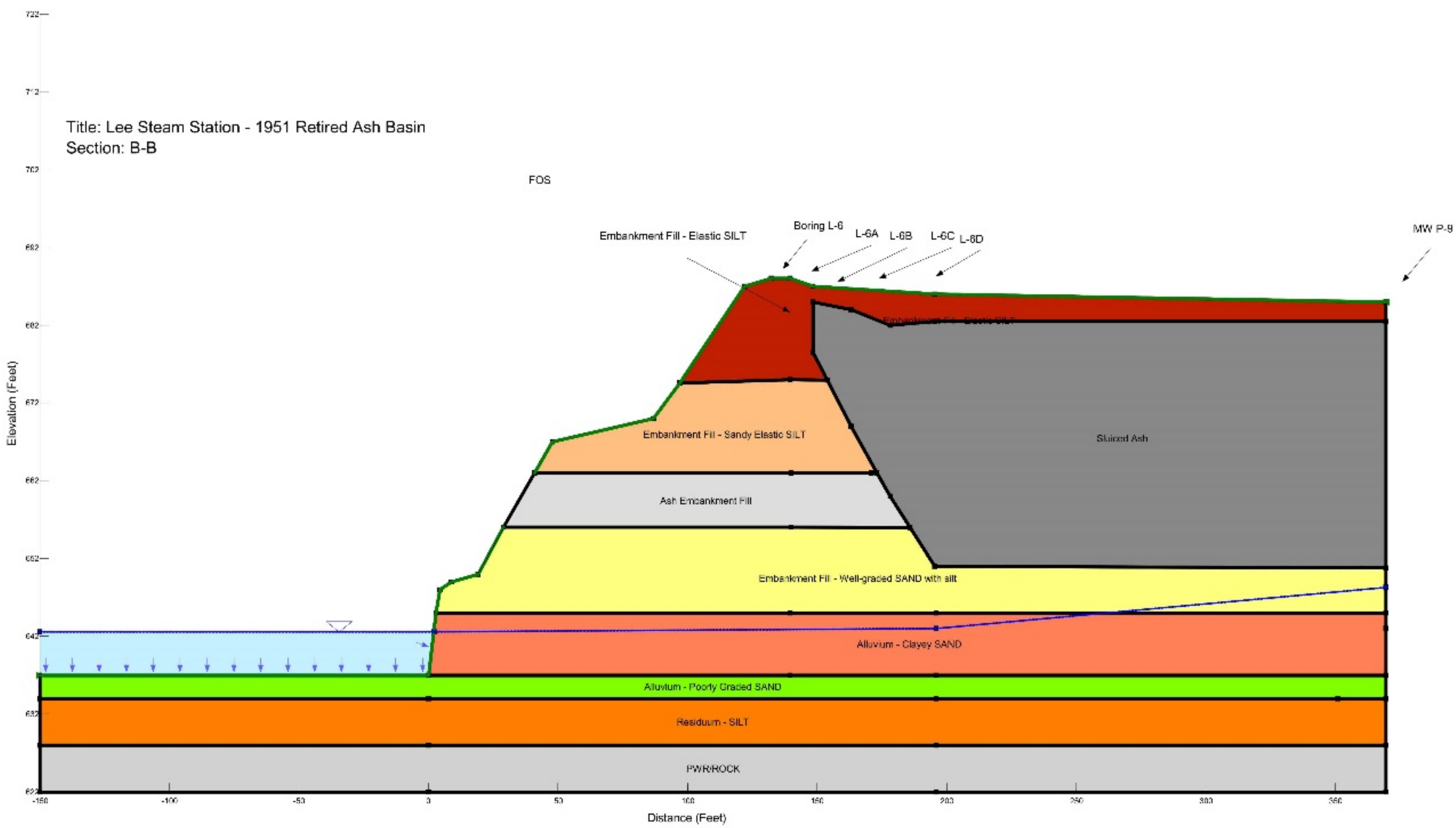


Figure Illustrating Existing Conditions

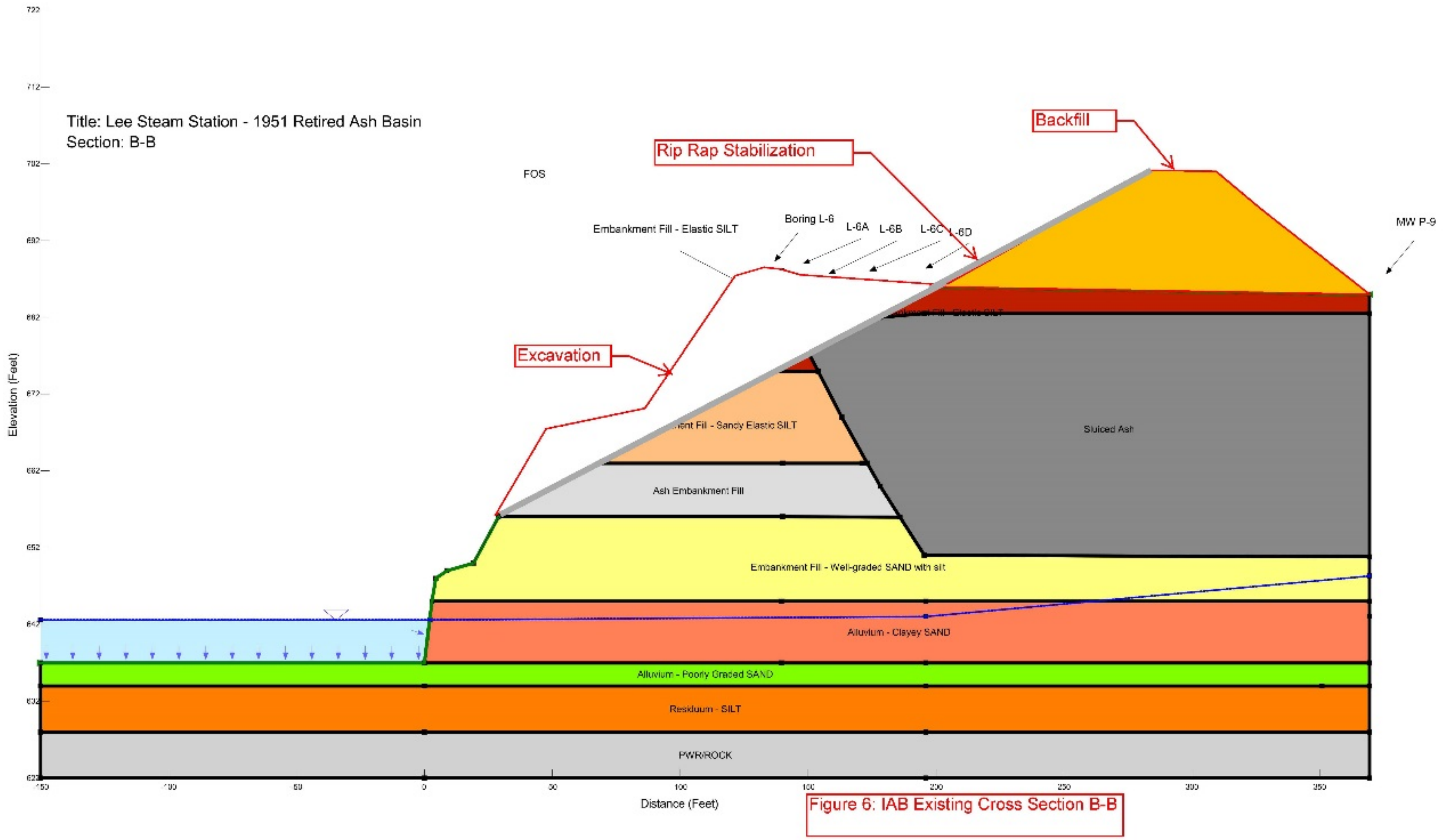


Figure Illustrating Grading and Slope Stabilization

Aerial View of Structural Fill Area

WS LEE - TABULATION OF MONTHLY DISALLOWANCE		
Period Begin	Period End	DISALLOWANCE
01/01/15	01/31/15	$ -
02/01/15	02/28/15	$ -
03/01/15	03/31/15	$ -
04/01/15	04/30/15	$ -
05/01/15	05/31/15	$ 142,612
06/01/15	06/30/15	$ 443,509
07/01/15	07/31/15	$ 798,468
08/01/15	08/31/15	$ 694,616
09/01/15	09/30/15	$ 763,115
10/01/15	10/31/15	$ 589,228
11/01/15	11/30/15	$ 331,167
12/01/15	12/31/15	$ 363,414
01/01/16	01/31/16	$ 548,862
02/01/16	02/29/16	$ 494,147
03/01/16	03/31/16	$ 782,283
04/01/16	04/30/16	$ 721,161
05/01/16	05/31/16	$ 625,975
06/01/16	06/30/16	$ 889,101
07/01/16	07/31/16	$ 758,050
08/01/16	08/31/16	$ 902,215
09/01/16	09/30/16	$ 964,184
10/01/16	10/31/16	$ 962,808
11/01/16	11/30/16	$ 770,864
12/01/16	12/31/16	$ 755,143
01/01/17	01/31/17	$ 1,061,713
02/01/17	02/28/17	$ 1,151,462
03/01/17	03/31/17	$ 1,349,614
04/01/17	04/30/17	$ 1,197,851
05/01/17	05/31/17	$ 1,455,893
06/01/17	06/30/17	$ 1,508,041
