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AMEREN MISSOURI LABADIE ENERGY CENTER
UTILITY WASTE LANDFILL (UWL)
SOLID WASTE DISPOSAL AREA
FRANKLIN COUNTY, MISSOURI

APPENDIX J
GEOTECHNICAL INVESTIGATION FOR CONSTRUCTION PERMIT APPLICATION

Prepared for

Ameren MISSOURI

Prepared by

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GEOTECHNICAL ENGINEERING REPORT FOR CONSTRUCTION PERMIT APPLICATION
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1.0 SCOPE OF GEOTECHNICAL INVESTIGATION

Reitz & Jens, Inc. (R&J) completed a geotechnical investigation for the design of the proposed Utility Waste Landfill (UWL) for the Ameren Missouri Labadie Energy Center in Franklin County, Missouri. The UWL will be used for disposal of coal combustion products (CCP) from the Labadie Energy Center in a utility waste landfill as defined in 10 CSR 80-2.020(119). R&J leads the design team for the UWL that includes GREDELL Engineering Resources, Inc. (GER). This investigation provides supporting geotechnical information, testing results, results of analyses, and documentation to be incorporated with the Construction Permit Application for this UWL. The scope of this geotechnical investigation included the following main tasks which are described in detail in this report.

1.1 Field Investigation

The field investigation was completed in conjunction with the Detailed Site Investigation, using drilled exploratory borings and cone penetrometer testing (CPT), and laboratory testing to characterize the geotechnical engineering properties of the subsurface soils strata at the site. It is not feasible to obtain suitable clay on site for the compacted clay liner. Therefore, most, if not all, of the clay liner material will be obtained from an off-site borrow source(s) that will be identified prior to construction. A preliminary field investigation with laboratory testing was completed to characterize the subsurface soils at a borrow site on Ameren Missouri's Callaway Energy Center property proposed for use as clay materials for the liner and cap construction at the Labadie UWL. Appropriate engineering properties were assumed for the compacted clay liner in our analyses. These properties will be confirmed after the borrow source(s) are identified and prior to construction.

1.2 Laboratory Testing of CCP

Laboratory tests were run on samples of CCP materials from the Labadie Energy Center to determine parameters for use in the design of the UWL. The materials tested included fly ash from the existing pond at the Labadie Energy Center, dry fly ash collected from the precipitators ("non-ponded fly ash"), and bottom ash. The CCP placed in the UWL may be a mixture of fly ash, bottom ash and flue gas desulphurization gypsum in the future. Since gypsum is not available from the Labadie Plant, a sample of dry gypsum from Ameren's Duck Creek Plant was used in our testing.

1.3 Seismic Risk Assessment and Analyses

The peak horizontal ground acceleration (PHGA) for the Labadie site was analyzed by two methods: 1) the PHGA obtained from the latest available USGS hazard map; and 2) a site specific seismic analysis using the seismic model program SHAKE2000. The PHGA from the USGS map was higher.

than that from our site-specific seismic analysis. Therefore, the PHGA from the USGS map was used in analyses of embankment stability under seismic load, potential for liquefaction, the potential effect of liquefaction on embankment stability, and potential for settlement induced by liquefaction. The derived time-histories of seismic accelerations for the St. Louis area that are built into SHAKE2000 were used for deformation analyses, to satisfy the requirements of Missouri sold-waste regulations.

We analyzed the potential of liquefaction of the subsurface strata at each boring and CPT sounding using the latest published method (Idress and Boulanger, 2008). We mapped areas of potential liquefaction under the existing site. Our analyses demonstrate that the potential for liquefaction beneath the UWL becomes negligible after the CCP fill is 20 feet thick. We also determined the residual shear strengths of potentially liquefiable natural strata using several published methods, for slope stability analyses and the horizontal ground accelerations that result in the onset of liquefaction.

Where only a Peak Horizontal Ground Acceleration (PHGA) was needed for our analyses, such as slope stability or liquefaction, we used the more conservative PHGA from the published USGS hazard maps. We used several derived time-histories of ground accelerations from our SHAKE2000 analyses for the deformation analyses to obtain more accurate estimates of the probable horizontal deformation using site specific data. This resulted in the UWL being designed to accommodate the design seismic event for this site in accordance with MDNR requirements and generally accepted engineering practices.

1.4 Slope Stability Analyses

We analyzed the stability of the side slopes of the perimeter berms and the CCP fill at five sections which had slightly varying subsurface soil profiles. We analyzed each section for the intermediate height of CCP fill using both short-term and long-term soil properties, and with the potential liquefaction; and for the full height of CCP fill using long-term soil properties, and with potential liquefaction. We also analyzed potential sliding block failures along the interface with the composite liner, and the stability of the final cover. All of these analyses demonstrate that the proposed design meets or exceeds the slope stability requirements.

The Missouri solid-waste regulations do not state a minimum factor of safety for the stability of the slopes under seismic load. Rather, the regulations state that the expected deformation cannot exceed a maximum of 6 inches (for a sanitary landfill). Our analyses demonstrate that the maximum anticipated lateral deformation due to the design PHGA would be negligible.

The stability analyses included the calculation of bearing capacity of the foundation soils in accordance with 80-11.010(5)(A)(4.A).

1.5 Settlement Analyses

The settlement of the subsurface soils under the final CCP landfill was estimated for the subsurface strata at groups of borings and CPT soundings. The results were graphed to produce the estimated settlement of the subgrade along four cross-sections of the completed landfill and along the existing Explorer pipeline. These estimates of settlement were used for the design of the leachate collection system. The results also demonstrate that the composite liner will not be subjected to damaging strains due to settlement.

1.6 Impacts Due to Flooding

Because the site is located in a floodplain, the Missouri solid-waste regulations require that the design of the UWL prevent damage to the composite liner that could result from hydrostatic uplift due to flooding. This requirement is satisfied by the initial operation of the UWL, during which sufficient CCP fill will be placed in each cell to resist the hydrostatic uplift. Also, we provide a design for a fabric-formed concrete mat (FCM) for the exterior berms to prevent erosion of the slopes due to the velocity of flows that may occur if the existing agricultural levee along the Missouri River were to be overtopped or fail during a flood event. The design of the exterior berms prevents flood water from contacting the CCP in the cells up to the 500-year flood in accordance with Franklin County ordinances.

1.7 Recommendations

Other recommendations are presented in this report for bearing capacity of subsurface soils, construction quality assurance procedures, impact of ground water in contact with the bottom composite liner, and the investigation and remediation of potential liquefaction damage during the initial operation of the UWL, in fulfillment of the requirements of the Missouri UWL regulations.

Our professional engineering judgment is that the Labadie UWL design and operating procedures described in this geotechnical report for the CPA are in accordance with generally accepted engineering practice and utilize conservative assumptions where necessary, and therefore meet or exceed all of the requirements of the Missouri Solid Waste Management Law and Regulations, as well as those of applicable Franklin County ordinances.

2.0 EXPLORATORY FIELD INVESTIGATION

2.1 Detailed Site Investigation (DSI)

The field and laboratory work for this investigation was completed as a component of the Detailed Site Investigation (DSI) for the proposed Labadie UWL. The DSI workplan utilized the 100 "temporary" or non-piezometer borings required for the DSI to provide data of subsurface conditions for the subsequent geotechnical analyses and design of the UWL. This work was completed in accordance with the workplan entitled, *Ameren Missouri Labadie Power Plant Utility Waste Landfill Detailed Site Investigation Work Plan*. The workplan was originally submitted to the Missouri Department of Natural Resources-Division of Geology and Land Survey (MDNR-DGLS) on May 14, 2009, and was approved on June 15, 2009. The results of the field and laboratory work are presented in Appendix 2, "Geotechnical Investigation Report," of the report *Detailed Site Investigation Report for Ameren Missouri Labadie Power Plant Proposed Utility Waste Disposal Area, Franklin County, Missouri*, dated February 4, 2011. The DSI report was subsequently revised on March 30, 2011, in response to questions from the Geological Survey Program (GSP). The GSP approved the DSI and report in a letter dated April 8, 2011.

The field investigation consisted of 119 borings and 93 Cone Penetrometer Test (CPT) soundings for a total of 212 test locations. Of the 119 borings, 22 were temporary geotechnical borings (labeled "B-"), and 97 were piezometer borings (labeled "P-"). The CPT soundings (labeled "C-") were alternated with the piezometer borings on a regular grid-like pattern. The plan of the borings and CPT soundings is shown in **Figure 1**. Some locations were moved from a linear pattern due to geographic restrictions or to better characterize the subsurface conditions. Confirmation borings were made for some of the CPT soundings. Confirmation CPT soundings were also made at randomly selected locations. The 119 borings were in addition to the preliminary geotechnical investigation by Reitz & Jens in 2007, which included the installation of three piezometers and five temporary geotechnical borings. The report of this investigation was included in the Preliminary Site Investigation (PSI) request submitted to MDNR-DGLS in December 2008, and in the approved DSI workplan.

The CPT soundings were made using a 1.5-inch diameter, 100-MPa capacity, electronic piezocone, which records tip pressure, sleeve friction and porewater pressure every 20 millimeters as the cone is hydraulically pushed into the ground at a specified rate. The testing was carried out according to ASTM D5778 "Electronic Friction Cone and Piezocone Penetration Testing of Soils". The final CPT sounding logs are presented in the DSI report. The analysis of the raw data from the CPT soundings is presented in Appendix D of the DSI report, which includes the side by side comparisons between the CPT soundings and other borings to validate the classification of subsurface soil strata developed from the CPT soundings, and comparisons between CPT soundings performed side-by-side in the field to demonstrate the reproducibility of the CPT results.

Details of the field work completed for the DSI and all of the results are presented in the DSI report referenced above.

