REVISION 0 – CLIENT ISSUE

2021 PERIODIC SAFETY FACTOR ASSESSMENT REPORT

BOTTOM ASH SETTLING AREA

JEFFREY ENERGY CENTER

St. Marys, Kansas

BLACK & VEATCH PROJECT NO. 192728
BLACK & VEATCH FILE NO. 41.0403

PREPARED FOR

Evergy Kansas Central, Inc.

26 AUGUST 2021

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Professional Engineer Certification

This Periodic Safety Factor Assessment Report documents the evaluation of the Jeffrey Energy Center Bottom Ash Settling Area consistent with applicable sections of 40 CFR § 257.73 and documents compliance with the U.S. Environmental Protection Agency Coal Combustion Residual Rule.

I hereby certify that the 2021 Periodic Safety Factor Assessment Report for the JEC BASA was conducted in accordance with the requirements of 40 CFR § 257.73 (e).

Signed:____
Professional Engineer

GARY DEAN SOMMERFELD
LICENSED
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PROFESSIONAL ENGINEER
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1.0 Executive Summary

This report presents a summary of the 2021 periodic safety factor assessment for the Evergy Jeffrey Energy Center (JEC) Bottom Ash Settling Area (BASA) near St. Marys, Kansas. The 2021 periodic safety factor assessment was completed in compliance with 40 CFR § 257.73(e) and includes compilation of the history of construction and modifications in compliance with 40 CFR § 257.73(c), as well as review of available information regarding the impoundment and inspections of the impoundment and appurtenant structures. This periodic assessment is an update to the initial assessment performed in 2016. The overall steps for the safety factor assessment are shown in Figure 1-1.

1.1 SUMMARY OF FINDINGS

The periodic safety factor assessment for the Bottom Ash Settling Area berm confirms that the calculated factors of safety equal or exceed the minimum safety factors for required by the CCR Rule (Table 1-1).

TABLE 1-1
2021 PERIODIC SAFETY FACTOR ASSESMENT RESULTS

LOADING CONDITION	MINIMUM FACTOR OF SAFETY(1)	CALCULATED FACTOR OF SAFETY
Long-term-maximum storage pool	1.50	1.56
Maximum surcharge	1.40	1.58
Seismic loading	1.00	2.82
Soil Liquefaction(2)	1.20	N/A(3)

Notes:
(1) CCR Rule Safety Factor Requirements (§257.73(e)).
(2) Soil liquefaction case is only required if soils are identified as having potential for liquefaction under seismic loading.
(3) Soils were determined to be non-liquefiable.

2.0 Bottom Ash Settling Area Characterization

2.1 LOCATION AND GENERAL DESCRIPTION

The JEC Bottom Ash Settling Area (BASA) is in St. Marys, Kansas, within Pottawatomie County, northeastern Kansas (Figure 2-1). The latitude and longitude of the BASA center is approximately 39.286N, 96.117W. On April 11, 2021, the BASA was removed from operation and flow into the impoundment ceased.

The BASA is a valley fill surface impoundment that previously collected bottom ash from the main plant. The bottom ash was delivered to the BASA as slurry via multiple pipes at the east end of the impoundment. The heavier bottom ash settled in the area near the slurry supply pipe outlet and was routinely removed to allow the water to flow into the remaining portion of the BASA where the suspended bottom ash was allowed to settle. An outlet pipe near the berm allows the clear water to exit. Bottom ash is removed from the BASA by excavation, dewatered at the adjacent Bottom Ash Landfill (BAL), and used beneficially or is placed in the BAL.

2.2 IMPOUNDMENT DESIGN/CONSTRUCTION HISTORY

Information provided by Westar Energy for a 2009 Black and Veatch Inspection and Engineering Evaluation report and the 2016 Annual Inspection Report prepared by Haley and Aldrich, Inc. (H&A) for the BASA indicated that the impoundment was constructed in the 1980's initially as a small, non-engineered structure. The berm was later enlarged by using a mixed fill consisting of fly ash and bottom ash spread and compacted in 1 to 2-foot lifts. Compaction generally was accomplished by using the dozers and scrapers that were used to place the material. No construction records were provided as part of this assessment. According to the 2009 report, the BASA has an inlet invert pipe elevation of 1231.72 feet and an outlet invert elevation of 1205.58 feet. The outlet pipe discharges to an open channel that continues towards the next bottom ash pond.

No design drawings were provided by Westar Energy. Three borings were drilled as part of the 2009 inspection and evaluation study (Figure 2-2). All three borings indicate that the fly ash/bottom ash mix (silty sand) rests in contact with weathered rock and bedrock shale and limestone. Based on the borings, the native soil (silty clay) appears to have been removed prior to building the berm.

2.3 IMPOUNDMENT MODIFICATIONS

In 2012 the BASA underwent a vertical expansion being raised by approximately 4 feet. The 2016 H&A Annual Inspection Report indicated that the berm was raised using a mixture of fly ash and bottom ash compacted in 8-inch-thick lifts. Based on Black & Veatch discussion with facility staff, there was no construction documentation of the vertical expansion. During this work, the inlet invert pipe elevation was also raised to 1239.5 feet.

In 2016, the face of the BASA berm was armored with rip-rap in to better control erosion. Vegetation clearing and regrading of the surrounding area was also completed at this time. In 2020, the inlet invert pipe was lowered to 1237.75 feet.

2.4 CURRENT IMPOUNDMENT DIMENSIONS AND CAPACITIES

Based on the December 2019 topographical survey, the berm has a nominal crest elevation of 1,241.6 feet. Elevation at the downstream toe of the berm at the lowest point is approximately 1,198 feet resulting in a maximum berm height of approximately 45 feet. The berm is 40 feet wide at the crest and approximately 1,500 feet long. In 2020, APTIM Environmental & Infrastructure LLC (APTIM) determined the BASA had an estimated storage capacity of 534,000 cubic yards.