07/01/17	07/31/17	$ 1,240,777
08/01/17	08/31/17	$ 1,585,592
09/01/17	09/30/17	$ 1,271,070
10/01/17	10/31/17	$ 1,356,939
11/01/17	11/30/17	$ 795,319
	TOTAL	**$ 27,275,192**

CONFIDENTIAL

FACILITY: WS LEE

CCR ID 1951 - 1959 Inactive Ash Basin

			CCR Hauling (includes Excavation, Loading, Transportation)						
Period Begin	Period End	CCR In-Place Period Beginning TON	CCR Hauled off-site via TRUCK TON	Unit Rate CCR Hauled off-site via TRUCK $/TON	EXCAVATION ($/TON)	LOADING, TRANSPORTATION ($/TON)	ADJUSTED Unit Rate CCR Hauled ON-SITE via TRUCK $/TON (1)	UNIT RATE OF DISALLOWANCE ($/TON)	DISALLOWANCE AMOUNT ($)
01/01/15	01/31/15	[redacted]							
02/01/15	02/28/15	[redacted]							
03/01/15	03/31/15	[redacted]							
04/01/15	04/30/15	[redacted]							
05/01/15	05/31/15	[redacted]							$130,653.60
06/01/15	06/30/15	[redacted]							$406,319.76
07/01/15	07/31/15	[redacted]							$731,514.63
08/01/15	08/31/15	[redacted]							$636,370.35
09/01/15	09/30/15	[redacted]							$699,126.12
10/01/15	10/31/15	[redacted]							$539,819.28
11/01/15	11/30/15	[redacted]							$303,397.71
12/01/15	12/31/15	[redacted]							$332,940.30