2.2 Preliminary Investigation of Off-Site Borrow Material at Callaway Plant

Twelve borings made at the potential clay borrow site located at Ameren Missouri's Callaway Power Plant. The borrow site is located in Callaway County approximately one mile east of the Callaway Power Plant on County Road 448 (see **Figure 1** in **Appendix A**). The borrow site was subdivided into areas based upon the present land use and topography. The purpose of these borings was to provide data on the subsurface conditions and to quantify the clay borrow that could be used for construction of clay liner and cover at Labadie UWL. Details of the field investigation and laboratory testing are presented in **Appendix A**. The borings were drilled to termination depths ranging from 14 feet to 31 feet, with some borings terminating on intact bedrock.

Reitz & Jens' report of the preliminary investigation for Ameren Missouri, dated May 25, 2011, is reproduced in **Appendix A**. Subsequent to submittal of our report to Ameren Missouri, Reitz & Jens' performed additional laboratory testing of the high plastic clay to obtain properties for the stability analyses of the liner and perimeter berm. The additional tests included consolidated-undrained (CU) triaxial compression tests with pore pressure measurements on composite samples of the clay compacted to 89% of the maximum dry unit weight obtained from a standard Proctor moisture-density test (ASTM D698-00a), and a direct shear test of the compacted clay with a double-textured 60-mil HDPE membrane. The results of these tests are included in **Appendix A-1**. The results of the CU triaxial tests were: a total cohesion (c) of 420 psf and total internal friction angle (ϕ) of 9.6º; and an effective cohesion (c') of 440 psf and an effective internal friction angle (ϕ') of 14.6º.

We ran direct shear tests of a molded sample of the clay, at a dry unit weight of 99 pcf, with a sample of textured HDPE liner on the bottom plate. This was run in a standard direct shear apparatus with a sample diameter of 2 inches. The peak shear strength properties were: c of 320 psf and ϕ of 29º. The residual shear strength properties were: c of 290 psf and ϕ of 26.9º (see results in **Appendix A-1**).

3.0 LABORATORY TESTING

3.1 Tests on Natural Soil Deposits

All laboratory testing was completed in accordance with the latest applicable ASTM procedures as contained in Reitz & Jens' Quality Manual. Reitz & Jens' soils laboratory maintains an AASHTO Materials Research Laboratory (AMRL) certification from National Institute for Standards and Technology (NIST). Details of the laboratory testing program on soil samples from the site of the proposed UWL and all of the results are presented in the DSI report.

The general purpose of the testing program was to obtain soil properties for the determination of: bearing capacity, short-term and long-term slope stability, seepage characteristics of the top stratum fine-grain soils and the underlying sand strata, liquefaction potential, settlement characteristics, and soil classifications for the potential use of soils for fill materials.

Grain-size analyses (ASTM D422) were performed on selected cohesionless samples (Unified Soil Classifications of SW, SP, SM, GW, GP, or GP-SP). Hydrometer analyses (ASTM D422) were run on 3 selected samples which had a high percentage of fine-grain soils (passing U.S. #200 sieve).

The shear strength properties of a soil mass are dependent about the mineralogy, size and shape of the particles; the density of the soil particles; and the pressure of the water in the pores of the soil mass. When the dry density of a soil is increased, the shear strength generally increases – more for a granular soil (gravels, sands and silts) and less for clays. If a laboratory test is performed on the soil sample at the dry density under existing field conditions, then sample is "unconsolidated." If the first step of a laboratory test is to apply a known pressure to densify the soil sample while draining off the increase in pore pressure under the applied load, then the soil is "consolidated" which more accurately estimates the properties in the field after a period of time under the added weight of the landfill. Pore water pressures in a soil mass also generally increase as the soil is sheared if the soil densifies or consolidates during shearing. If the shear stress is applied quickly, or the pore pressures are not allowed to dissipate, then the measured shear strength properties are "undrained." If the shear stress is applied slowly such that the pore pressures can dissipate during shearing, then the shear strength properties are "drained." This type of test represents the shear strength properties of the soil mass over a long time. Pore pressures dissipate very rapidly in large-grain soils (gravels and sands), so the measured shear strength is always considered to be "drained." If the pore water pressures are measured during shearing, then the pore pressure can be subtracting from the measured stress on the soil mass (called "total" stress) resulting in the "effective" stress. The "effective" shear strength properties are essentially the same as the drained or long-term properties, and are the actual shear strength properties of the soil mass.

Unconsolidated-undrained (UU) triaxial shear strength tests, ASTM D2850 "Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils", were performed on selected Shelby tube samples from each major cohesive soil stratum. The UU tests were performed at the estimated confining pressure of the sample in the field conditions, to measure the *in situ* undrained shear strength of the soil. Nine UU tests were performed.

Series of consolidated-undrained (CU) triaxial compression tests, ASTM D4767 "Consolidated Undrained Triaxial Compression Test for Cohesive Soils," were performed on each major cohesive soil stratum from different locations around the proposed disposal area. The tests were performed with the measurement of internal pore water pressures so that the effective strength properties of the soil could be determined. Each series has a minimum of two points, and three points where possible. Five series of CU tests were performed.

Three one-dimensional consolidation tests, ASTM D2435 "One-Dimensional Consolidation Properties of Soil Using Incremental Loading," were performed on selected relatively undisturbed Shelby tube samples from each major cohesive soil stratum beneath the UWL.

Two flexible-wall hydraulic conductivity tests, ASTM D5084 "Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter," were performed on selected relatively undisturbed Shelby tube samples of the upper clays. Also, two flexible-wall hydraulic conductivity tests were performed on samples from the preliminary Boring B-4: one on high plastic clay and one on sandy silt, both of which were obtained from 3.5 to 5.5 feet deep. The data from the hydraulic conductivity tests are included in Appendix B of the DSI Report and are summarized in the following table:

Boring No.	Sample	Depth, feet	Soil Description	k, cm/sec
B-4	ST-2	3.5 – 5.5	High Plastic Clay (CH)	1.2×10^{-8}
B-4	ST-2	3.5 – 5.5	Sandy Silt (SM)	2.0×10^{-3}
B-52	ST-2	4 – 6	High Plastic Clay (CH)	5.6×10^{-9}
P-175	ST-0	1 – 3	High Plastic Clay (CH)	5.5×10^{-8}

The appropriate physical properties that were measured by the field and laboratory testing of the natural soil strata found on this site were used in all of our geotechnical analyses.

3.2 Tests on CCP from Labadie Energy Center

We tested different samples of CCP from the Labadie Energy Center to determine engineering properties to use in the design of the UWL. Samples included: CCP (fly ash) which was collected from the precipitators prior to wetting; CCP from the fly ash pond which had been mixed with water to form a slurry and then was deposited in the pond by sedimentation; and bottom ash. These were tested because the method of transporting the CCP to the UWL may change over time. Initially, the CCP from the existing pond will be excavated, partially dried, and then hauled to the UWL by truck. In the future, Ameren may choose to convey dry CCPs directly to the UWL for moisture conditioning and disposal. The results of the lab testing are reproduced in **Appendix B**.

3.2.1 Tests on Non-Ponded Fly Ash

Bucket samples of dry fly ash were collected at the Labadie Energy Center on December 13, 2008, and again on February 2, 2009. The particle-size distribution was determined using ASTM D422 "Standard Test Method for Particle-Size Analysis of Soils." The fly ash tended to form clumps (i.e. flocculate) in the hydrometer test, so a second sample was mixed with sodium hexametaphosphate (SHMP) which is a dispersing agent to prevent the flocculation of particles. We ran the fly ash with SHMP and without SHMP, to determine the effect on the particle-size distribution. The reports of particle-size distribution results are presented in **Appendix B.** The reports give the uniformity coefficient (c_u) which is defined at D_{60}/D_{10}, where D_{60} is the particle-size or diameter for which 60% of the dry material by weight is smaller, and D_{10} is the diameter for which 10% of the material by weight is smaller. A c_u of 1 is perfectly uniform, and a c_u greater than 6 may be well-graded by the Unified Soil Classification System (USCS). The reports also give the percentage by weight finer than a U.S. #30 sieve (0.60mm). The plots of particle-size distribution clearly show the flocculation of the sample: without SHMP, the distribution is almost uniform (c_u = 1.69); with SHMP, the distribution is well-graded (c_u = 10.86). The fly ash sample had about 93% finer than a #200 sieve (0.075 mm).

The bulk specific gravity (SG) was determined using ASTM D854 “Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer.” The SG of the non-ponded fly ash sample was 2.87.

The fly ash sample was very pozzolanic and hardened in a few minutes after it was mixed with water. To run a standard Proctor Moisture-Density Test (ASTM D698), we mixed five samples of the fly ash at selected moisture contents (MC) between 8.5% and 20%, and then grated the moistened fly ash before it had completely hardened. Thus, the methodology does not mimic field procedures, and would be expected to create a sample with lower measured strength than will occur in the field. The results are presented in **Appendix B**. The maximum dry unit weight ($\gamma_{d,max}$) was 107.9 lbs/ft^3 (pcf) and the optimum moisture content (w_{opt}) was 17.3%. The cylinders molded from the Proctor test were trimmed immediately to a diameter of 2 inches and appropriate height, and then were broken the next day in unconfined compression tests. The results are presented in **Appendix B**. The moist unit weights and 24-hour unconfined compressive strengths (Q_u) were:

Molded Moisture Content %	Moist Unit Weight lbs/ft^3	Unconfined Compressive Strength, Q_u Psi
11.3	116	158
15.5	127	156
16.3	127	108
20.0	127	71

This testing shows that the non-ponded fly ash when wetted and semi-compacted will achieve much greater cohesive shear strength than soil. However, a lower strength was used for the non-ponded fly ash in the analyses for the UWL to eliminate the need for construction quality control during routine placement of non-ponded fly ash in the UWL. Also, this means that the stability and seismic analyses completed for the UWL and reported herein are very conservative with regard to the placement of non-ponded fly ash in the UWL.