2.5 IMPOUNDMENT INSTRUMENTATION

Currently, no instrumentation exists in the berm at the BASA. Two piezometers were installed during the 2009 Black & Veatch investigation; however, the 2016 H&A Annual Inspection Report noted that the piezometers were non-functioning and removed shortly after the inspection.

2.6 IMPOUNDMENT INSPECTIONS

In accordance with the CCR Rules, a visual inspection of the BASA is performed by Westar Energy Inc. on an interval not exceeding seven days. The unit is inspected for any signs of potential structural weakness or other conditions that have the potential to disrupt the operation or safety of the impoundment.

The following previous inspections were also reviewed as part of this assessment.

2.6.1 2009 Black & Veatch Inspection

Black & Veatch performed a visual inspection of the BASA in 2009 as part of the engineering evaluation. The inspection indicated no signs of instability; however, several areas of erosion were noted.

2.6.2 2015 Haley & Aldrich Annual Inspection

Haley & Aldrich performed an annual inspection of the BASA on 8 October 2015. According to the report, the elevation of the pool at the time of the inspection was 1239.5 feet. Based on their inspection, no signs of instability or unusual movement of the berm was noted. Haley & Aldrich did note several areas of seepage and erosion along the face of the berm.

2.6.3 2016 Black & Veatch Inspection

As part of the initial safety factor assessment, Black & Veatch performed a visual inspection of the impoundment on 29 July 2016. The primary objective of the inspection was to observe the berm slope conditions and identify any issues that would affect the stability of the berm. Consistent with the 2015 H&A Annual Inspection, Black & Veatch also observed several areas of seepage evidence along the downstream slope face of the berm. At the time of the inspection, measurements indicated that the seepage was occurring fairly consistently approximately 50 feet from the crest edge of the slope.

2.6.4 2016 CB&I Annual Inspection

CB&I Environmental and Infrastructure, Inc. (CB&I) performed an annual inspection of the BASA on 29 November 2016. According to the report, the elevation of the pool at the time of the inspection was 1239.5 feet. The report noted overgrown vegetation and erosion areas along the embankment had been repaired. During the inspection, rip-rap was being placed on the face of the embankment. Minor erosion was noted at the base of the embankment at the location of the discharge pipe. Based on their inspection, no signs of instability or unusual movement of the berm was noted.

2.6.5 2017 APTIM Annual Inspection

APTIM Environmental & Infrastructure LLC (f/k/a CB&I) performed an annual inspection of the BASA on 6 November 2017. There was little to no water in the surface impoundment. Water was moving through a small channel within the surface impoundment to the outlet, as designed. Based on their inspection, no signs of instability or unusual movement of the berm was noted.

2.6.6 2018 APTIM Annual Inspection

APTIM performed an annual inspection of the BASA on 28 November 2018. Based on their inspection, no signs of instability or unusual movement of the berm was noted.

2.6.7 2019 APTIM Annual Inspection

APTIM performed an annual inspection of the BASA on 4 December 2019. It was noted in the report that dredging of CCR material had occurred within the BASA. Based on their inspection, no signs of instability or unusual movement of the berm was noted.

2.6.8 2020 APTIM Annual Inspection

APTIM performed an annual inspection of the BASA on 2 December 2020. Based on their inspection, no signs of instability or unusual movement of the berm was noted.

3.0 Subsurface Characterization

The initial step in the safety factor assessment was to gather and review the existing information on the BASA to fully characterize the subsurface conditions of the berm. Black & Veatch reviewed the existing subsurface investigations and analysis to determine if any data gaps existed. The results of the data collection and data gap process are described in the following sections.

3.1 PREVIOUS INVESTIGATIONS

3.1.1 Initial Geotechnical Investigation

The subsurface investigation for the Jeffrey Energy Center was conducted in 1974. No soil borings were performed in the immediate area of the BASA as part of the subsurface investigation for the plant, coal storage area, and railroad spurs. The closest boring that was part of the initial investigation is located on the opposite side of the north-south railroad spur east of the BASA.

The existing berm and impoundment developed from a small non-engineered impoundment that was collecting bottom ash. It appears that no borings were performed as part of the design and development of the early impoundment.

3.1.2 2009 Investigation

In 2009, Black & Veatch was contracted to perform an inspection and evaluation of the BASA berm at Jeffrey Energy Center. The study included:

- Site monitoring and inspection of the berm
- Survey of the berm
- Geotechnical investigation
- Slope stability analysis of the berm
- Report of results

The geotechnical investigation included three borings along the crest of the berm (Figure 2-2). The depths of the borings varied from 31 feet to 61 feet, and each boring cored at least 10 feet into the underlying bedrock. Sampling included Standard Penetration Tests (SPT), Thin-walled samples (Shelby Tubes), bulk samples, and rock cores. Laboratory testing includes soil moisture, dry density, Atterberg limits, grain size analysis, unconfined compressive strength, and direct shear testing. A standard Proctor test was performed on the fly ash/bottom ash material that is used as berm material.

The boring logs indicate that the soil underlaying the berm is characterized as very dense silty sand that is gray, brown, reddish orange, or tan, fine grained, and contains a trace to some gravel. Layers with a trace of cementation were also observed. The silty sand overlies bedrock composed of grayish green shale or tan to orange limestone. The shale ranges from highly weathered (can be broken by hand, texture indistinct, and fabric intact) to residual soil (advanced state of decomposition resulting in plastic soil). Limestone was encountered at the base of the berm in boring B-3 and was described as moderately weathered (discoloration throughout, slight loss of strength, and texture intact).

