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October 17, 2016

Mr. Gordon Criswell
Talen Montana—Environmental & Engineering Compliance Dept.
P.O. Box 38
Colstrip, MT 59323

RE: INITIAL SAFETY FACTOR ASSESSMENT REPORT, UNITS 1 & 2 SURFACE IMPOUNDMENTS, COLSTRIP STEAM ELECTRIC STATION, COLSTRIP, MONTANA
PROJECT NO: 16419

Dear Mr. Criswell:

As requested by Talen Montana, the attached report summarizes the initial safety factor assessments performed for surface impoundments of Units 1 & 2 of the Colstrip Steam Electric Station (CSES) in Colstrip, Montana. We have prepared this report to comply with new coal combustion residual (CCR) regulations published in the Federal Register on April 17, 2015, specifically to Title 40 CFR §257.73(e).

Safety factor assessments were performed on critical cross-sections of embankments surrounding surface impoundments at the Units 1 & 2 Second Stage Evaporation Ponds (STEP) and the Units 1 & 2 Bottom Ash Pond. Calculated factors of safety for these embankments achieve the required safety factors specified by §257.73(e)(1)(i) through (iv) and indicate stability. Engineering services relevant to the annual inspection and monitoring were conducted by or under the direct supervision of a Montana registered Professional Engineer.

If you have any questions about this report, or if we may provide other services to you, please contact us.

Respectfully submitted,
JORGENSEN GEOTECHNICAL, LLC

Colter H. Lane, E.I., M.S.

Ray Womack, P.E., P.G.

INITIAL SAFETY FACTOR ASSESSMENTS
COLSTRIP STEAM ELECTRIC STATION UNITS 1 & 2
COLSTRIP, MONTANA

Prepared for:

Mr. Gordon Criswell
Talen Montana
Environmental & Engineering Compliance Dept.
P.O. Box 38
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Prepared by:

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October 17, 2016

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1.0 INTRODUCTION AND CERTIFICATION

Regulations addressing disposal of the Coal Combustion Residuals (CCR) from electric utilities (Title 40 of the Code of Federal Regulations, Part 257, Subpart D) were published in the federal register on April 17, 2015 and became effective on October 19, 2015. Section 257.73(e)(1) requires the owner or operator to conduct safety factor assessments on surface impoundments containing CCR material to document whether calculated factors of safety achieve the minimum stability safety factors for several loading conditions. Loading conditions and required safety factors are shown in Table 1-1. These loading conditions are to be applied to the critical cross-section(s) of each embankment, where the critical cross-section is defined as the cross-section most susceptible of all cross-sections to structural failure based on appropriate engineering considerations.

Table 1-1: Safety Factor Requirements Summary

Loading Condition	Described in Section	Required Safety Factor
Static, Long-term, Maximum Storage Pool	§274.73(e)(1)(i)	1.50
Static, Maximum Surcharge Pool	§274.73(e)(1)(ii)	1.40
Seismic	§274.73(e)(1)(iii)	1.00
Liquefaction	§274.73(e)(1)(iv)	1.20

The Colstrip Steam Electric Station (CSES) in Colstrip, Montana deposits and stores CCR produced by Units 1 & 2 in surface impoundments at two main areas: the Units 1 & 2 Second Stage Evaporation Ponds (STEP) and the Units 1 & 2 Bottom Ash Pond. This report summarizes the findings of the initial safety factor assessment of surface impoundments in both areas. Calculated factors of safety for embankments and dikes surrounding CCR surface impoundments of the CSES Units 1 & 2 exceed the required safety factors summarized above and indicate stability under the required loading conditions. Results of the safety factor assessments are presented in Section 5.0.

I, Wesley Raymond Womack, a registered Professional Engineer in the State of Montana (License No. 8312PE), certify that the ***Initial Safety Factor Assessments*** performed for surface impoundments of the Colstrip Steam Electric Station Units 1 & 2 meet the requirements of ***§257.73(e)(1) Periodic safety factor assessments***. This certification is made to comply with the specific requirement of §257.73(e)(2).

Jorgensen Geotechnical, LLC

Ray Womack 10/17/2016

MONTANA
WESLEY R. WOMACK
8312PE
LICENSED
PROFESSIONAL ENGINEER

Ray Womack, P.E., P.G.
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2.0 REVIEW OF PAST STABILITY ANALYSES

Since Bechtel's original design (Bechtel, 1979), numerous stability analyses have been performed on the CSES Units 1 & 2 surface impoundments by Womack & Associates (WAI) and Jorgensen Geotechnical (JG). These reports provide valuable information regarding the internal and external geometry and material parameters of the facility's embankments.