We ran a flexible-wall hydraulic conductivity test on the non-ponded fly ash. A sample was molded at 22.5% moisture, that is about 5% wetter than optimum. The initial dry unit weight of the sample was

101.4 pcf, or a moist unit weight of 124.2 pcf. The results of the hydraulic conductivity test are presented in **Figure B-5** in **Appendix B**. The hydraulic conductivity (k) was 8.3×10^{-6} cm/sec.

A one-dimensional consolidation test was run on a molded sample of the non-ponded fly ash to determine its compressibility. The moist unit weight of the sample was 117.5 pcf. The fly ash was mixed at a moisture content of 22.5%; however, due to the pozzolanic action the measured moisture content after molding was 8.2%. The coefficient of consolidation (C_c) was 0.02. The calculated previous consolidation pressure (P_c) was 1.0 tons/ft^2. The results are presented in **Appendix B, Figure B-6.**

To estimate the probable moist unit weight of the non-ponded fly ash if it were wetted and lightly compacted in the UWL, we determined the dry unit weight of a sample of non-ponded fly ash that was densified in a mold using a vibratory table similar to the maximum density test (ASTM D4254) but without using a confining weight. The dry unit weight was 92 pcf. We then determined the maximum moisture content that would pass the paint filter test (Environmental Protection Agency Method 9095B, Rev. 2, November 2004), which was 21.6%. It would not be necessary nor desirable to add this much water to the dry fly ash for handling and placement in the UWL. However, this represents the probable moist unit weight (112 pcf) that might be achieved in the UWL without applying a controlled compaction effort. Therefore, this moist unit weight was used for the non-ponded fly ash in the various analyses.

3.2.2 Tests on Ponded Fly Ash

We ran a series of tests on a bucket sample of fly ash from the operating pond at the Labadie Energy Center. The sample was saturated when it arrived at our lab. The sample was air dried to a moisture content of 8%. We assumed that the fly ash would be excavated and dried at the pond until it would pass the paint filter test. A dry sample of fly ash was run through a U.S. #4 sieve (4.75mm opening). Water was added to achieve a specific moisture content. The water and dry sample were mixed by hand to obtain a uniform consistency. The duration of mixing was not more than 1 minute. Then, a 100-gram sample was placed in the #60-mesh conical paint filter and a timer was started. If no water dripped from the filter in 5 minutes, then the wetted sample passed the test. We determined that the ponded fly ash could pass the paint filter test if dried to a moisture content of 55%.

A sample of ponded fly ash was molded with light compaction to a dry unit weight of 60 pcf and at a moisture content of 55%. The light compaction was to simulate placing the fly ash in the UWL using only compaction by tracked earth moving equipment that is not compacting the fly ash to a specified dry unit weight. We determined that the minimum dry unit weight is about 60 pcf (moist unit weight of about 90 pcf). A staged consolidated-undrained triaxial compression tests with pore pressure measurements was run on the molded sample. The results are presented in **Figure B-7**. The effective friction angle (ϕ') was 36.4°.

Each cell of the UWL will be filled initially using fly ash from the existing pond. Therefore, the ponded fly ash will be in contact with a portion of the HDPE membrane of the top cover. We ran direct shear tests of a molded sample of the fly ash, at a dry unit weight of 60 pcf, with a sample of textured HDPE liner on the bottom plate. This was run in a standard direct shear apparatus with a sample diameter of 2 inches. We also ran direct shear tests of a molded fly ash sample against smooth HDPE liner. The peak interface friction angles (δ) were 35.2° against the textured HDPE liner, and 21.0° against the smooth

HDPE liner. The residual interface friction angles were 35.2° against the textured HDPE liner, and 17.5° against the smooth HDPE liner (see results in **Appendix B**). The peak and residual interface friction angles are similar to those reported by Koerner and Narejo (2005) for granular soil and textured HDPE liner from numerous direct shear tests:

Interface	Peak Shear Strength		Residual Shear Strength	
	Interface Friction Angle	Cohesive Shear Strength	Interface Friction Angle	Cohesive Shear Strength
Textured HDPE / Granular Soil	34°	0	31°	0
Textured HDPE / Cohesive Soil	18°	200 psf	16°	0
Textured HDPE / NW-NP* Geotextile	25°	160 psf	17°	0
NW-NP Geotextile / Granular Soil	33°	0	33°	0
NW-NP Geotextile / Cohesive Soil	30°	100 psf	21°	0

*Non-woven – Needle-punched

We ran a flexible-wall hydraulic conductivity test on the ponded fly ash. A sample was molded at 55% moisture content (the maximum moisture content that will pass the paint filter test). The initial dry unit weight of the sample was 59.2 pcf, or a moist unit weight of 90.4 pcf. The results of the hydraulic conductivity test are presented in **Figure B-10**. The hydraulic conductivity (k) was 4.5×10^{-5} cm/sec.

A one-dimensional consolidation test was run on a molded sample of the ponded fly ash to determine its compressibility. The moist unit weight of the sample was 94.9 pcf. The coefficient of consolidation (C_c) was 0.25, with an apparent pre-consolidation pressure (P_c) of 2.46 tons/ft^2. The results are presented in **Figure B-11.**

The appropriate physical properties measured by the laboratory testing on the ponded fly ash from the Labadie Energy Center that is to be incorporated in the UWL were used in all of our geotechnical analyses.

3.2.3 Tests on Bottom Ash

A sample of the bottom ash from the Labadie Energy Center was collected from the pond on December 17, 2009. The particle-size distribution results are presented in **Appendix B**. The bottom ash is poorly-graded with particle-sizes ranging from fine gravel to fine sand, with only 1% fines (passing a #200 sieve or 0.075 mm). The Specific Gravity of the bottom ash sample was 2.80.

The compaction of granular materials is based on the minimum and maximum densities determined by laboratory tests (ASTM D4253 and D4254). The minimum dry unit weight of the bottom ash is 83.6 pcf, and the maximum dry unit weight is 109.6 pcf, based upon our tests. Relative densities (D_r) of compacted granular fill in the field typically range from about 55% to 75%, which for the bottom ash sample would be dry unit weights of about 96 pcf to 102 pcf.

A staged unconsolidated-undrained triaxial compression test was run using an applied vacuum to hold the sample until the triaxial cell could be assembled. The results are presented in **Figure B-15**. The bottom ash had a ϕ' of 40.3° at a dry unit weight of about 90 pcf (D_r = 30%). There was no cohesion.

A constant-head permeability test was run on a sample of bottom ash molded at a dry unit weight of 81.7 pcf. The permeability (K) at 20°C was 0.50 cm/sec. A second sample molded at a dry unit weight of 96.3 pcf had a K at 20°C of 0.07 to 0.10 cm/sec.

3.2.4 Tests on Mixtures of CCP

We ran tests on possible combinations of fly ash, FGD gypsum and bottom ash. The ponded fly ash and bottom ash were from Ameren's Labadie Energy Center. Because it is not currently produced at the Labadie Energy Center, the gypsum was obtained from Ameren's Duck Creek Energy Center. We made cylinders of 3 different ratios of materials in the same manner described above for the fly ash alone. The 3 ratios were: 1) 46% fly ash, 20% bottom ash, 34% gypsum; 2) 30% fly ash, 25% bottom ash, 45% gypsum; and 3) 36% fly ash and 64% gypsum. The dry unit weights of the mixtures varied from 99 pcf to 87 pcf – compared to the 92 pcf dry unit weight which we determined for the non-ponded fly ash alone. The primary assumption that impacts the moist unit weight is how much water will be added to the mix prior to placement in the UWL. If only 13% water were added – which is reasonable – then the 112 pcf for the moist unit weight is appropriate for the heaviest dry mix (46% fly ash, 20% bottom ash and 34% gypsum). If we were to assume that Ameren added as much water as possible to the heaviest dry mix, then the maximum moist unit weight is estimated to be 120.4 pcf. The addition of more water to the CCP than is necessary is a time-consuming and costly activity. Therefore, it is unlikely that the in-place moist unit weight will reach 120.4 pcf and is not representative of what will occur during landfilling operations. Therefore, we used a moist unit weight of 112 pcf for the non-ponded CCP in our analyses, but ran sensitivity stability and settlement analyses to determine the possible impact of this extreme maximum moist unit weight for the combined CCP.

Gypsum and bottom ash both have larger grain-size particles than fly ash. The addition of gypsum or bottom ash to the CCP will increase the shear strength properties of the mixed CCP. Therefore, we used the shear strength properties of the fly ash alone in our stability analyses, which is conservative.

3.3 Tests on Samples from Callaway Plant Borrow Site

Details of the laboratory testing on samples from the clay borrow site at the Callaway Plant are presented in **Appendix A.** Geotechnical soil tests performed included water content (ASTM D2216) and dry unit weight, Atterberg Limits (ASTM D4318), soil finer than the #200 sieve (ASTM D1140), and grain size analysis of soil (ASTM D422). The grain size analyses were performed on samples where more than 10% by weight was retained on the #200 sieve. The results of the sieve analyses are reported in **Appendix A**. Additional tests for shear strength properties were run as described in Section 2.2 and presented in **Appendix A-1**.

3.3.1 Regulatory Requirements for Clay Liner Material

Soils for the liner must have the following properties from 10 CSR 80-11.01(10):

- Have particles with 30% or more passing a #200 U.S. sieve
- Have a liquid limit $\geq$ 20%
- Have a plasticity index $\geq$ 10%
- USCS Soil Classification of CL, CH or SC

3.3.2 Hydraulic Conductivity Tests

We collected the leftover materials from the Shelby tubes and produced two composite samples for further laboratory testing. The first composite contains silt and low plastic silty clay, and the second contained high plastic clay. Compaction tests were performed on both composites using the Standard Proctor procedure according to ASTM D698. A hydraulic conductivity test according to ASTM 5084 was completed using the silty clay Proctor point compacted nearest to 95% of the maximum dry unit weight and on the wet side of the optimum moisture content. We selected the sample with the lower liquid limit of the two clays that were compacted. The test results determined that the silty clay sample had a hydraulic conductivity (k) of $1.1\text{x}10^{-8}$ cm/sec. Clays with liquid limits greater than that tested (37%) and compacted to a similar degree will have hydraulic conductivities equal to or less than the composite sample that was tested.

