JORGENSEN
GEOTECHNICAL, LLC

PO Box 9550 · 1315 HWY 89 S., Suite 201
Jackson, WY 83002
PH: 307.733.5150
| www.jorgeng.com

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 3 & 4 EHP 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 Units 3 & 4 surface impoundment 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 3 & 4 Effluent Holding Pond (EHP). 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 Ray Womack

Colter H. Lane, E.I., M.S. Ray Womack, P.E., P.G.

INITIAL SAFETY FACTOR ASSESSMENTS
COLSTRIP STEAM ELECTRIC STATION UNITS 3 & 4
COLSTRIP, MONTANA

Prepared for:

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

Prepared by:

JORGENSEN
GEOTECHNICAL, LLC

PO Box 9550
Jackson, WY 83002

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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 3 & 4 in surface impoundments in an area called the Units 3 & 4 Effluent Holding Pond (EHP) located approximately 3.5 miles southeast of the plant. This report summarizes the findings of the initial safety factor assessment of surface impoundments at the EHP. Calculated factors of safety for embankments and dikes surrounding CCR surface impoundments of the CSES Units 3 & 4 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 3 & 4 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

WESLEY R. WOMACK
8312PE

Ray Womack, P.E., P.G.
Montana P.E. License No. 8312PE

2.0 REVIEW OF PAST STABILITY ANALYSES

Since Bechtel's original design (Bechtel, 1982) numerous stability analyses have been performed on the CSES Units 3 & 4 surface impoundments by Hydrometrics, 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 related to slope stability modeling for input into the initial safety factor assessment of the Units 3 & 4 surface impoundments:

- Bechtel, 1979. "Second Stage Evaporation Pond Design Report." Prepared by Bechtel Power Corporation, December 1979.
- Bechtel, 1982. "Effluent Holding Pond Design Report." Prepared by Bechtel Power Corporation, October 1982.
- Hydrometrics, 2000. "Units 3 & 4 EHP Saddle Dam – Saddle Dam Geotechnical Report." Prepared by Hydrometrics, Inc., July 2000.
- Hydrometrics, 2001. "Units 3 & 4 Saddle Dam – Remedial Measures, Preliminary Design Report." Prepared by Hydrometrics, Inc., February 2001.
- WAI, 2007a. "Units 3 & 4 Clearwell [B-Cell] – Geotechnical Investigation Report." Prepared by Womack & Associates, Inc., September 2007.
- WAI, 2007b. "C/G Cell Dike Improvements, Memorandum." Prepared by Womack & Associates, Inc., October 2007.
- WAI, 2009a. "Units 3 & 4 – C/CW [*C/J*] Dike Piezometers and Slope Stability." Prepared by Womack & Associates, Inc., May 2009.
- WAI, 2009b. "Units 3 & 4 Main Dam – Memorandum of 3/4 EHP Main Dam Observations and Stability Review Update." Prepared by Womack & Associates, Inc., May 2009.
- WAI, 2009c. "C-Cell – Old Clearwell (C/CW) [*C/J*] Divider Dike Buttress Slope Stability." Prepared by Womack & Associates, Inc., August 2009.
- WAI, 2009d. "Units 3 & 4 Saddle Dam – Geotechnical Investigation Report for EPA Recommended Corrective Measures at the Colstrip Power Plant." Prepared by Womack & Associates, Inc., December 2009.
- WAI, 2010. "Units 3 & 4 Main Dam – Geotechnical Investigation Report for EPA Recommended Corrective Measures at the Colstrip Power Plant." Prepared by Womack & Associates, Inc., February 2010.
- WAI, 2011a. "Geotechnical Investigation Report – CP 102 EHP Dam Raise Project, Stage 2 Dam Raise – Inboard Embankment Fill." Prepared by Womack & Associates, Inc., January 2011.
- WAI, 2011b. "Units 3 & 4 C-Cell Divider Dike – Slope Stability Report" Prepared by Womack & Associates, Inc., July 2011.
- WAI, 2014. "C-Cell Divider Dike Stability Assessment—Units 3 & 4 EHP." Prepared by Womack & Associates, Inc., December 2014.
- Jorgensen, 2016a. "Geotechnical Investigation and Embankment Stability Report—Revision 1." Prepared by Jorgensen Geotechnical, LLC, March 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. For cells of the Units 3 & 4 EHP impounding water, modeled water level elevations are the maximum storage pool under normal operations, as summarized in Table 3-1. Calculated factors of safety for this loading condition are summarized in Section 5.1.