JG reviewed the following reports and data sources for input into the initial safety factor assessment of the Units 1 & 2 surface impoundments:

- Bechtel, 1979. "Second Stage Evaporation Pond Design Report." Prepared by Bechtel Power Corporation, December 1979.
- WAI, 2010a. "Units 1 & 2 Stage Two Evaporation Pond (STEP) Dam – Geotechnical Investigations Report for the EPA Recommended Corrective Measures at the Colstrip Power Plant." Prepared by Womack & Associates, Inc., January 8, 2010.
- WAI, 2010b. "Units 1 & 2 Pond "A" Waste Impoundment Embankment – Geotechnical Investigations & Analyses Report for the EPA Recommended Corrective Measures at the Colstrip Power Plant." Prepared by Womack & Associates, Inc., January 14, 2010.
- WAI, 2010c. "Units 1 & 2 Bottom Ash Waste Impoundment Pond – Geotechnical Investigations & Analyses Report for the EPA Recommended Corrective Measures at the Colstrip Power Plan." Prepared by Womack & Associates, Inc., January 22, 2010.
- WAI, 2010d. "Units 1 & 2 STEP Dam – Geotechnical Investigation & Analyses Memo for the Units 1 & 2 STEP Dam Inspection Issue, Prepared by Womack & Associates, Inc., March 22, 2010 [Additional piezometers to detect seepage in embankment shell].
- Jorgensen, 2016a. "Geotechnical Investigation and Embankment Stability Report—Revision 1." Prepared by Jorgensen Geotechnical, LLC, March 3, 2016.

Additional references are listed in Section 8.0.

3.0 SLOPE STABILITY METHODOLOGY

Safety factors for the loading conditions described in §274.73(e)(1)(i) through (iii) may be produced with two-dimensional limit equilibrium stability modeling. Slope stability analyses described in this report were performed using GEO-SLOPE International's SLOPE/W limited equilibrium program (GeoStudio 2012, V8.15). Reports produced by SLOPE/W of the settings, model and slip surface geometry, and calculated strengths applied to slices within the critical slip surfaces are attached in Appendix B. Slope stability models were developed and analyses were performed using the following methodology:

3.1 Analyses

The Morgenstern-Price limit equilibrium method, which considers both moment and force equilibrium, was used to compute structural stability factors of safety for each cross-section. Limit equilibrium analyses do not indicate complex failure mechanisms nor do these sites require computation of displacements; specialized analytical methods are not necessary.

According to the requirements of §257.73(e), stability factors of safety are to be calculated for the following loading conditions:

1. **Static Factor of Safety: Long-Term, Maximum Storage Pool - §274.73(e)(1)(i)**

The maximum storage pool loading is the maximum water level that can be maintained that will result in the full development of steady-state seepage. As summarized in Table 3-1, the water level elevations for the modeled surface impoundments are the maximum storage pool under normal operations. Calculated factors of safety for this loading condition are summarized in Section 5.1.

Table 3-1: Maximum Storage Pool

SURFACE IMPOUNDMENT	WATER LEVEL ELEVATION
STEP B-Cell	3,267-ft
STEP E-Cell	3,267-ft
STEP Clearwell	3,267-ft
STEP D-Cell	3,267-ft
1&2 Bottom Ash Pond	3,260-ft

In the case of several embankments at the STEP, the embankment serves as a divider dike between CCR surface impoundments. Water in a surface impoundment acts as load resisting failure on the inboard face of the embankment. The most conservative evaluation of the embankment face then is to assume the pond is empty. As such, in order to assess critical conditions, safety factors were calculated assuming the surface impoundment on the downstream side of the analyzed embankment face is dry.

2. Static Factor of Safety: Maximum Surcharge Pool - §274.73(e)(1)(ii)

The maximum surcharge pool is considered a temporary water surface elevation that is higher than the maximum storage pool. This represents a condition in which the CCR surface impoundment is, for instance, passing a design flood surcharge and is considered temporary. Therefore, this condition has a lower required factor of safety (FS ≥ 1.40). Water levels used in the models are summarized in Table 3-2. Calculated factors of safety for this loading condition are summarized in Section 5.2.