01/01/16	01/31/16	[redacted]							$502,838.49
02/01/16	02/29/16	[redacted]							$452,711.49
03/01/16	03/31/16	[redacted]							$716,686.74
04/01/16	04/30/16	[redacted]							$660,690.03
05/01/16	05/31/16	[redacted]							$573,485.22
06/01/16	06/30/16	[redacted]							$814,547.58
07/01/16	07/31/16	[redacted]							$694,485.33
08/01/16	08/31/16	[redacted]							$826,561.89
09/01/16	09/30/16	[redacted]							$883,334.76
10/01/16	10/31/16	[redacted]							$882,073.50
11/01/16	11/30/16	[redacted]							$706,224.75
12/01/16	12/31/16	[redacted]							$690,912.08
01/01/17	01/31/17	[redacted]							$934,742.80
02/01/17	02/28/17	[redacted]							$554,111.52
03/01/17	03/31/17	[redacted]							$651,510.08
04/01/17	04/30/17	[redacted]							$630,368.32
05/01/17	05/31/17	[redacted]							$725,124.16
06/01/17	06/30/17	[redacted]							$751,949.36
07/01/17	07/31/17	[redacted]							$604,355.04
08/01/17	08/31/17	[redacted]							$763,953.04
09/01/17	09/30/17	[redacted]							$583,802.32
10/01/17	10/31/17	[redacted]							$552,232.96
11/01/17	11/30/17	[redacted]							$0.00
									$18,936,843.21