Table 3-1: Maximum Storage Pool Elevations

SURFACE IMPOUNDMENT	WATER LEVEL ELEVATION
H-Cell	3,289-ft
F-Cell	3,287-ft
B-Cell	3,287-ft

J-Cell was lined during the summer of 2016 and will receive CCR material as paste. The Maximum Storage Pool loading case has been applied to the Main and Saddle Dams bordering J-Cell when analyzing the stability of the outboard face. Paste in J-Cell is modeled at an elevation of 3,285-ft. Similarly, G-Cell will be lined in the future and receive CCR material utilizing dry storage methods. The Maximum Storage Pool loading case was examined for cross-sections of the Saddle Dam bordering G-Cell when the stability of the outboard face is considered. Paste is modeled at an elevation of 3,283-ft within G-Cell.

Many of the embankments bordering J-Cell and G-Cell were modeled assuming existing conditions as this resulted in a more conservative case. Stability factors of safety of the embankment face in the direction of either J-Cell or G-Cell are currently the lowest they will be. Deposition of CCR material increases factors of safety over time by adding resisting load to the face of the embankment. Factors of safety for existing conditions are calculated for cross-sections A-A’, B-B’, C-C’, D-D’, E-E’, L-L’, M-M’, N-N’, O-O’, and P-P’ (see Table 5-2).

In the case of several embankments at the EHP, the embankment serves as a divider dike between 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. This is the case in cross-sections I-I’, J-J’, K-K’, and Q-Q’.

CCR material is deposited as paste into C-Cell. Water impounded in C-Cell comes from precipitation, water left from the evaporators, and water decanting from the paste and plant operations are currently limiting the volume of water in the cell. In the six months between April and October 2016, C-Cell water surface elevations ranged from 3,267.7-ft and 3,273.4-ft and, due to paste elevations within the cell, is generally contained within the southwest quadrant of the surface impoundment. Paste is assumed to be limiting loss of water from C-Cell by seepage into other portions of the EHP. In particular, a stand pipe piezometer in the C-J Dike (JC-15-06 SP) does not indicated seepage through the embankment. Thus, C-Cell water levels are not incorporated into the stability modeling.

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 loading 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
H-Cell	3,292-ft
F-Cell	3,290-ft
B-Cell	3,290-ft
J-Cell	3,288-ft*
G-Cell	3,286-ft*

* Maximum Surcharge water levels are modeled in J-Cell and G-Cell as a surcharge load above paste elevations of 3,285-ft and 3,283-ft, respectively.

As described above, water levels in C-Cell are not incorporated into the models. Thus, Maximum Surcharge Pool analyses of cross-sections J-J' and K-K' in the direction of B-Cell, L-L' and M-M' in the direction of J-Cell, and O-O' and P-P' in the direction of G-Cell are identical to that of the Maximum Storage Pool condition and the calculated factors of safety are not presented twice.

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 impounded water or CCR material (J-Cell, G-Cell, and C-Cell) and calculated factors of safety for the modeled Maximum Surcharge Pool condition (Table 5-3) are the same as those for the Maximum Storage Pool condition (Table 5-1).

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 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.

The seismic loading condition was applied assuming a Maximum Storage Pool condition (Table 5-4) or existing conditions (Table 5-5), whichever resulted in a more conservative analysis (see discussion in Section 3.1.1). Factors of safety for existing conditions are calculated for cross-sections A-A', B-B', C-C', D-D', E-E', L-L', M-M', N-N', O-O', and P-P'.

3.2 Geometry

In general, 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. Where previous stability analyses were unavailable, internal model geometry was developed using construction drawings and mapping from the original Bechtel reports (1982, 1985a, 1985b) in addition to knowledge of construction practice at the EHP and experience with local geology. 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 figures showing model geometry are in Appendix A.

Figure 1: Units 3 &4 EHP Cross-Section Location Map

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. In addition to the downstream face of the Main and Saddle Dams, factors of safety have also been calculated for the inboard faces under existing conditions (i.e., limited water and CCR material) as indicated in Table 5-2 and Table 5-4. Critical slip surfaces of each analysis are indicated on the figures of Appendix A.

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.