Table 3-2: Maximum Surcharge Pool Elevations

SURFACE IMPOUNDMENT	WATER LEVEL ELEVATION
STEP B-Cell	3,270-ft
STEP E-Cell	3,270-ft
STEP Clearwell	3,270-ft
STEP D-Cell	3,274-ft
1&2 Bottom Ash Pond	3,262.5-ft

The results of the analysis indicate there is no influence of the elevation of the impounded water on the stability of the downstream face of the embankment. Critical slip surfaces are not impacted by surcharge from the impounded water and calculated factors of safety for the modeled Maximum Surcharge Pool condition (Table 5-2) are the same as those for the Maximum Storage Pool condition (Table 5-1) for each cross-section.

3. Seismic Factor of Safety - §274.73(e)(1)(iii)

All embankments surrounding CCR surface impounds must be able to withstand a design earthquake without damage to the embankment or to the foundation that would cause the impoundment to discharge its contents. Seismic loading conditions have been calculated using a peak ground acceleration (PGA) with a 2% probability of exceedance in 50 years, equivalent to a return period of about 2,500 years.

Seismic factors of safety have been evaluated using a pseudo-static approach where inertial forces from seismic accelerations are applied statically to the model. These forces are assumed to be proportional to the weight of the sliding mass times a horizontal seismic coefficient k_h. A seismic coefficient of k_h = ½PGA has been used in this assessment with a 20% reduction in the shear strength of soil materials (Hynes-Griffin and Franklin, 1984). Seismic loads have been applied to the critical slip surface determined by static analysis for each cross-section as is it assumed to be the most stressed region within the slope (Abramson et al., 2002). Factors of safety for this loading condition are summarized in Section 5.3.

3.2 Geometry

Internal and external geometry have been taken from previous stability analyses performed by this office and others (see Section 2.0). When necessary, external geometry was updated from the most recent topographic data provided by Talen Montana. Cross-sections were chosen as 1) what appear to be the most critical section based on appropriate engineering considerations and 2) where the most data were available (i.e., sections through areas with subsurface exploration data). Cross-section locations are shown on Figure 1 and Figure 2. Figures showing model geometry are in Appendix A.

Slip surfaces are generated within the model using entry and exit specification. Circular slip surfaces were selected, as they are found to be the most critical in homogenous slopes. Many of the embankments are keyed into the underlying foundation soil or rock and foundation materials are too strong to be susceptible to translational failure. Entry and exit zones on the ground surface were selected using engineering judgement based on where critical slip surfaces are anticipated to daylight and we verified that minimum factors of safety are located within these zones. Critical slip surfaces of each analysis are indicated on the figures of Appendix A.

Figure 1: Units 1 & 2 STEP Cross-Section Location Map

Figure 2: Units 1 & 2 Bottom Ash Pond Cross-Section Location Map

3.3 Material Parameters

Properties of embankment, foundation, and CCR materials were characterized using a Mohr-Coulomb strength model and are summarized in Table 3-3 and Table 3-4. The site-specific field and laboratory results compare well with Bechtel's original design parameters of embankment and foundation soils and Bechtel's shear strength parameters were used for modeling these materials (Bechtel, 1979). Strength parameters of paste materials were adopted from laboratory testing performed by Golder Associates and Womack & Associates (Golder, 2001).

Table 3-3: Material Strength Parameters – Units 1 & 2 STEP Area

MATERIAL	UNIT WEIGHT		EFFECTIVE FRICTION ANGLE, Φ'	EFFECTIVE COHESION, C'
	MOIST (PCF)	SAT. (PCF)		
EMBANKMENT FILL*	124.5	130	33°	50 psf
CLAY CORE*	127	-	33.5°	0 psf
DRAIN*	130	135	35°	0 psf
FOUNDATION SOIL*	115	125	32° (27°‡)	0 psf
PASTE†	-	112	34.4°	0 psf
BEDROCK	IMPENETRABLE			

* Bechtel, 1979

† Golder, 2001

‡ Foundation material under the south embankment of D-Cell assigned ϕ'=27° based on recent direct shear testing (Jorgensen, 2016a)

\- Denotes material parameter not applied in models

Effective stress parameters are used as required loading conditions are long-term and excess pore pressures are not anticipated. Cyclic loading from seismic accelerations may cause a reduction in soil shear strength and soil strength has been reduced by 20% in pseudo-static analyses (Abramson et al, 2002).