3.3.3 Suitability of Callaway Plant Borrow Site

The results of the laboratory testing are summarized in **Figure 3** in **Appendix A**. Liquid Limits ranged from 28% to 101%. Plasticity indices ranged from 16% to 33%. All of the samples had 40% or more passing the #200 sieve. Therefore, all the soils described in the boring logs as low plastic silty clay, low plastic clay, medium to high plastic clay, and high plastic clay without significant amounts of sand and gravel, satisfy the requirements to be used for the compacted clay liner at the Labadie UWL.

3.3.4 Estimate of Quantities of Borrow Materials

Calculations of the estimated quantities of borrow materials are presented in **Appendix A**. The linear footage of liner quality clay in each boring was estimated using only clay with a liquid limit greater than 40 and which did not have a significant amount of sand and gravel. We estimated that clays with these parameters will result in hydraulic conductivities of less than $1\text{x}10^{-7}$ cm/sec when compacted. The total estimated amount of liner quality clay available is roughly 4.5 million cubic yards.

A second calculation was made in the same manner as the first, but using all fine-grain soils (silts and low plastic clays) that did not have significant amounts of sand and gravel. The total estimated amount of available fine-grain soil is roughly 5.7 million cubic yards. All of the fine-grain soils that do not have significant amounts of sand and gravel are expected to be suitable for the compacted clay liner; however, the additional 1.2 million cubic yards would also be suitable for final cover.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General Stratigraphy

The site of the UWL is located in the flood plain of the Missouri River. Soil deposition in the flood plain of a river is dependent on the velocity of the water – as the flood waters slow the larger size particles are deposited first, and then the finer particles. The velocities of the water vary over the flood plain and with each flood as the topography changes. Therefore, soil deposits in a flood plain ("alluvial" deposits) vary greatly both with depth and in horizontal extent. The borings and CPT soundings at the site revealed a typical alluvial stratigraphy.

The generalized logs are illustrated in the profiles in **Figures 2 through 5**. The graphic logs for the CPT soundings were derived from the detailed logs in the DSI Report. The surface soils are generally clays and silty clays with scattered seams and layers of low plastic silt, underlain by silts. The thicknesses of these fine-grain deposits ranged from 2 to 13 feet. Profile D-D' (**Figure 5**) is from the Missouri River to the southern boundary of the site. There is not an overall pattern to the stratification of the upper fine-grain soils, except for the presence of clayey sandy silt at the surface near the southern end. Section B-B' (**Figure 3**) is west to east across the site. Section B-B' also does not show an overall pattern in the upper fine-grain soils.

The upper fine-grain soils are underlain by sandy silts, silty fine sands, and fine sands, generally to depths of 22 to 36 feet. These upper sandy soils are generally loose to medium-dense. The upper sandy soils are underlain by fine to coarse, poorly-graded sands (SP), with some silty sands (SM) and gravelly sands at greater depths. These lower sands generally ranged from medium dense to very dense, increasing in density with increasing depth.

Three deep borings were extended to drilling or sampler refusal on bedrock or boulders. The final depths of the deep borings were: 91.5 feet in P-1, 104.5 feet in B-7, and 107.6 feet in B-100.

The stratigraphy of the natural soils determined by the Detailed Site Investigation was used in all of our geotechnical analyses. The UWL has been designed for the site specific subsurface conditions in accordance with MDNR requirements and generally accepted engineering practices.

4.2 On-Site Materials Available for Liner and Final Cover

The stratification of the upper fine-grain soils makes it very problematic to consistently obtain suitable clay liner material within the DSI boundaries. We judge that there is a low probability of obtaining sufficient quantity of clay liner material.

The surface fine-grain soils are suitable for intermediate or final cover material even though it would contain some fine sand. However, the high ground water levels will hinder deep borrow excavations.

4.3 Materials for Berm Construction

The surface soils within the DSI limits would be suitable for the construction of the perimeter berms. The only requirements for the perimeter berms would be the shear strength properties that were used for design, which are presented in **Table E-1** and summarized in Section 10.1.

4.4 Groundwater Levels

The existing ground surface ranges from about el. 471 to el. 465[1] below the current planned footprint of the bottom of the UWL. The areas of lower ground surface elevations (below about el. 464) located in the southeast region of the site have been excluded from the proposed developed area of the UWL.

The ground water levels at the site were monitored monthly for the DSI from December 2009 through November 2010. The data show that the alluvial aquifer discharges toward the Missouri River during periods of relatively low flow, during which time the ground water levels below the site will be 1 to 3 feet above the Missouri River level. However, when the Missouri River is above about el. 461 for a sustained period, the ground water flow reverses and the ground water levels approach the level of the Missouri River near the river (in the northwest portion of the site) and about 5 feet or more below the river level over the majority of the site. There is still a slight downward gradient toward the northeast, that is downstream.

An analysis of the observed ground water levels correlated with the Missouri River levels at the Labadie Energy Center is presented in Appendix Z of the Construction Permit Application. Based upon the 12 months of monitoring of ground water levels at the site and almost 11 years of daily Missouri River level readings at the Labadie Energy Center, using el. 464 as the average "Natural Water Table" at the site would appear to be an extreme event that occurs for a relatively short duration only about two times in a 10-year period. While it is rare that groundwater levels will ever reach the existing ground surface beneath the UWL, due to the variability of the ground water levels and to be conservative, the ground water was assumed to be at the ground surface in our stability analyses.

[1] Elevations herein refer to the North American Vertical Datum of 1988 (NAVD88) which is the datum used in FEMA's new Flood Insurance Rate Maps (FIRM). NAVD88 corrects many of the problems with the earlier NGVD of 1929.

5.0 SEISMIC RISK ANALYSES

5.1 Peak Horizontal Ground Acceleration (PHGA)

Several approaches were taken to determine the peak horizontal ground acceleration (PHGA) for the proposed UWL. The PHGA is critical for determination of slope stability under seismic loading, liquefaction potential, liquefaction settlement, potential of lateral spreading, and slope deformation. The design earthquake for this project is a 2475-year reoccurrence earthquake, or 2% probability of exceedance in 50 years (approximately equivalent to 10% probability of exceedance in 250 years). The procedure that was used followed EPA 1995 Manual *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities*, and the 1998 *Draft Technical Guidance Document on Static and Seismic Slope Stability for Solid Waste Containment Facilities* produced by the MDNR Solid Waste Management Program and Timothy Stark, Ph.D., P.E. of the University of Illinois at Urbana-Champaign.

5.1.1 PHGA from USGS Maps

The published 2008 USGS hazard map for the project site is reproduced in **Figure C-1** in **Appendix C**. This is the latest map available from the USGS website. The probabilistic PHGA for the design earthquake at the Labadie site is 0.179g (that is, 17.9% of standard gravity acceleration of 32.2 feet/sec^2). This value takes into account attenuation of bedrock shaking with distance from the probable sources and general soil interactions such as damping for a hypothetical soil profile. This value is meant to be a conservative estimate.

USGS deaggregation data were used to determine the approximate hard bedrock “outcrop” acceleration and earthquake magnitude. These data were found on the USGS website and are shown in **Figures C-2 and C-3** for St. Louis and for Labadie, respectively. The 2475-year earthquake peak hard bedrock acceleration for St. Louis and Labadie are 0.153g and 0.111g, respectively. The peak hard bedrock acceleration at Labadie is less than that for St. Louis due to attenuation of the wave from the epicenter of the probable earthquakes. Based upon the data, the most probable earthquake magnitudes (M_w) for these accelerations are between 7.0 and 8.0.

The design earthquake for this site has a PHGA of 0.179g, which has a 2% probability of being exceeded in 50 years based upon the 2008 USGS hazard maps. The corresponding peak bedrock acceleration at the site is 0.111g, and the most probable earthquake magnitudes (Mw) are between 7.0 and 8.0. This conservative PHGA (0.179g) was used in our analyses of slope stability, liquefaction, and settlement resulting from liquefaction. The Mw used for liquefaction analyses was 7.5.

5.1.2 PHGA from SHAKE2000 Analyses

A site-specific seismic analysis was completed using the program SHAKE2000. Whereas the other procedures use generalized parameters for the soil properties and earthquake motions, this procedure is more site-specific because it uses lab and field data for the soils, coupled with earthquake acceleration time histories. A site-specific seismic analysis has two components – to determine the probable seismic acceleration (or “time history”) for the bedrock beneath the site, and to determine the impact or amplification of the seismic acceleration at the ground surface due to the soils.

Ten pseudo bedrock acceleration time-histories specific to St. Louis were used in the analyses. These bedrock time-histories are provided with SHAKE2000 and illustrate the variety of earthquakes that affect this area. The development of these pseudo earthquakes is documented in the Chiun-Lin Wu and Y.K. Wen (1999) report "Uniform Hazard Ground Motions and Response Spectra for Mid-American Cities." Their method of simulation is based on the latest seismicity information in the region, and the most recent ground motion and simulation models that are appropriate for engineering applications in this region. The seismological data are mainly from the USGS open-file Report 96-532. The sets of ground motions were selected from a large pool of simulated ground motions such that the median of the response spectra matched those of the 10% and 2% exceedance in 50 years. Wu and Wen generated 8290 ground motions for St. Louis centered at 38.667º north latitude and -90.190º east longitude, which corresponds to about 6000 years of records. This point is about 35 miles closer to the probable sources of seismic events than is the Labadie UWL site. Therefore, this is considered a conservative assumption in that the bedrock accelerations at the site are expected to be less than those in the pseudo time-histories generated by Wu and Wen. All 10 provided pseudo earthquakes that had a 2% probability of exceedance in 50 years were used in our analyses. The earthquake magnitudes ranged from 5.9 to 8.0, with most being of magnitude 8. Bedrock peak accelerations averaged 0.104g, which is approximately equal to the deaggregated peak bedrock acceleration of 0.111g from the USGS data for the Labadie site. Plots of the earthquake pseudo bedrock acceleration time histories from Wu and Wen are shown in **Appendix C**.