SHADING INDICATES DATA PROVIDE BY DEC

SHADING INDICATES AN ADJUSTED UNIT RATE

(1) EXCAVATION UNIT RATE PLUS ADJUSTED LOADING AND TRANSPORTATION RATE

CONFIDENTIAL

FACILITY: WS LEE

CCR ID 1951 - 1959 Inactive Ash Basin

		CCR Disposal (includes Unloading, Development, Placement, Overhead, Profit & Fee)								
Period Begin	Period End	CCR Disposed Off-site at Facility 1 TON	Unit Rate CCR Disposed off-site at Facility 1 $/TON	Unloading, development, placement ($/TON)	Overhead ($/TON) (2)	Profit ($/TON) (2)	ADJUSTED Unit Rate CCR Disposed off-site at Facility 1 ($/TON)	UNIT RATE OF DISALLOWANCE ($/TON)	DISALLOWANCE AMOUNT ($)
01/01/15	01/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	
02/01/15	02/28/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	
03/01/15	03/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	
04/01/15	04/30/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	
05/01/15	05/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 11,958.40
06/01/15	06/30/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 37,189.44
07/01/15	07/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 66,953.72
08/01/15	08/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 58,245.40
09/01/15	09/30/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 63,989.28
10/01/15	10/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 49,408.32
11/01/15	11/30/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 27,769.24
12/01/15	12/31/15	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 30,473.20
01/01/16	01/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 46,023.56
02/01/16	02/29/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 41,435.56
03/01/16	03/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 65,596.56
04/01/16	04/30/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 60,471.32
05/01/16	05/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 52,489.68
06/01/16	06/30/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 74,553.52
07/01/16	07/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 63,564.52
08/01/16	08/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 75,653.16
09/01/16	09/30/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 80,849.44
10/01/16	10/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 80,734.00
11/01/16	11/30/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 64,639.00
12/01/16	12/31/16	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 64,230.52
01/01/17	01/31/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 86,898.20
02/01/17	02/28/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 51,512.88
03/01/17	03/31/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 60,567.52
04/01/17	04/30/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 58,602.08
05/01/17	05/31/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 67,411.04
06/01/17	06/30/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 69,904.84
07/01/17	07/31/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 56,183.76
08/01/17	08/31/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 71,020.76
09/01/17	09/30/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 54,273.08
10/01/17	10/31/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ 51,338.24
11/01/17	11/30/17	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	$ -
									$1,743,940.24

SHADING INDICATES DATA PROVIDE BY DEC

SHADING INDICATES AN ADJUSTED UNIT RATE

(2) OVERHEAD AND PROFIT REDUCED BY AMOUNT OF DISSALOWANCE

CONFIDENTIAL

FACILITY: WS LEE

CCR ID 1951 - 1959 Inactive Ash Basin

			CCR Hauling (includes Excavation, Loading, Transportation)						
Period Begin	Period End	CCR In-Place Period Beginning TON	CCR Hauled off-site via TRUCK TON	Unit Rate CCR Hauled off-site via TRUCK $/TON	EXCAVATION ($/TON)	LOADING, TRANSPORTATION ($/TON)	ADJUSTED Unit Rate CCR Hauled ON-SITE via TRUCK $/TON (1)	UNIT RATE OF DISALLOWANCE ($/TON)	DISALLOWANCE AMOUNT ($)
01/01/15	01/31/15	[redacted]							
02/01/15	02/28/15	[redacted]							
03/01/15	03/31/15	[redacted]							
04/01/15	04/30/15	[redacted]							
05/01/15	05/31/15	[redacted]							$130,653.60
06/01/15	06/30/15	[redacted]							$406,319.76
07/01/15	07/31/15	[redacted]							$731,514.63
08/01/15	08/31/15	[redacted]							$636,370.35
09/01/15	09/30/15	[redacted]							$699,126.12
10/01/15	10/31/15	[redacted]							$539,819.28
11/01/15	11/30/15	[redacted]							$303,397.71
12/01/15	12/31/15	[redacted]							$332,940.30
01/01/16	01/31/16	[redacted]							$502,838.49
02/01/16	02/29/16	[redacted]							$452,711.49
03/01/16	03/31/16	[redacted]							$716,686.74
04/01/16	04/30/16	[redacted]							$660,690.03
05/01/16	05/31/16	[redacted]							$573,485.22
06/01/16	06/30/16	[redacted]							$814,547.58
07/01/16	07/31/16	[redacted]							$694,485.33
08/01/16	08/31/16	[redacted]							$826,561.89
09/01/16	09/30/16	[redacted]							$883,334.76
10/01/16	10/31/16	[redacted]							$882,073.50
11/01/16	11/30/16	[redacted]							$706,224.75
12/01/16	12/31/16	[redacted]							$690,912.08
01/01/17	01/31/17	[redacted]							$934,742.80
02/01/17	02/28/17	[redacted]							$554,111.52
03/01/17	03/31/17	[redacted]							$651,510.08
04/01/17	04/30/17	[redacted]							$630,368.32
05/01/17	05/31/17	[redacted]							$725,124.16
06/01/17	06/30/17	[redacted]							$751,949.36
07/01/17	07/31/17	[redacted]							$604,355.04
08/01/17	08/31/17	[redacted]							$763,953.04
09/01/17	09/30/17	[redacted]							$583,802.32
10/01/17	10/31/17	[redacted]							$552,232.96
11/01/17	11/30/17	[redacted]							$0.00
									$18,936,843.21