A small amount of cohesion (c' = 50 psf) was assumed for embankment fill. The critical slip surface (lowest factor of safety) generated using a cohesionless material will consistently approach an infinite slope condition (i.e., representing only ravel or very thin failure surfaces). A little cohesion drives the slip surfaces deeper into the embankment material to represent more reasonable (larger and more dangerous) anticipated failure mechanisms. In fact, unsaturated fine-grained soils will exhibit "apparent cohesion" due to soil suction (negative pore pressures) and the small amount of cohesive strength added to the model is not unreasonable.

Bedrock is modeled as "Impenetrable". Slip surfaces encountering the edge of bedrock material follow the surface of the bedrock. Slices with bases on bedrock assume base shear strength (i.e., resistance) based on the shear strength parameters of the material immediately above the bedrock.

In effective stress analyses, materials are assigned total unit weights and pore water pressures are accounted for using internal pressures calculated from a piezometric line. Total unit weights are generally equal to moist unit weights in the analyses performed for this assessment due to the presence of liners and the lack of seepage detected by the vibrating wire piezometers. Saturated unit weights are used for CCR waste materials stored inboard of cell liners within the surface impoundments (e.g., paste), foundation material under the drain on the downstream side of the STEP Main Dam, and soils below observed groundwater surface levels at the Units 1 & 2 Bottom Ash Pond (JG, 2016, Initial Annual Inspection Report).

Table 3-4: Material Strength Parameters – Units 1 & 2 Bottom Ash Pond

MATERIAL	UNIT WEIGHT		EFFECTIVE FRICTION ANGLE, Φ' (DEG)	EFFECTIVE COHESION, C' (PSF)
	MOIST (PCF)	SAT. (PCF)		
EMBANKMENT FILL* Section A-A' (Figure 2)	127	146.4	30.9	372.9
EMBANKMENT FILL** Section B-B' (Figure 2)	120	126.8	31.5	204.2
CLAY CORE†	127	140.4	33.5	0
FOUNDATION SOIL**	118.2	123.8	29.2	107.8
BEDROCK	IMPENETRABLE			

* Parameters based on laboratory testing (WAI, 2010c)

** Parameters based on laboratory testing (WAI, 2010b)

† Bechtel, 1979.

Material parameters for Units 1 & 2 Bottom Ash ponds differ somewhat from material models at the Units 1 & 2 STEP. Embankment fill material was sampled and tested by Womack & Associates as a part of the two "EPA Recommended Corrective Measures at the Colstrip Power Plant" dated January 2010 (see Section 2.0). As at the STEP, core material proved too stiff to sample and material parameters from Bechtel's STEP Design report (1979) were used.

3.4 Phreatic Surface

All cells analyzed have been constructed with liners (Geosyntec, 2016a) and vibrating wire piezometers installed in each embankment have not detected seepage within embankment materials (Jorgensen, 2016b). Therefore, a phreatic surface has not been applied. The single exception to this rule is the foundation material underlying Section B-B' at the Units 1 & 2 Bottom Ash Pond where a piezometric surface line was drawn at an elevation of 3,225 feet to model the influence of groundwater detected by VW piezometer PONDA-09-3P (Jorgensen, 2016b).

Impounded water is modeled as a surcharge load of 62.4 pcf applied “normal” to the liner surface of the pond. Water surfaces within the surface impoundments are modeled according to the elevations discussed in Sections 3.1.1 and 3.1.2 (Table 3-1 and Table 3-2).

Critical slip surfaces generated in the stability models do not encounter saturated materials or piezometric lines. In general, seepage pressures do not affect the stability models of the Units 1 & 2 facilities.

3.5 Seismicity

CSES facilities are in an area of low seismic activity and predicted accelerations are relatively low. Online tools exist to select a site specific PGA for the CSES facilities (USGS Seismic Design Maps Application, 2014). These are based on USGS seismic hazard maps published in 2008, which form the basis of seismic loads for the ASCE 7-10 Minimum Design Loads for Buildings and Other Structures. The CSES facility (approximate Latitude = 45.9^{o} N and Longitude = 106.6^{o} W) has a site specific PGA with 2% probability of exceedance in 50-years of 0.047g, according to Figure 22-7 of the ASCE 7-10.

The USGS seismic hazard maps were updated in 2014 to account for new methods, models, and data that have been obtained since the 2008 maps were released. According to Figure 7 of Petersen, et al. (2014), PGA values for Colstrip have increased by 0.01g to 0.05g on the updated maps. Accordingly, seismicity is conservatively assessed in the stability models using a PGA = 0.06g and k_h = 0.03g.