Two piezometers were installed at soil borings B-1 and B-3 as part of the 2009 geotechnical investigation (Figure 2-2). The piezometer at B-1 (B-1A) was installed with the screened interval from a depth of 19 feet to 29 feet, which would be at the base of the berm. The piezometer at B-3 (B-3B) was installed with a screened interval from 12.5 feet to 22.5 feet, which is also at the base of the berm. A measurement of groundwater during drilling was at a depth of 28.5 feet. This value should be considered suspect because it was below the base of the berm. Two other values were reported on the piezometer construction logs; however, the date of measurement is not consistent with the piezometer logs. The measured water depths at B-1A and B-3B were 19.1 feet (elevation 1225.9 feet) and 12.1 feet (elevation 1226.9 feet), respectively. Both of these depths would place the groundwater surface within the berm.

3.1.3 2014 Survey

As documented in the 2016 Annual Inspection Report, a combination topographic and bathymetric survey was completed by Professional Engineering Consultants in 2014.

3.1.4 2016 Monitoring Wells

In March 2016 Haley & Aldrich installed a total of six monitoring wells around the area of the BASA berm. The locations of these six borings are shown on Figure 2-2.

3.1.5 2016 Supplemental Investigation

As part of the initial factor of safety assessment, three test pits were excavated along the crest of the BASA to obtain samples of the fill material used to raise the BASA. Samples were evaluated for compacted unit weight and strength. The locations of these three test pits are shown on Figure 2-2.

3.1.6 2017 Survey

A topographic survey of the BASA was performed by Professional Engineering Consultants in March 2017.

3.1.7 2018 Survey

A topographic survey of the BASA was performed by Professional Engineering Consultants in March 2018.

3.1.8 2018 Location Restrictions Demonstration Investigation

In September 2018 Haley & Aldrich performed six soil borings within the BASA. Borings were advanced through the BASA to determine the bottom the CCR material and evaluate the separation between the bottom of the unit (base of CCR) and the upper most aquifer. The locations of these six borings are shown on Figure 2-2

3.1.9 2019 Survey

A topographic survey of the BASA was performed by Professional Engineering Consultants in December 2019.

3.2 DATA GAP ANALYSIS

Black & Veatch completed a data gap analysis of the existing available information to identify if there were any data gaps that would need to be filled prior to completing both the factor of safety and liquefaction analysis. Table 3-1 presents a matrix that presents the results of the data gap analysis.

TABLE 3-1
DATA GAP ANALYSIS MATRIX

REQUIRED DATA	QUALITATIVE EFFECT ON ANALYSIS	INFORMATION SOURCE	QUALITY OF DATA	DATA GAP
Slope Geometry	High	Topographic/Bathymetric survey in 2014 and 2019	High	No
Material Unit Weight	Medium	2009 B&V Report and 2016 Supplemental Investigation	High	No
Material Strength	High	2009 B&V Report & 2016 Supplemental Investigation	Medium	No
Groundwater Conditions within Slope	Medium	Seepage observations /Annual Inspections/ 2018 Location Restrictions Demonstration Investigation	Low (no piezometer measurements in dike)	No
Seismic Loading	Low (minimal seismic loading in this area)	USGS Seismic Hazard Map	High	No
Required Design Margins	High	CCR Rules	High	No

Based on the data gap analysis, Black & Veatch determined there are no data gaps for performing the stability analysis.

3.3 DESIGN SUBSURFACE CONDITIONS

The design subsurface conditions for the factor of safety assessment were developed based on review of the previous investigations and supplemental investigation. The 2016 Haley & Aldrich borings were advanced along the toe of the impoundment. Review of the four borings (MW-BAA-1, 2, 3, and 4) indicated that the toe of the impoundment generally consisted of zero to 7 feet of overburden/clayey sand followed by limestone and shale bedrock. While no geotechnical laboratory testing was provided for these borings, the soil layering was used to develop the critical slope stability section.

The three borings from the 2009 Black & Veatch Report were drilled along the center axis of the berm (Figure 2-2). All three borings indicated that the berm fill consisted of fly ash/bottom ash mix that classified as silty sand. The material within the berm was described as very dense, tan, brown and gray, silty, fine grained sand with a trace of gravel. Design soil properties for this layer were determined based on average values from laboratory testing and are listed in Table 3-2. Note that portions of the berm face are currently armored with a rip-rap surfacing. This material is not accounted for in the slope stability as the effect on the stability is considered negligible.

The three tests pits that were completed as part of the 2016 supplemental investigation indicated that the material used to raise the berm in 2012 was slightly coarser than the original berm fill and had a slightly lower strength. Design soil properties of this material were determined based on the direct shear test results and are listed in Table 3-2.

Based on the 2014 topographic and bathymetric survey, upstream of the berm in the impoundment, bottom ash appears to have settled from the slurry water and has collected along the bottom of the impoundment and upstream toe of the berm. No samples of this material were collected during the 2009 Black & Veatch study. This material was considered to have no strength; only a unit weight was accounted for. Since this material is on the upstream portion of the berm, the material does not affect the stability of the berm.

The berm fill rests directly on a weathered bedrock profile composed of shale and limestone. Based on the three 2009 borings, and the six 2018 borings, the top of the profile is a grayish green shale that is weathered to residual soil. The material properties for the weathered shale were developed based on the results from the 2009 Black & Veatch report. Drained or effective stress strength parameters were developed based on published correlations. The design values for the weathered shale are presented in Table 3-2.

Below the weathered shale, the bedrock is composed of shale and limestone. Unconfined compression testing indicated the strength of the intact rock varied between approximately 400 and 9,000 pounds per square inch. The design properties listed in Table 3-2 were based on the lowest strength and average unit weights from the laboratory testing.