SHADING INDICATES DATA PROVIDE BY DEC

SHADING INDICATES AN ADJUSTED UNIT RATE

(1) EXCAVATION UNIT RATE PLUS ADJUSTED LOADING AND TRANSPORTATION RATE

CONFIDENTIAL

FACILITY: WS LEE

CCR ID 1951 - 1959 Inactive Ash Basin

		CCR Disposal (includes Unloading, Development, Placement, Overhead, Profit & Fee)							
Period Begin	Period End	CCR Disposed Off-site at Facility 1 TON	Unit Rate CCR Disposed off-site at Facility 1 $/TON	Unloading, development, placement ($/TON)	Overhead ($/TON) (2)	Profit ($/TON) (2)	ADJUSTED Unit Rate CCR Disposed off-site at Facility 1 ($/TON)	UNIT RATE OF DISALLOWANCE ($/TON)	DISALLOWANCE AMOUNT ($)
01/01/15	01/31/15	[redacted]							
02/01/15	02/28/15								
03/01/15	03/31/15								
04/01/15	04/30/15								
05/01/15	05/31/15								$ 11,958.40
06/01/15	06/30/15								$ 37,189.44
07/01/15	07/31/15								$ 66,953.72
08/01/15	08/31/15								$ 58,245.40
09/01/15	09/30/15								$ 63,989.28
10/01/15	10/31/15								$ 49,408.32
11/01/15	11/30/15								$ 27,769.24
12/01/15	12/31/15								$ 30,473.20
01/01/16	01/31/16								$ 46,023.56
02/01/16	02/29/16								$ 41,435.56
03/01/16	03/31/16								$ 65,596.56
04/01/16	04/30/16								$ 60,471.32
05/01/16	05/31/16								$ 52,489.68
06/01/16	06/30/16								$ 74,553.52
07/01/16	07/31/16								$ 63,564.52
08/01/16	08/31/16								$ 75,653.16
09/01/16	09/30/16								$ 80,849.44
10/01/16	10/31/16								$ 80,734.00
11/01/16	11/30/16								$ 64,639.00
12/01/16	12/31/16								$ 64,230.52
01/01/17	01/31/17								$ 86,898.20
02/01/17	02/28/17								$ 51,512.88
03/01/17	03/31/17								$ 60,567.52
04/01/17	04/30/17								$ 58,602.08
05/01/17	05/31/17								$ 67,411.04
06/01/17	06/30/17								$ 69,904.84
07/01/17	07/31/17								$ 56,183.76
08/01/17	08/31/17								$ 71,020.76
09/01/17	09/30/17								$ 54,273.08
10/01/17	10/31/17								$ 51,338.24
11/01/17	11/30/17								$ -
									$1,743,940.24

SHADING INDICATES DATA PROVIDE BY DEC

SHADING INDICATES AN ADJUSTED UNIT RATE

(2) OVERHEAD AND PROFIT REDUCED BY AMOUNT OF DISSALOWANCE