4.0 LIQUEFACTION EVALUATION

A liquefaction evaluation is required by §274.73(e)(1)(iv) if dikes are constructed of soils susceptible to liquefaction. In general, liquefaction requires three things: 1) loose, cohesionless soils, 2) saturated conditions, and 3) high enough seismicity to drive ground shaking and increase pore water pressures in soil materials.

Conditions of the embankments of the Units 1 & 2 surface impoundments are as follows:

1. Materials: Embankment soil (i.e., shell and core materials) is too stiff and fine-grained to be susceptible to liquefaction. Foundation materials underlying the embankments also have too many fines to be liquefiable. SPT blow counts observed at Units 1 & 2 embankments are too high to predict liquefaction at this site.
2. Saturation: Since seepage has not been detected within embankment materials by piezometers, saturated conditions are only expected below the depth of the instrumentation resulting in relatively small differences in the soil's total stress (σ_v) and effective stress (σ_v'), which is an important component of the soil's cyclic stress ratio (CSR) in current liquefaction evaluation methods (Boulanger and Idriss, 2014, Idriss and Boulanger, 2008).
3. Seismicity: The PGA with a probability of exceedance of 2% in 50 years is conservatively estimated for embankment analysis at CSES facilities as 0.06g. Low accelerations yield low values of CSR and are not expected to produce liquefaction.

Therefore, embankments and dikes constructed at the Units 1 & 2 STEP and Bottom Ash ponds are not constructed with soils that are susceptible to liquefaction and factors of safety against liquefaction have not been calculated.

5.0 SAFETY FACTOR ASSESSMENT RESULTS SUMMARY

The results of stability analyses are summarized in Table 5-1, Table 5-2, and Table 5-3. Cross-section figures from the slope stability models are in Appendix A.

5.1 Results of Loading Condition: Static, Long-Term, Maximum Storage Pool

Calculated factors of safety for this loading condition must equal or exceed 1.50 per §274.73(e)(1)(i). Stability analysis results of each cross-section indicate factors of safety that exceed the requirements.

Table 5-1: Results Summary – Static, Maximum Storage Pool

<table>
<tr><th>Embankment</th><th>Stability Section*</th><th>Direction</th><th>Calculated Factor of Safety</th></tr>
<tr><td>E/C Divider Dike</td><td>A-A’</td><td>Downstream (North)</td><td>1.56</td></tr>
<tr><td rowspan="4">E/B Divider Dike</td><td rowspan="2">B-B’</td><td>B-Cell (NW)</td><td>2.42</td></tr>
<tr><td>E-Cell (SE)</td><td>1.78</td></tr>
<tr><td rowspan="2">C-C’</td><td>B-Cell (NW)</td><td>2.41</td></tr>
<tr><td>E-Cell (SE)</td><td>2.19</td></tr>
<tr><td rowspan="2">E/D Divider Dike</td><td rowspan="2">D-D’</td><td>D-Cell (South)</td><td>2.26</td></tr>
<tr><td>E-Cell (North)</td><td>4.37</td></tr>
<tr><td rowspan="2">CW/D Divider Dike</td><td rowspan="2">E-E’</td><td>D-Cell (South)</td><td>2.27</td></tr>
<tr><td>Clearwell (North)</td><td>1.76</td></tr>
<tr><td>D-Cell: South Embankment</td><td>F-F’</td><td>Downstream (South)</td><td>2.09</td></tr>
<tr><td>STEP Main Dam</td><td>G-G’</td><td>Downstream (East)</td><td>2.11</td></tr>
<tr><td rowspan="2">1 & 2 Bottom Ash Pond</td><td>A-A’</td><td>Downstream (East)</td><td>2.41</td></tr>
<tr><td>B-B’</td><td>Downstream (NW)</td><td>1.99</td></tr>
</table>

* Cross-sections for STEP embankments are shown on Figure 1.
Units 1 & 2 Bottom Ash Pond cross sections are shown on Figure 2.

5.2 Results of Loading Condition: Static, Maximum Surcharge Pool

Calculated factors of safety for this loading condition must equal or exceed 1.40 per §274.73(e)(1)(ii). Critical slip surfaces generated by the stability models are not influenced by changes in loading due to higher water surface elevations within the surface impoundments and calculated factors of safety for this loading condition are same as for that of the Maximum Storage Pool loading condition (see Table 5-1). Refer to the discussion in Section 3.1.2. Stability analysis results of each cross-section indicate factors of safety that exceed the requirements.