TABLE 3-2
STABILITY ANALYSIS PARAMETERS

LAYER	BERM MATERIAL		BOTTOM ASH	WEATHERED SHALE	BEDROCK
	UPPER	LOWER			
Total Unit Weight	125	125	115	125	140
Total Stress (Undrained) Parameters					
Cohesion (c) (psf)	330	1300	0	2000	30,000
Angle of Internal Friction (ϕ) (degrees)	33	38	0	0	0
Effective Stress (Drained) Parameters					
Cohesion (c’) (psf)	330	1300	0	0	30,000
Angle of Internal Friction (ϕ') (degrees)	33	38	0	28	0

3.4 DESIGN GROUNDWATER CONDITIONS

No long-term field measurements of the pore pressures have been collected within the berm. For the static and seismic analysis, it was assumed that the water elevation on the upstream side would be at the elevation of the current outlet invert (1237.75 feet). Water level measurements were noted on the piezometer logs from the 2009 Investigation. The water depths at B-1 and B-2 were reported at 19.1 feet deep (elevation 1225.9 feet) and 12.1 feet (elevation 1226.9 feet), respectively with an average value of elevation 1226.4 feet. Water levels were measured in the 2018 borings taken within the BASA. Measurements indicated the water levels were 3 to 4 feet below the existing ground surface.

Based on the observation of seepage in the 2015 and 2016 inspections, the phreatic surface within the slope was shown to intercept the downstream face of the berm. According to the 2016 Black & Veatch inspection, the seepage was occurring approximately 50 feet downslope from the berm crest which was approximately elevation 1222 feet. Groundwater elevations measured at the monitoring wells located along the western toe of the berm in April 2016 indicated the groundwater elevation was approximately 1211 to 1212 feet. These groundwater elevations are higher than the lowest surface elevation at the berms toe; therefore, the phreatic surface at the toe of the berm was modelled at the same elevation as the ground surface.

For the maximum surcharge analysis, the water elevation on the upstream side was increased to the same elevation as the top of the berm. The phreatic surface in the berm was also increased the same amount; therefore, these analyses are considered very conservative.

4.0 Safety Factor Assessment

In accordance with the CCR Rule §257.73(e), initial and periodic safety factor assessments are required for CCR impoundments.

The assessments are to be performed for the cross section of the embankment that is anticipated to be the most susceptible to structural failure. Black & Veatch identified the critical cross section based on engineering judgment, the embankment geometry, loading conditions, worst-case phreatic water levels within the embankment cross-section as well as expected subsurface soil conditions. Using the topographic and bathymetric survey data, Black & Veatch analyzed three surface profiles through the berm to identify the critical section (Section 1) shown on (Figure 4-1). Overall, the crest elevation and upstream and downstream berm slopes were consistent; therefore, the critical profile was identified based on the lowest toe elevation, which corresponded to the highest berm.

For modeling the soil and rock layers within the model, the base of the berm was determined by connecting a straight line from the downstream toe of the berm through the bottom of the fill in boring B-2 to the intersection with the ground surface upstream of the berm. The upstream slope of the berm was extended to the line forming the base of the berm. In the cross section the berm consists of the downstream slope, crest, upstream slope and base of the berm between the upstream and downstream slopes. The extension of the base of the berm and the ground surface upstream of and adjacent to the berm forms a small area that is assumed to be filled with bottom ash that has settled from the slurry. The 2018 borings indicated that the CCR was slightly deeper than estimated in the previous analysis. As a result, the upstream geometry was updated. The boring logs along the centerline of the berm indicate that the upper portion of bedrock is weathered shale that is weathered to residual soil. At the centerline, a 10-foot thick layer of residual shale was modelled below the base of the berm. At the downstream toe, the thickness of the weathered shale was reduced based on the observed thickness of soil in the 2016 monitoring well logs. The phreatic surface within the berm for both long-term maximum pool and maximum surcharge cases was modelled as described in Subsection 3.5.

The CCR Rule requires the critical section to be analyzed under the four loading conditions listed in Table 4-1. Each of these loading conditions as well as the results are discussed further in Section 4.2.

TABLE 4-1
CCR RULE SAFETY FACTOR REQUIREMENTS (§257.73(E))

LOADING CONDITION	MINIMUM FACTOR OF SAFETY
Long-term-maximum storage pool	1.50
Maximum surcharge	1.40
Seismic loading	1.00
Soil Liquefaction*	1.20
Note: Soil liquefaction case is only required if soils are identified as having potential for liquefaction under seismic loading.	

4.1 SLOPE STABILITY ANALYSIS

Black & Veatch performed the slope stability analysis using the SLOPE/W computer program that is part of the GeoStudio 2019 analysis software. The SLOPE/W program is a limit equilibrium method that allows for complex soil layering and has the capability of performing optimization of the slip surface.

4.1.1 Long-Term Maximum Storage Loading

The long-term maximum storage loading condition represents the condition with the pool at normal operating condition under steady-state seepage conditions. The minimum factor of safety determined for the Long-term Maximum Storage analysis is 1.56, which is greater than the required factor of safety listed in Table 4-1 (Figure 4-1).

4.1.2 Maximum Surcharge Loading

According to the CCR Rule Preamble Part VI (E)(3)(b)(ii)(c), the maximum surcharge pool loading condition is meant to ensure that the impoundment can withstand a temporary rise in the pool elevation. The berm has an outlet invert pipe elevation of 1237.75 feet. Additionally, a culvert along the north edge of the berm allows water to exit the impoundment prior to the berm over topping. If these systems are both unserviceable, then the low elevation along the top of the berm will control the water elevation, which is 1242 feet. This water elevation will be used to compute the maximum surcharge loading. The CCR rule Preamble notes that this loading condition should consider the condition to occur long enough for steady-state seepage conditions to occur within the embankment; therefore drained, or effective stress soil properties were used for this case. The results for this case indicated a minimum factor of safety of 1.58, which is greater than the required factor of safety listed in Table 4-1 (Figure 4-2).