The second step in the site specific seismic analyses – determination of the impact or amplification of the seismic acceleration at the ground surface due to the soils – was completed using the SHAKE2000 computer program. The seismic soil properties were determined based upon Boring B-100 and CPT sounding C-100. Boring B-100 was chosen because it is centrally located on the site and it extended to refusal on firm bedrock. The CPT data from C-100 were used for the top 5 feet of silts and clays. The seismic properties (shear wave velocities, damping and shear modulus) were derived from SHAKE2000 using input soil classifications, unit weights and shear strength properties from Boring B-100 and CPT sounding C-100. The inputs and outputs are included in **Appendix C**. From the analysis of these 10 pseudo bedrock time-histories, the calculated average PHGA is 0.144g for the existing site conditions, compared to 0.179g from the USGS website. Because the PHGA from the USGS hazard map is greater than that derived from our SHAKE2000 analyses for the existing site conditions, we chose to use the more conservative published USGS PHGA of 0.179g in our analyses. The SHAKE2000 time-histories were used in the Newmark analyses of deformation as described in Section 5.3.

Subsequent SHAKE2000 analyses were performed using a long-duration and a short-duration earthquake in order to determine the PHGA of the proposed landfill. These analyses were run for a 24-foot high embankment placed on the native soil, and for 100 feet of compacted CCP fill placed on the native soil. In both cases, the PHGA was found to be significantly less than the existing site conditions. This was anticipated due to the additional vertical compressive stresses in the soils created by the imposed weight of the landfill. These analyses are conservative in that they do not take into consideration the densification of the soils and the resultant increase in shear strength properties. After placement of the earthen embankment, PHGA was estimated to be 0.08g at the top of the berm and 0.12g at the bottom. After completion of the CCP fill, PHGA are anticipated to be 0.07g to 0.08g at the top of the fill and 0.10g to 0.11g at the bottom.

Our site specific seismic analyses confirmed that the PHGA based on the 2008 USGS hazard maps is conservative for this site and the configuration of the completed UWL. This PHGA was used in our analyses of slope stability, liquefaction, and settlement resulting from liquefaction. Several derived time-histories of ground accelerations from the SHAKE2000 analyses were used for the deformation analyses to obtain more accurate estimates of the probable horizontal deformation using site specific data.

5.2 Liquefaction Analyses

Liquefaction occurs when ground shaking is sufficient to produce cyclic particle movements that cause excess pore water pressures to build to the point that nearly all the strength of the soil is lost. After ground shaking has stopped, the soil will potentially reconsolidate to denser configuration, which results in settlement. Liquefaction is most problematic in loose sandy soils with less than about 35 percent fines (soils which are finer than standard sieve size #200), but can occur in very loose soils with up to 50 percent fines, and soils up to the size of fine gravel. Because these types of soils are present throughout the site, analyses were run on every geotechnical boring and CPT hole made on site. These results are included in **Appendix D**.

Factors of Safety (FS) against liquefaction were calculated for both CPT and SPT borings using the cyclic stress approach outlined in Idress and Boulanger (2008). The SPT borings were analyzed using N-values for clean sand and corrected for vertical overburden stress, termed $(N_1)_{60\text{-}cs}$ and the fines contents of the soils determined from laboratory grain size tests. The CPT soundings were analyzed using the cone tip pressure, which was corrected for overburden pressure and fines content, termed $(q_{1N})_{CS}$. The content of fine-grain soils in the CPT soundings were determined from correlation soundings and borings that were performed at the same location. We conservatively determined from these tests the following fines contents associated with the descriptions used on the CPT Logs:

CPT Log Descriptive Phrase	Fines Content (%)
Sand	1
Sand to Silty Sand	10
Silty Sand to Sandy Silt	36

The above values were the smallest fines contents found in a boring adjacent to the CPT sounding for the same CPT description, in all cases.

The design earthquake used for the calculations had a PHGA of 0.179g and magnitude (M_w) of 7.5. We used the PHGA from the USGS for the existing conditions, rather than 0.144g, because this is more conservative.

The borings and CPTs were analyzed for current ground surface conditions and for cases involving the addition of CCP fill up to 100 feet. For the cases with this additional overburden, only the effective and total overburden stresses were modified on the cyclic stress side of the liquefaction equation. We conservatively did not consider the higher resistance to liquefaction that would be gained by densification of the underlying sands due to consolidation. For each boring, in order to quantify the boring's liquefaction potential as a whole; the incremental depth factors of safety were inverse averaged together.

The inverse average weighs the factors of safety with much greater weight placed on the lower values. (This same averaging procedure is used in the International Building Code 2009 for the development of the seismic site classification.) The borings and CPTs with factors of safety less than or equal to 1.0 are shown in **Figure D-3** along with those with factors of safety less than 1.0 after 10 feet of CCP fill is placed. This figure shows the effectiveness of adding fill to decrease liquefaction potential. After 20 feet of CCP has been placed, there are no cases which still have an inverse average factor of safety less than or equal to 1.0. Additionally for demonstration purposes, the inverse averaged factors of safety were averaged together across the site, and plotted versus the height of CCP fill. This is shown in **Figure D-1**.

5.3 Estimate of Yield Acceleration and Lateral Spreading

The criterion for the seismic stability analyses of a landfill is based upon the estimated lateral deformation or spreading that may occur, rather than a factor of safety against failure with a pseudo-static seismic load (MDNR-SWMP and Stark, 1998). The procedure described by MDNR-SWMP and Stark is to calculate the yield acceleration (K_y) for the landfill geometry for which the pseudo-static seismic load results in a minimum factor of safety against slope failure of 1.0. The K_y is compared to the ground accelerations in a time-history. When the ground acceleration exceeds the K_y the associated lateral displacement is calculated using the empirical relationship developed by Makdisi and Seed (1978). Therefore, the lower the K_y of the landfill geometry with respect to the PHGA, the greater the deformation or displacement. The guidance document (MDNR-SWMP and Stark, 1998) provides an empirical graph of displacement versus the ratio of PHGA to K_y. The lateral displacement is more accurately calculated by summing over the time-history all of the displacements in the same direction. The procedure, developed by Newmark (1965), is part of the SHAKE2000 program. The proposed geometry of the berm and CCP fill was analyzed in SHAKE2000 for both a short-duration time-history (#10, M_w = 5.9, Peak rock acceleration = 0.17g, PHGA = 0.19g) and a long-duration time-history (#3, M_w = 7.1, Peak rock acceleration = 0.08g, PHGA = 0.16g). The estimated cumulative displacements for a range of yield accelerations are given in the following table:

Calculated Cumulative Lateral Deformations from SHAKE2000 Analyses

Yield Acceleration K_y	Deformation for Short-Duration Event, inch	Deformation for Long-Duration Event, inch
0.165g	0.0004	0.0
0.15g	--	0.001
0.10g	0.02	0.05
0.05g	0.73	1.02
0.04g	1.28	2.16
0.03g	2.32	4.43
0.025g	3.14	6.12

When the calculated K_y is greater than the ground acceleration in the time-history, there is no deformation. The Missouri regulations for a utility waste landfill (10 CFR 80-11.010) do not specify the maximum allowed deformation. The regulations for a sanitary landfill (10 CFR 80-3.010) stipulate that the cumulative lateral deformation must be less than 6 inches. From the above SHAKE2000 analyses, the maximum allowable cumulative lateral deformation is estimated to occur when the calculated K_y is about 0.025g.

Our analyses estimated the probable horizontal deformation due to a seismic event for a range of yield accelerations (Ky) to satisfy MDNR guidance. These analyses demonstrated that the estimated probable horizontal deformations of the UWL are much less than the maximum deformation of 6 inches allowed by MDNR for a sanitary landfill.

6.0 STABILITY ANALYSES

6.1 Stability of Final CCP Landfill

Slope stability analyses were performed on the proposed UWL profile. Generalized soil profiles were developed for 5 widely-spaced sections, the locations of which are shown in **Figure E-1** in **Appendix E**. The soil and CCP properties used in the slope stability analyses are shown in **Table E-1** and depicted graphically in **Figures 6** and **7**. These were based upon the laboratory soil testing and field testing (SPT N-values and CPT soundings) described in the DSI Report, and the laboratory testing of the CCP summarized in Section 3.2.

The slope stability analyses were performed using the computer program SLIDE 5.0. This program uses the Spencer method, which resolves the static forces on each vertical slice of soil profile along a given circular or irregular assumed failure surface. The program searches for the minimum factor of safety (FS) against slope failure for each center point in the grid by incrementally varying the radius of the failure surface. The plotted results from the program show the minimum FS, the center and radius of the failure surface with the minimum FS. The output of the program also plots contours of equal FS within the grid of possible center points. The input to the slope stability analyses and graphical representations of the results are included in **Appendix E**. The results of the stability analyses are summarized in **Table E-2**.

6.1.1 Static Analyses

Stability analyses were run for each of the five cross-sections of the UWL and subsurface soil stratification for the initial filling of the CCP and for the final configuration of the CCP, for: 1) static, 2) with seismic load (horizontal pseudo-static seismic load) and 3) with residual shear strength in potential liquefied subsurface soil strata. The appropriate shear strength properties for the CCP, compacted liner, and subsurface soils were used for each case, as previously discussed and as listed in **Table E-1**.

The DSI determined that clay will have to be imported for the compacted clay liner. Therefore, the properties of the clay liner will have to be determined by laboratory testing after the clay borrow sources are identified. For these analyses, we used conservative properties for the compacted clay liner, and interface shear properties, based upon previous testing on appropriate clays and representative published values. We used a minimum moist unit weight for the clay liner of 115 pcf, an unconsolidated-undrained cohesive shear strength (c) of 600 psf and a $\phi = 0$, and an effective ϕ' of 25° for drained conditions. We did not include the slightly higher unit weight and ϕ of the leachate collection layer (if used) and the protective aggregate layer for the global stability analyses because we used circular failure surfaces for the global stability analyses and the impact of an interface plane in the composite liner would be insignificant for a circular failure surface.