Table 5-2: Results Summary – Static, Maximum Surcharge Pool

Embankment	Stability Section*	Direction	Calculated Factor of Safety
E/C Divider Dike	A-A’	Downstream (North)	1.56
E/B Divider Dike	B-B’	B-Cell (NW)	2.42
		E-Cell (SE)	1.78
	C-C’	B-Cell (NW)	2.41
		E-Cell (SE)	2.19
E/D Divider Dike	D-D’	D-Cell (South)	2.26
		E-Cell (North)	4.37
CW/D Divider Dike	E-E’	D-Cell (South)	2.27
		Clearwell (North)	1.76
D-Cell: South Embankment	F-F’	Downstream (South)	2.09
STEP Main Dam	G-G’	Downstream (East)	2.11
1 & 2 Bottom Ash Pond	A-A’	Downstream (East)	2.41
	B-B’	Downstream (NW)	1.99

* Cross-sections for STEP embankments are shown on Figure 1.
Units 1 & 2 Bottom Ash Pond cross sections are shown on Figure 2.

5.3 Results of Loading Condition: Seismic, Maximum Storage Pool

Calculated factors of safety for this loading condition must equal or exceed 1.00 per §274.73(e)(1)(iii). Stability analysis results of each cross-section indicate factors of safety that exceed the requirements.

Table 5-3: Results Summary – Seismic, Maximum Storage Pool

Embankment	Stability Section*	Direction	Calculated Factor of Safety
E/C Divider Dike	A-A’	Downstream (North)	1.11
E/B Divider Dike	B-B’	B-Cell (NW)	1.71
		E-Cell (SE)	1.28
	C-C’	B-Cell (NW)	1.68
		E-Cell (SE)	1.55
E/D Divider Dike	D-D’	D-Cell (South)	2.05
		E-Cell (North)	3.80
CW/D Divider Dike	E-E’	D-Cell (South)	1.58
		Clearwell (North)	1.25
D-Cell: South Embankment	F-F’	Downstream (South)	1.49
STEP Main Dam	G-G’	Downstream (East)	1.47
1 & 2 Bottom Ash Pond	A-A’	Downstream (East)	1.74
	B-B’	Downstream (NW)	1.43

* Cross-sections for STEP embankments are shown on Figure 1.
Units 1 & 2 Bottom Ash Pond cross sections are shown on Figure 2.

5.4 Loading Condition: Liquefaction

Liquefaction requirements are described in §274.73(e)(1)(iv). It has been determined that embankments of the Units 1 & 2 surface impoundments are constructed of soils not susceptible to liquefaction (see discussion in Section 4.0). Soils are not anticipated to liquefy in a seismic event and factors of safety have not been calculated.

6.0 CONCLUSIONS

In general, embankment dams surrounding the Units 1 & 2 surface impoundments were designed and constructed using conservative approaches to stability. In particular, the embankment slopes are not steep and the highest embankment (STEP Main Dam) employed zoned construction with drains and filters to prevent piping. Placement of fill appears to have been carefully controlled. Embankments evaluated in this report are adjacent to surface impoundments with membrane liners and seepage has not been observed. Therefore, embankments are expected to be stable and their performance has, in fact, been good.

The stability analyses indicate that the analyzed embankments are stable under existing soil shear strength and soil moisture conditions. Calculated factors of safety exceed the minimums required by §257.73(e)(1) of Title 40 of the Code of Federal Regulations, Part 257, Subpart D.

7.0 LIMITATIONS

This report has been prepared based the data available, which includes, but is not limited to, borehole and test pit logs recorded by this office and others, piezometric data collected by this office and others, and topographic mapping data provided to us by others. Data collected by others has generally been relied upon without independent verification of accuracy. Although the database of information for the Colstrip Steam Electric Station is very large and has been found to be reliable, there is inherent uncertainty in engineering analyses based on subsurface data. In addition, subsurface conditions may be affected as a result of plant operations or construction. Should subsurface conditions be different than those assumed for the analyses described in this report, whether through the addition of data or by changing conditions, this office must be notified immediately in order to revise our analyses.

These services have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar conditions. No other warranty is made or implied.

8.0 ADDITIONAL REFERENCES

See Section 2.0 for a list of references related to stability analysis.

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APPENDIX A

Stability Model Cross-Section Figures

APPENDIX B

Slope Stability Analyses Reports