In addition to the maximum surcharge pool loading case, the CCR Rule Preamble Part VI (E)(3)(b)(i) also addresses the potential for the rapid or sudden drawdown case. The rule clearly states that the conventional rapid drawdown case as is typical for a dam structure is not applicable to CCR impoundments, because at no point would a CCR impoundment be drawn down like a dam. However, a second consideration for this case is discussed specifically for impoundments adjacent to a body of water. The intent of this case is that the adjacent body of water experiences a flood condition and the exterior of the CCR impoundment is inundated by the adjacent body of water. While this condition presents a stabilizing force on the exterior of the impoundment, when the adjacent body of water returns to normal conditions, it may occur rapidly enough that the exterior slopes could remain in a saturated condition. This loading scenario is not possible at the BASA due to the CCR impoundment not being adjacent to streams, ponds or reservoirs that can rise to the point that the downstream slope of the berm is inundated. Therefore, no additional case was analyzed.

4.1.3 Seismic Loading

In addition to slope stability analyses for the embankments under the prescribed static loading conditions, slope stability analyses were also performed for seismic loading conditions as prescribed in the CCR Rule (§257.73 (e)).

As discussed in the CCR Rule Preamble Part III (D)(3)(b)(2) the seismic stability analysis was completed based on the methodologies described in the 2009 Mine Safety and Health Administration (MSHA) Engineering and Design Manual for Coal Refuse Disposal Facilities. Following the MSHA's guidance, a simplified pseudo-static procedure was applicable since the impoundment is not within a seismic impact zone and the embankment and foundation did not contain material that was susceptible to significant strength loss during the design seismic event. The pseudo-static method considers the potential inertial forces due to ground accelerations during an earthquake by the inclusion of a static horizontal force in the limit equilibrium analyses. The static horizontal force is determined based on the weight of the sliding mass and the horizontal seismic coefficient (k_h) which is taken as one-half of the Peak Ground Acceleration (PGA) at the bedrock per Hynes-Griffin & Franklin (1984).

In accordance with the CCR rule, the PGA value was determined based on an earthquake event with a 2% probability of exceedance in 50 years which is equivalent to a return period of approximately 2,475 years. The earthquake conditions were determined based on the U.S. Geological Survey (USGS) National Seismic Hazard Maps which indicated PGA of 0.084 at the bedrock. A k_h value of 0.5*0.084 or 0.042 was used to simulate the horizontal earthquake loading using pseudo-static methods in the limit equilibrium slope stability analyses for the seismic loading condition. The results for this case indicated a minimum factor of safety of 2.82, which is greater than the required factor of safety listed in Table 4-1 (Figure 4-3).

4.1.4 Soil Liquefaction

Based on the CCR Rule, 257.73, soil liquefaction analysis of the embankment and foundation soils were evaluated to determine if the soils are susceptible to liquefaction under the design earthquake. Liquefaction of soils typically occurs in loose, saturated or partially saturated soils that undergo a loss of strength due to the generation of pore pressures during a seismic event.

Based on the borings from the 2009 Black & Veatch inspection and evaluation report, the berm consists of materials that include fly ash/bottom ash fill, weathered bedrock, and bedrock composed of shale and limestone. The bottom ash/fly ash fill was compacted in 1 to 2-foot lifts during berm construction. This material is very dense silty sand with N-values greater than 50. Thus, the berm is not susceptible to liquefaction. The berm sits on weathered shale and limestone that is not considered susceptible to liquefaction.

5.0 References

Black & Veatch (2009) Bottom Ash Settling Berm Inspection and Engineering Evaluation Report, December 2009.

Haley & Aldrich (2016), report on Initial Annual CCR Surface Impoundment PE Inspection Bottom Ash Area 1 Impoundment, File number 41938.006, January 2016.

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Hynes-Griffin, Mary E. and Franklin Arley G., (1984), Rationalizing the Seismic Coefficient Method, Department of the Navy, Miscellaneous Paper GL-84-13, July 1984.

U.S. Environmental Protection Agency (2015), Hazardous Solid Waste Management System; Disposal of Coal Combustion Residuals From Electric Utilities; Federal Register Volume 80, No. 74 40 CFR Parts 257 and 261, April 17, 2015.