To analyze the impact of the interfaces of the HDPE and geocomposite on the slope stability, we also ran “sliding block” analyses. For the minimum shear strength along an interface in the composite liner or the geocomposite, we used a ϕ of 15° along the base of the block and no cohesion. We used a slightly higher unit weight of 120 pcf for the leachate collection layer (if used) or the protective aggregate layer that would be above the composite liner. These minimum design values should be confirmed by laboratory testing on the identified borrow clays, HDPE and geocomposite at the time of construction.

The MDNR-SWMP regulations do not specify a minimum factor of safety. The guidance document (MDNR-SWMP and Stark, 1998) recommends a minimum factor of safety of 1.5 for static stability analyses.

When each phase is constructed and authorized to accept CCP, it will be initially filled with about 18 feet of ponded CCP (approximately el. 483), for protection against heave of the liner during Missouri River flooding. This "initial" configuration was analyzed using short-term (i.e. "undrained") shear strength properties. The minimum FS ranged from 2.30 to 3.19, which is greater than the minimum required factor of safety (FS) of 1.5. The initial configuration was also analyzed using long-term (i.e. "drained") shear strength properties. The minimum FS ranged from 1.45 to 2.70, which are essentially 1.5 or greater. The actual FS in the long-term will be greater because the "initial" configuration is temporary and the fully drained shear strength properties are conservative.

It may happen in later phases of the project that previously-ponded fly ash will not be available. The laboratory tests on non-ponded fly ash described in Section 3.2.1 show that the shear strength properties of the non-ponded, moisture-conditioned fly ash are greater than that of the previously-ponded fly ash. Therefore, using the lower shear strength properties of the previously-ponded fly ash is conservative. A greater moist unit weight of 112 pcf was used for the CCP above the initial height of 18 feet in anticipation of the use of non-ponded CCP in the future.

The global stability of the completed UWL was analyzed using drained strength properties. The FS of the global stability of the CCP and berm varied from 1.46 to 2.27. The actual FS would be greater because these analyses did not incorporate the compressive strength of the CCP due to cementation, nor the gain in shear strength of the foundation soils due to consolidation.

The static analyses of a non-circular failure surface along the composite liner had a static FS of 1.99. A interface friction angle (δ) of 15º was used, to represent the minimum shear strength properties of the clay liner and textured HDPE interface, the HDPE-drainage layer interface, or the interface between the lightly-compacted CCP and the drainage layer.

Our analyses of the static slope stability of both the partially-completed and completed UWL using the various natural soil stratigraphies found on the site demonstrated that the design of the UWL satisfies the requirements of MDNR and is in accordance with generally accepted engineering practice.

6.1.2 Seismic Analyses

Numerous stability analyses were completed to determine the yield acceleration (K_y) for both the initial configuration and the final or full configuration of the landfill, as well as failure along the interface of the composite liner. For seismic analyses, we used the consolidated-undrained shear strengths of the CCP and the compacted clay liner because seismic loading is an undrained condition. The results of the stability analyses are shown in **Appendix E** and the calculated yield accelerations are summarized in **Table E-2**. The calculated K_y ranged from 0.13g to 0.17g for the full cell. The minimum K_y of 0.13g was found for the long-term conditions for the full landfill at Section B-B' and for sliding along the interface of the composite liner. From the table in Section 5.3, the calculated cumulative deformation is less than 0.05 inch, much less than the maximum of 6 inches allowed under 10 CFR 80-3.010. As a check, we also determined the lateral deformation for this section and K_y utilizing the pseudo bedrock short time-history #2 (see **Appendix C**), which had a lower magnitude than pseudo bedrock short time-history #3 but a slightly higher peak bedrock acceleration. The calculated cumulative deformation was 0.016 inch. For comparison, MDNR-SWMP and Stark (1998) state that when the K_y is equal to or greater than 80% of the PHGA, then the lateral spreading should be less than 1 cm (approximately 0.4 inch).

Our analyses of the seismic slope stability of both the partially-completed and completed UWL using the various natural soil stratigraphies found on the site demonstrated that the design of the UWL satisfies the requirements of MDNR and is in accordance with generally accepted engineering practice.

6.1.3 Impact of Potential Liquefaction

At the locations where the liquefaction analyses indicated a high potential for liquefaction in existing soil strata prior to the construction of the berm and CCP fill, residual cohesive shear strengths were input for the liquefied soil strata. The residual cohesive shear strengths were interpolated from the empirical relationships recommended by Gutierrez, et al (2004), Stark and Mesri (1992), H. Bolton Seed (1987), and Seed and Harder (1990), based on corrected N-values with corrections for fines content.

Both the initial configuration of the CCP and perimeter berm and that of the full UWL were analyzed using the post-liquefied shear strengths of the subject soil strata and no applied horizontal acceleration in accordance with the draft technical guidance document from MDNR-SWMP and Stark (1998). The results are summarized in **Table E-2**. The minimum factor of safety against the onset of liquefaction (FS_{liq}) ranged from 1.76 to 1.98 for the initial configuration, and from 1.46 to 1.77 for the completed UWL. A minimum FS_{liq} of 1.2 to 1.3 is recommended by Idriss and Boulanger (2008) to allow for errors in estimation of residual shear strengths and to limit shear strains. MDNR-SWMP and Stark (1998) suggest the same minimum FS_{liq}. Therefore, the stability of the UWL is shown to be adequate when anticipated liquefaction is present. As a sensitivity check of the conservative nature of our assumptions, we also ran the stability analyses of the five UWL sections with the fully liquefied soil strata without consideration of the impact of the overburden stress due to construction of the berms and CCP fill, as shown in Figure D-3. The FS_{liq} for this conservative assumption ranged from 1.13 to 1.72, which are slightly less than the above criterion but greater than 1.0 which is acceptable.

Before sufficient CCP fill has been placed in the UWL to eliminate the risk of liquefaction, there may be a slight risk of damage to the partially completed berms and composite liner as a result of lateral spreading, settlement or formation of sand boils. We back-calculated the "threshold" ground acceleration for the onset of liquefaction for select critical locations. The minimum back-calculated threshold ground acceleration is 0.10g. Therefore, if a seismic event would occur with a ground acceleration greater than 0.10 g before sufficient berm or CCP fill had been placed, then an investigation would have to be completed to determine whether the composite liner had been damaged. This investigation could be completed in stages. The initial stage would be a survey of the perimeter berms in those areas indicated in **Figure D-3** as the highest potential areas of liquefaction. The survey would determine whether settlement or lateral movement had occurred. Also, the area outside of the perimeter berms should be visually examined for evidence of settlement, lateral movement or sand boils. If there were evidence of liquefaction from the initial investigation, then the adjacent storm water pond would be drained for visual examination, and the bottom composite liner would be surveyed to compare with the final survey of the completed liner. If there were evidence of heave (due to sand boils), water under the HDPE liner, differential settlement, or damage to the liner, then the final stage would be to remove CCP in the affected area of the cell to examine the composite liner for similar evidence of damage. Any damaged area of the composite liner in either the storm water pond or the cells would have to be removed and replaced.

6.1.4 Stability Analyses with Potential Clay Liner Material from Callaway Plant

The potential borrow source for clay liner material at the Callaway Energy Center was identified after the initial stability analyses were completed. Subsequently, we used the shear strength properties for the Callaway clay liner material to check our stability analyses. The shear strength properties are summarized in Section 2.2 and are presented in **Appendix A-1**. The impact of the shear strength properties on the global circular stability analyses is minimal due to the thickness of the compacted clay liner. We ran a sliding block stability analysis with a failure surface through the clay liner. The results are presented in **Figure E-44**. The minimum global stability FS is 1.98, compared to the FS of 1.99 for the assumed clay liner material properties. The minimum K_y for the Callaway clay liner material is 0.145g (see Figure E-45) compared to K_y of 0.13g for the assumed clay liner material properties. Therefore, the use of the clay liner material from the Callaway Energy Center would result in the same calculated FS and greater K_y compared to the assumed clay liner material properties.

6.2 Stability of Interior CCP Berms

Interior berms are proposed to be constructed using compacted CCP from the existing ash pond. These berms would be temporary and between cells, and will eventually be buried by the CCP fill. The composite clay liner and drainage layer would extend under the interior berm, to permit extension of the liner and drainage layer for the next cell. The FS for the slope stability of the interior berm was analyzed using the drained shear strength properties of compacted CCP. The CCP should be compacted to a minimum 95% of the maximum dry unit weight from a standard Proctor moisture-density test. The minimum FS for a global circular slope failure and the full height of CCP fill is 1.91. The minimum FS for a sliding block failure along the extension of the composite clay liner and drainage layer beneath the interior berm is 1.59. The K_y is 0.06g for a sliding block failure. From the table in Section 5.3, the calculated lateral deformation is about 1 inch, which is less than the maximum allowable 6 inches.

6.3 Stability of Final Cover

The stability analysis of the final cover on the side slopes is shown in **Appendix E**, using 2 foot of nominally compacted soil over a double-textured HDPE membrane. The shear strength along the interface between the soil cover and the HDPE is based upon an interface friction angle of 15º and an adhesion of 246 psf, which governs the minimum FS. The calculated FS for the saturated soil cover with seepage parallel to the slope is 3.78. The FS with a pseudo-static horizontal force of 0.179g is 2.61.