Figures

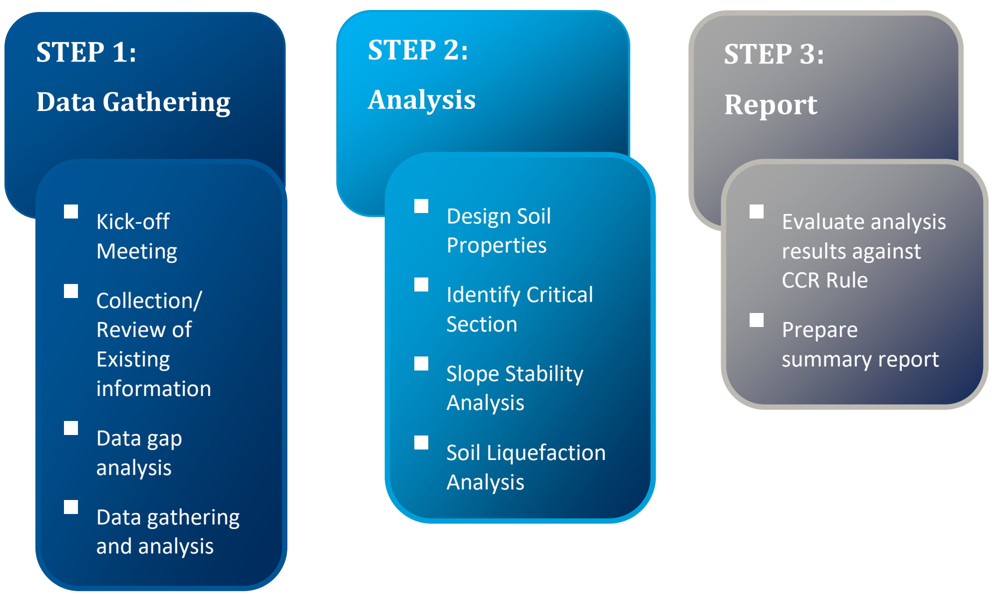

Figure 1-1 Safety Factor Assessment Process Outline

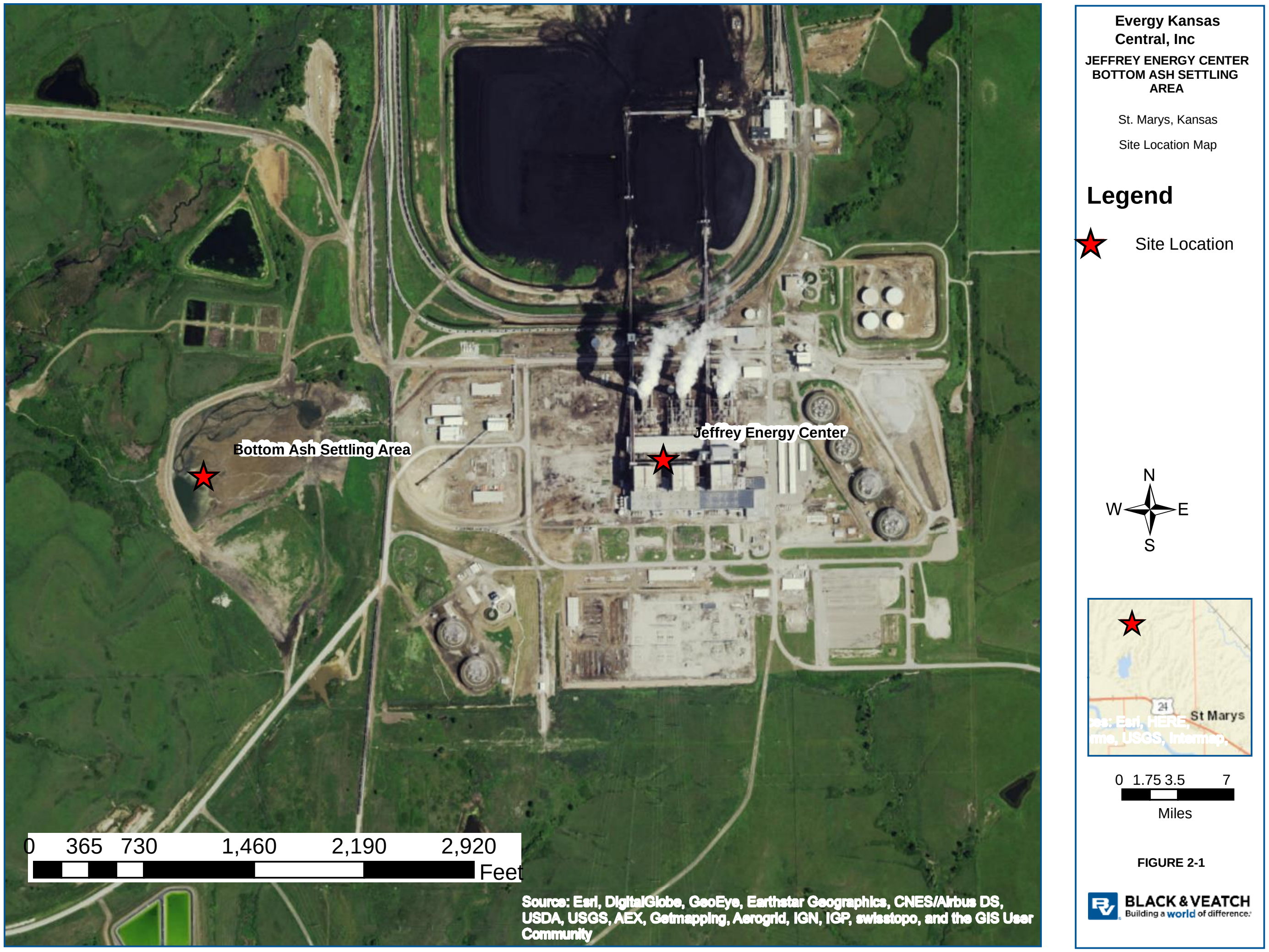

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Figure 2-1 Site Location

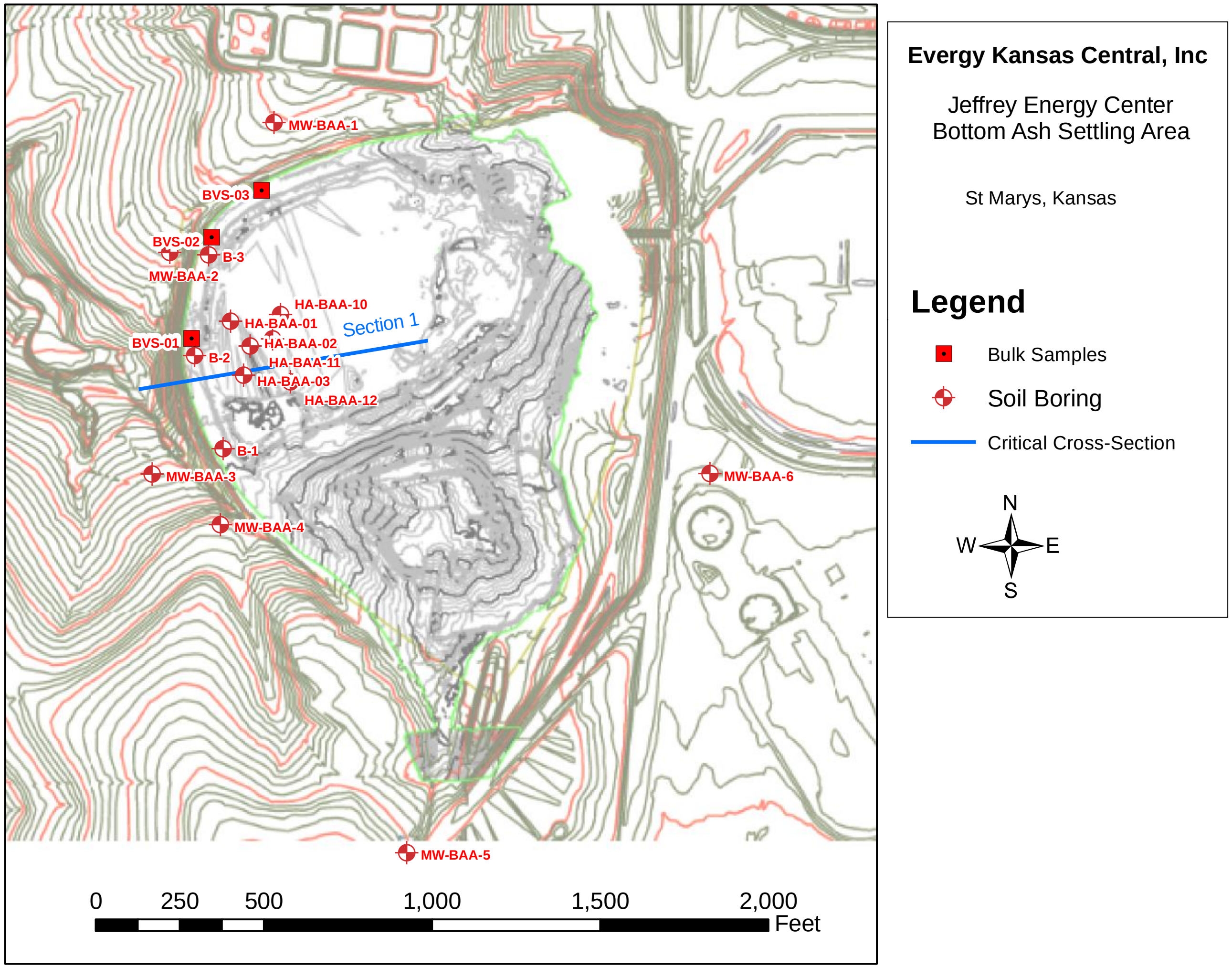

Figure 2-2 Subsurface Investigations

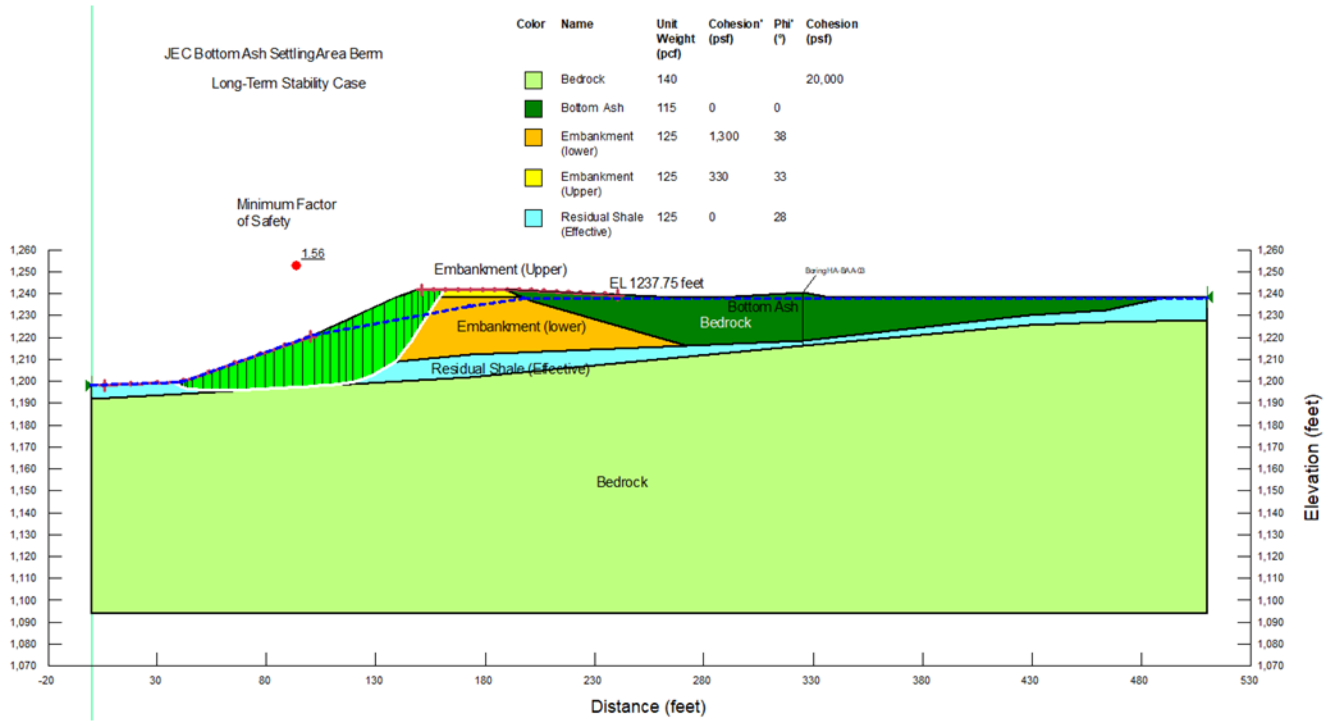

Figure 4-1 Slope Stability Results – Long-Term Stability Case