6.4 Bearing Capacity Analysis

The bearing capacity of the stratified foundation soils was analyzed using SLIDE 5.0 with an uniform load applied to the surface and assuming a circular failure surface. The results of the analysis are shown in **Figure E-43**. The ultimate bearing capacity of a semi-infinite continuous load on the surface is 5000 psf. For a factor of safety of 2.0, the allowable bearing pressure is 2500 psf. This bearing capacity is applicable to the unconfined, original (unconsolidated) soil strata at the end of the perimeter berm. The bearing capacity below the CCP fill is much greater due to the confinement of the soil strata by the perimeter berms. Our analysis of the ultimate bearing capacity of the surface soil strata represents the "worst case" condition because it did not include the effect of consolidation of the soil strata and confinement under the weight of the CCP fill. Therefore, the ultimate bearing capacity of 5000 psf is applicable to the edge of the perimeter berm.

To estimate the bearing capacity beneath the completed CCP fill, we used the consolidated-undrained shear strength properties of the surface silty clay. The undrained shear strength (s_u) of the silty clay beneath the completed CCP fill would be about 2800 psf. If the consolidated natural soil beneath the completed CCP fill were homogeneous silty clay, then the ultimate undrained bearing capacity would be 9 times s_u or about 25000 psf. If the maximum pressure beneath the completed CCP fill is about 10,800 psf, then the factor of safety against a bearing capacity failure in the natural soil would be about 2.3, which is in accordance with generally accepted engineering practice.

6.5 Stability Analyses with Maximum Unit Weight of Non-Ponded CCP

As explained under Section 3.2.1 and 3.2.4, our analyses used an average in-place unit weight of the non-ponded CCP of 112 pcf, whether for wetted fly ash or moistened combined CCP. If the combined CCP were mixed with as much water as possible without failing the paint filter test, an unlikely and more costly option, the maximum unit weight of the combined CCP could be as high as 120.4 pcf (see Section 3.2.4). To check the sensitivity of our assumed unit weight of 112 pcf, we ran the stability analyses for the full CCP fill with 120.4 pcf for the non-ponded mixed CCP above el. 483. These results are shown in **Table E-2**. The factors of safety were 0.04 lower for profiles B-B' and D-D', but were unchanged for the other sections.

The unit weight of the CCP fill can only be estimated at this time, because the composition and method of placement (wet ponded CCP or dry non-ponded CCP) may vary. Our analyses using the range of probable unit weights demonstrates that the design of the UWL, and resultant factors of safety, satisfy the requirements of MDNR and are in accordance with generally accepted engineering practice regardless of the probable range of unit weights of the CCP.

7.0 SETTLEMENT ANALYSES

7.1 Estimated Settlements

Settlement analyses were completed using one-dimensional consolidation theory (Terzaghi and Peck, 1948) using the computer program SETTLE3D. The program calculates the effective vertical stress at depths for a uniform surface load on an assumed elastic half-space using the Boussinesq stress distribution. SETTLE3D does not allow for variations in subsurface soil conditions. Therefore, the program was run for multiple soil profiles. The soil profiles were developed for circles as shown in **Figure F-1,** combining the data from the pertinent borings and CPT soundings for each circle. The development of the soil profile for each circle is shown in hand calculations in **Appendix F**. The settlement values were calculated at the circles for the final configurations of full Cells 1 and 2, and full Cells 3 and 4. The configuration of the CCP fill is represented by a combination of uniform surface loads of varying dimensions. The profile used to calculate the surface loads is illustrated in **Figure 8**. The input loads are presented in the output for each circle in **Appendix F.** The plan view of the cumulative surface loads are depicted in the output from SETTLE3D in **Figure F-8**. The soil stratification at a given circle is modeled, and the settlement at the surface is computed for each load configuration. The results were graphed to produce the estimated settlement of the subgrade along four cross-sections of the completed landfill and along the existing Explorer pipeline.

Consolidation coefficients (C_C and C_R) for cohesive materials were obtained from load increment consolidation tests run on representative undisturbed samples from the DSI. The stress-strain modulus (E_S) for granular materials was estimated using cone penetration test (CPT) data obtained from the DSI. E_S is approximately 4 times the measured CPT q_c-value of resistance (Lunne et al, 1997). This multiplier of 4 was the minimum that was applicable for recent normally-consolidated sands or “aged” normally-consolidated sands for an average axial strain of 0.1%, which is applicable to this site. The calculated values of Es from the CPT data and the range of values used in the settlement analyses are plotted in **Figure F-6** in **Appendix F.**

Settlements of the natural subsurface soils were calculated along four profile lines, as shown in **Figures F-2 through F-5**. Generally, the calculated settlement at the top of the perimeter berms varied from 5.5 inches to 9 inches. The calculated settlement at the inside toe of the perimeter berms, where the leachate collection sumps will be located, ranged from 10 to 17 inches.

The calculated settlements at the midpoint of the CCP slope ranged from 14 to 20 inches, and at the top of the 1(v)-to-3(h) slope ranged from 18 to 26 inches. The maximum calculated settlement in the center of the CCP fill was 26 inches.

7.2 Liquefaction-Induced Settlement

Liquefaction settlement for the SPT borings was determined using the procedure outlined in Idress and Boulanger 2008, which determines the post-liquefaction volumetric strain based upon the corrected-normalized N-value $(N_1)_{60}$ and the calculated factor of safety against liquefaction. For CPT soundings, volumetric strain was determined using the procedure outlined in Zhang et. al. (2004) which uses the corrected-normalized-clean sand equivalent-point resistance $(q_{C1N})_{CS}$. The average liquefaction-induced settlement associated with different quantities of fly ash fill are shown in **Figure D-2**. These values do not account for settlement and are in addition to the normal consolidation settlement or immediate settlement. As can be seen in this figure, the addition of fill significantly reduces the estimated liquefaction induced settlement. There is one location along the southern edge of Cell 1 where there is a potential for liquefaction beneath the perimeter berm with the addition of 10 feet of CCP fill. The estimated liquefaction-induced settlement is 3 inches. This amount of settlement creates inconsequential additional strain on the HDPE liner. After 20 feet of CCP fill has been placed, there are no potential areas of liquefaction beneath the landfill, so there is no potential liquefaction-induced settlement.

Prior to the placement of sufficient CCP to mitigate the liquefaction potential, an investigation would be completed if a seismic event with a PHGA of 0.10g or greater would occur, as explained in Section 6.1.3.

7.3 Strain of HDPE Liner and Leachate Collection System

The estimated settlements will occur over long distances, such that the differential settlement will be small, at a slope of about 1%. The liner will undergo a maximum differential settlement of about 5 inches between the crest of the perimeter berm to the inside toe of the berm (a horizontal distance of 69 feet), and about 11 inches from the inside toe of the berm to a point below the crest of the CCP fill (206 feet). The increase in lengths of the slopes after full settlement has occurred compared to the initial lengths will be 0.002% and 0.001%, respectively. A strain of less than 1% is acceptable since the yield strength of most HDPE liners occurs at more than 12%. Therefore, the strain in the HDPE liner resulting from the estimated differential settlements will not negatively impact the liner.

7.4 Impact of Settlement on Existing Explorer Pipeline

Cells 1 and 2 will be constructed along the west side of the existing Explorer pipeline. Cells 3 and 4 will be constructed along the east side of the pipeline. The plan leaves a 100-foot buffer between the pipeline and the toe of the berms. We calculated the settlement along the pipeline that would result from completing the CCP fill for Cells 1 through 4, and from construction of the two roadway berms if nothing were done to mitigate the settlement. The calculated settlements are plotted in **Figure F-7** in **Appendix F.** The maximum calculated settlement is less than about ¼-inch except in the vicinity of the two roadway berms, which is within the error of the method of analysis. We judge that this amount of settlement is inconsequential. The maximum calculated settlement beneath the two roadway berms is about 4.5 inches, over a distance of about 140 feet, which is a rotation of about 0.3º. As stated in the CPA Report, this issue will be resolved with Explorer Pipeline during the final design of the future expansion to Cells 3 and 4.

7.5 Settlement Analyses with Maximum Unit Weight of Non-Ponded CCP

As explained under Section 3.2.1, our analyses used an average in-place unit weight of the non-ponded CCP of 112 pcf, whether for wetted fly ash or moistened combined CCP. If the combined CCP were mixed with as much water as possible, an unlikely and more costly option, the maximum unit weight of the combined CCP could be as high as120.4 pcf (see Section 3.2.4). To check the sensitivity of our assumed unit weight of 112 pcf, we ran settlement analyses for the full CCP fill with 120.4 pcf for the non-ponded CCP above el. 483. For the settlement in the central area of a given cell, the increase in the anticipated settlement is an average of 1 to 1.5 inches, or about 4.5% to 6.6%. The maximum increase in settlement is 1.3 to 2 inches, or about 4.9% to 7.1%. Given the inherent variations in properties of both the CCP and the natural soils, and the accepted method of estimating settlement, no revision to our original settlement estimates is necessary to accommodate the unexpectedly higher unit weight of mixed, saturated CCP.

8.0 HYDROSTATIC PRESSURES

8.1 Flood Levels for Design

The UWL site is currently protected from regular Missouri River flooding by the Labadie Bottom Levee District agricultural levee with heights at or near the 100-year flood elevation. In the unlikely event that the agricultural levee is overtopped or breached, the UWL site is further protected from direct Missouri River flood currents by the Labadie Energy Center itself which is upstream and higher than the 500-year flood elevation, creating a low velocity shadow, or ineffective flow area, over the entire UWL site. The regulatory 100-year base flood elevation (BFE) of 483.98 at the upstream end of the UWL site became effective on October 18, 2011. The 500-year flood elevation at this river station is reported by FEMA to be 487.55. By comparison, the flood crest at this location in August 1993 was about el. 483.6. The planned top of the constructed perimeter berms of the Labadie UWL will be at el. 488.