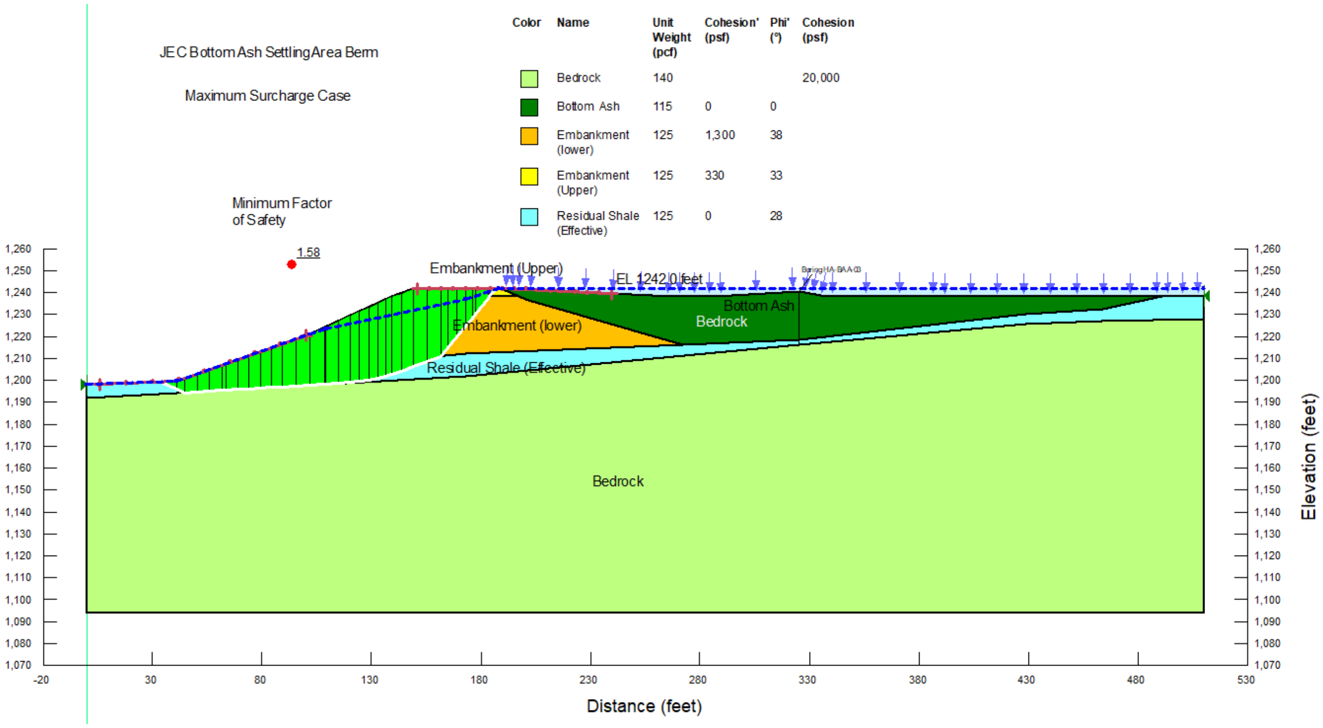

Figure 4-2 Slope Stability Results – Maximum Surcharge Case

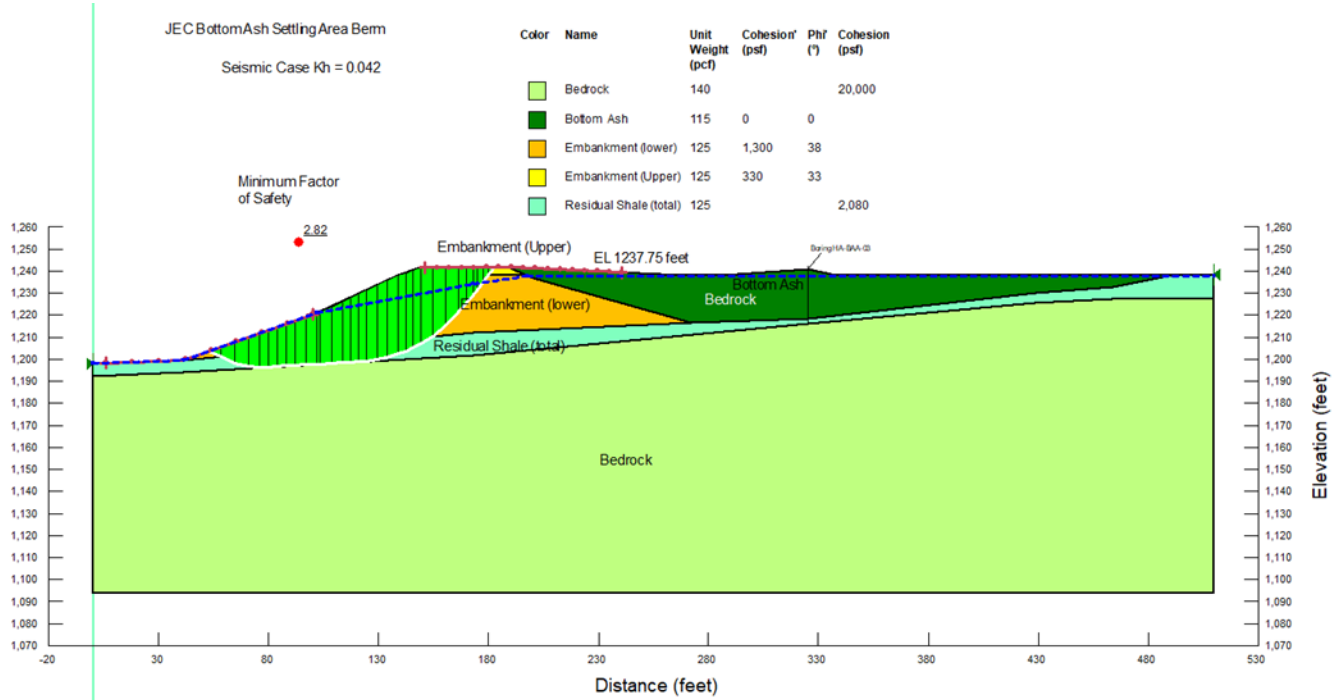

Figure 4-3 Slope Stability Results – Seismic Case