8.2 Protection of Liner from Hydrostatic Uplift

A flood condition surrounding the UWL would impose a hydrostatic uplift pressure on the bottom of the composite liner. This uplift pressure is initially only resisted by the weight of the composite liner, specifically the compacted clay, before the leachate collection layer or any fill is placed in the cell. To maintain a factor of safety (FS) of 1.1 against upward displacement and rupture of the liner, the 2 feet of clay can resist an upward pressure equal to about 3.3 feet of water. Therefore, the level of the flood water surrounding the cell must remain no more than 3.3 feet above the clay liner before CCP fill is placed. If the 12-inch gravel leachate collection layer is used, then the flood water surrounding the cell must remain no more than 5.25 feet above the gravel layer before CCP fill is placed. Once the 12-inch thick protective sand layer is in place, the maximum allowable difference in height between the water level outside of the berm and top of the protective sand layer is 7.0 feet for a FS of 1.1.

CCPs from the existing ash pond will be placed immediately after receipt of authorization to operate the UWL to protect the compacted clay liner against heave from hydrostatic uplift. The required height of the CCP fill may be calculated using the equation illustrated in **Figure 10**. We have assumed that the initial CCPs will be placed at a moist unit weight of 90 pcf. For simplicity in the calculation, we assumed that the bottom of the clay liner is at the top of the sand or permeable layer. For example, if the base of the clay liner were at el. 466 at the lowest point, then the uplift hydrostatic head (H_w) for the 100-year flood level (el. 484) would be 18 feet. The required height of the CCP (H_{CCP}) for a FS of 1.1 is 8.1 (el. 478.1) feet with the 12-inch gravel leachate collection layer, or 9.5 (478.5) feet if a light weight geo-composite is used in lieu of the gravel layer. These are examples to illustrate the calculation; the actual calculations of the heights of CCP required are included in **Appendix Y.**

9.0 EROSION PROTECTION FROM LEVEE OVERTOPPING OR FAILURE

Franklin County amended their Unified Land Use Regulations on October 25, 2011 to add regulations concerning Non-Utility Waste and Utility Waste Landfills (UWL) in Franklin County, Missouri. Article 10, Section 238(C)(3) of these amended regulations requires in part that:

d.) All "cells" shall be designed and constructed so that they shall be protected by an exterior berm meeting the following criteria:

- *i.) The top of the berm at a minimum shall be equal to the five hundred (500) year flood level in the area of the proposed Utility Waste Landfill.*
- *ii) … All berms shall be constructed of concrete or cement-based material sufficiently thick for the purpose intended and approved by the Independent Registered Professional Engineer.*

Some exterior berms may infrequently be in contact with flood water from the Missouri River, but only if the Labadie Bottom Levee District levee is overtopped or breached. A floodplain analysis performed by CDG for Ameren Missouri estimated that the maximum velocity that may occur is less than 2 feet/second along the west berm of Cells 1 and 2. The interior berms may also infrequently come in contact with flood water, but the water velocities will be too low to cause erosion. In both instances a vegetated cover alone would provide sufficient erosion protection, as with standard levee design. To meet Franklin County regulations, concrete and/or cement-based material will be used to prevent possible erosion of the exposed slopes of perimeter berms that may be subject to the flow of flood water.

The exterior slopes of all perimeter berms will be covered with a fabric-formed concrete mat (FCM) as illustrated in **Figure 9**. The design of the FCM is presented in **Appendix G**. A 56mm thick FCM, such as Hydrotex FP220, will provide adequate protection for flows up to 11.4 feet/second when placed on a 1(v)-to-3(h) slope of cohesive soil. A non-woven filter geofabric will be placed between the FCM and the compacted soil of the berm to prevent loss of soil through the drainage openings in the FCM. The 56mm thickness is the minimum required for the anticipated velocity of flow. However, a thicker FCM may be used for constructability and durability. The final design of the FCM may include anchor rods.

10.0 CONSTRUCTION RECOMMENDATIONS

10.1 Field and Laboratory Classification of Soils

As discussed previously, the clay for the liner and top cover will be imported. While preliminary tests have been completed on the clay borrow material from Ameren's Callaway Plant, additional tests will be needed before these soils can be placed as the clay soil component of the composite liner system. As an alternate, the contractor for each phase may be permitted to import clay liner material from another off site source. If this alternate is accepted, the contractor will be required to identify and provide access to the off-site borrow sites for geotechnical materials testing of the proposed clay liner quality soils with sufficient lead time to complete exploratory investigation, sampling and testing prior to transporting the off-site soil materials onto the UWL site. Hydraulic conductivity tests on compacted clay samples may require 2 months to complete. We suggest stockpiling an adequate volume of clay liner material for each phase on site prior to the start of the clay liner construction. This would provide adequate time to perform the required test pad construction and testing prior to the start of construction, and would help ensure that an adequate supply is on hand throughout the liner construction. Clay soil materials to be used for clay liner construction must be tested and subsequently placed in accordance with the site specific state-approved CQA Plan for the UWL. Section 10.2 below describes the testing and placement criteria. To verify that future, constructed compacted clay liners meet the minimum criteria used in our above analyses, the clay should have a minimum undrained shear strength of 600 psf, a moist unit weight of 115 pcf, a drained internal friction angle of 25º, and a minimum interface friction angle with the HDPE liner of 15º, in addition to meeting the other requirements established in 10 CSR 80-11.01(10).

The requirements for the soil to be used to construct the perimeter berm are less stringent. Off-site sources may be tested several days prior to use of the fill material on site. Continuous monitoring by a geotechnical engineer or a qualified soils technician working under the direction of a geotechnical engineer will be required to ensure that the imported soil fill has consistent properties, such as grain-size, plasticity, and compaction characteristics. The general berm fill when compacted should have a minimum undrained shear strength of 1000 psf, an approximate moist unit weight of 120 pcf, and a drained internal friction angle of 30º or greater.

10.2 Compaction Criteria

Grab samples of liner material will be tested for grain-size distribution (i.e. hydrometer test), and liquid and plastic limits. If any volume of the stockpile differs significantly in these index properties, then that volume can be delineated, and a separate compaction criteria can be developed for that material, or it can be rejected as liner material. Compaction criteria for clay liner material will be developed using the "Daniel Method." Daniel and Benson (1990) have determined that compaction criterion as a percentage of the maximum dry unit weight alone is not sufficient to assure the required minimum hydraulic conductivity. They recommend a series of compaction tests and hydraulic conductivity tests on each soil type to determine the acceptable "window" of dry density and moisture content that will meet the hydraulic conductivity requirements which will require up to 3 months to complete.

The stability of the perimeter berm requires higher shear strength than for the liner. Therefore, the average compaction of the materials in the perimeter berm should be no less than 95% of the maximum dry unit weight determined by the standard Proctor moisture-density test, with no tests less than 92% of the same maximum dry unit weight. The moisture content at the time of compaction should be at optimum or a maximum of 4% above optimum. The engineering properties of the berm materials compacted to the above minimum criterion must meet or exceed the following: moist unit weight of 120 pcf, undrained shear strength of 1000 psf, drained cohesion of 0, and a drained internal friction angle of 30º.

Fills should be placed in horizontal lifts not exceeding 8 inches in loose thickness and compacted by uniform coverage with a suitable compactor. Cohesive fill should be compacted using a heavy tapered-foot compactor, with or without vibration. The final lift of cohesive fill should be compacted by a smooth-drum roller. Cohesionless fill, if any, such as the silty sand or fly ash, should be compacted by a heavy vibratory compactor.

10.3 Construction Quality Assurance

10.3.1 Test Pad

The plasticity index of some of the clay liner material from the Callaway Plant exceeded 30%. Therefore, a test pad will be required prior to construction to test the materials to be used for the liner, and the construction methods. The test pad must be large enough to accommodate the actual construction methods and equipment that will be used for the construction of the liner. The compaction criteria previously developed for the liner material will be used to construct the test pad. In accordance with MDNR-SWMP regulations, the geotechnical testing agency is required to take undisturbed samples of the fill to measure the density and hydraulic conductivity. Bulk samples of the fill material must be taken to perform LL and PI tests and standard Proctor tests. Also, a minimum of two test pits are required to examine the interface between lifts of materials, to verify bonding of the lifts. A field permeability test is also required. A test pad is not necessary for the fill to be placed in other areas, such as the perimeter berm.

10.3.2 Quality Assurance during Construction

The successful completion of the test pad will verify the acceptable construction methods for the liner for the known material from the liner materials stockpiled on site. **Appendix P** of the CPA Engineering Report provides a construction quality assurance plan for the composite liner system which will be followed to document adequate minimum construction of the composite liner system.

10.4 Investigation and Remediation of Possible Liquefaction Damage

As discussed in 6.1.3, there is potential for damage to the composite liner during construction before a sufficient amount of CCP fill had been placed in the UWL, which is about 20 feet. A procedure for an investigation is presented in Section 6.1.3. A topographic survey of the liner will have been completed for the CQA of the liner. Permanent benchmarks will need to be installed along the perimeter berms to perform an accurate horizontal survey to detect movements that may have occurred, since the calculated lateral deformations are very small.

11.0 RECOMMENDATIONS FOR INITIAL OPERATION

The initial filling of each cell must take into consideration protection against heave of the bottom composite liner due to flooding outside of the cell, and possible damage to the liner due to liquefaction resulting from a seismic event with a PHGA of 0.10g or greater.

Protection against heave of the bottom composite liner due to flooding has been discussed in Section 8.2. The placement of CCP in each cell will have to be expedited to minimize the risk of a significant flood or high water event occurring before the cell has sufficient CCP fill. This will include a stand-by plan to flood the cell with flood water pumped over the perimeter berm.

The potential for damage due to a seismic event has been discussed in Section 6.1.3. The initial filling of Cell 1 should begin on the west side where liquefaction potential remains after the construction of the berm, and on the east aide (see **Figure D-3**). Similarly, the initial filling of Cells 3 and 4 should begin on the west and south sides, respectively, to mitigate the potential for liquefaction.

There is risk of liquefaction beneath storm water Ponds 1 and 3 after completion of the berms for Cells 1 and 4, unless the ponds are filled with water at the time of a significant seismic event. This would not impact the stability of the CCPs in Cells 1 and 4 or the composite liner. If this occurs, an investigation of Ponds 1 and 3 should be completed, as outlined in Section 6.1.3.

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