Table 3-3: Material Strength Parameters – Units 3 & 4 EHP

MATERIAL	UNIT WEIGHT		EFFECTIVE FRICTION ANGLE, Φ’	EFFECTIVE COHESION, C’
	MOIST (PCF)	SAT. (PCF)		
EMBANKMENT FILL*	125	130	33°	50 psf
CLAY CORE*	125	130	28.5°	0 psf
DRAIN*	130	135	35°	0 psf
FOUNDATION SOIL*	112	124	28° (SD and H-Cell) 31° (MD and F-Cell)	0 psf
CLINKER ASH**	120.4	125	26.6°	950 psf
FLY ASH SLURRY**	100	103.4	28°	700 psf
PASTE†	102	112	35°	0 psf
BOTTOM ASH FILL†	90	100	40°	50 psf
CLINKER FILL‡	130	-	40°	50 psf
BEDROCK	IMPENETRABLE			

* Bechtel, 1982

** WAI, 2011a

† WAI, 2009c; Golder, 2001

‡ WAI, 2011b

- Denotes material parameter not applied in models

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). Clinker “ash,” or soil-like clinker, was observed during subsurface investigation at the EHP Saddle Dam and soil parameters for this material were selected from the Phase 2 Dam Raise Report (WAI, Stage 2 Dam Raise, 2011).

Strength parameters of CCR materials stored in surface impoundments and used in construction of embankment buttressing were adopted from laboratory testing performed by in previous investigations and reports. See the footnotes of Table 3-3 for references.

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 the Embankment Fill, Clinker Fill, and Bottom Ash Fill in some of the cross-sections. 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, in unsaturated conditions 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.

Sandstones, siltstones, and claystones underlying embankments have all been considered Bedrock and are 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 of materials are modeled using moist unit weights above measured water surface elevations and saturated unit weights below measured water surface elevations. Piezometers within the embankments are being monitored on a monthly basis as part of §257.83(a)(iii) of the new CCR regulations and results from monitoring have been incorporated into the model.

3.4 Phreatic Surface

F-Cell and H-Cell have membrane liners (Geosyntec, 2016d) and vibrating wire piezometers installed in these embankments have not detected seepage within embankment material. Therefore, a phreatic surface has not been applied. Piezometric lines have been added to models for the EHP Main and Saddle Dams based on surface water elevations measured in piezometers and standpipes installed in the embankments (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 (see stability model cross-section figures in Appendix A). In general, seepage pressures do not affect the stability models of the Units 3 & 4 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 3 & 4 surface impoundments are as follows:

1. Materials: Embankment material (i.e., shell and core materials, compacted bottom ash fill) 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 3 & 4 embankments are too high to predict liquefaction at this site.
2. Saturation: Although seepage has been detected within embankment materials by piezometers, saturated conditions exist very low in the embankment 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 3 & 4 EHP 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 through Table 5-5. Cross-section figures of 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

Embankment	Stability Section	Direction	Calculated Factor of Safety
Main Dam	A-A’	Outboard (North)	2.08
Saddle Dam	B-B’	Outboard (NW)	2.19
	C-C’	Outboard (East)	2.34
	D-D’	Outboard (East)	2.35
	E-E’	Outboard (East)	2.44
Southeast EHP Embankment	F-F’	Outboard (SE)	1.91
	G-G’	Outboard (SE)	2.07
Western EHP Embankment	H-H’	Outboard (West)	2.04
B-F Divider Dike	I-I’	F-Cell (South)*	3.99
		B-Cell (North)*	3.79
B-C Divider Dike	J-J’	B-Cell (West)*	3.57
	K-K’	B-Cell (West)*	3.22
G-J Divider Dike	N-N’	G-Cell (South)†	2.83
C-H Divider Dike	Q-Q’	H-Cell (South)*	3.34
		C-Cell (North)	1.98

* Factors of safety were conservatively calculated assuming the cell toward which the analysis was performed was dry (see discussion in Section 3.1.1.)

† G-Cell modeled as dry, J-Cell modeled with Maximum Storage Pool condition with paste elevation of 3,285-ft.

Table 5-2: Results Summary – Static, Existing Conditions

Embankment	Stability Section	Direction	Calculated Factor of Safety
Main Dam	A-A’	Inboard (South)	2.18
Saddle Dam	B-B’	Inboard (SE)	2.59
	C-C’	Inboard (West)	3.21
	D-D’	Inboard (West)	2.95
	E-E’	Inboard (West)	2.20
C-J Divider Dike	L-L’	J-Cell (North)	2.85
	M-M’	J-Cell (North)	3.04
G-J Divider Dike	N-N’	J-Cell (North)	3.22
C-G Divider Dike	O-O’	G-Cell(East)	1.64
	P-P’	G-Cell(East)	1.71

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 condition (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 5-3: Results Summary – Static, Maximum Surcharge Pool

Embankment	Stability Section	Direction	Calculated Factor of Safety
Main Dam	A-A'	Outboard (North)	2.08
Saddle Dam	B-B'	Outboard (NW)	2.19
	C-C'	Outboard (East)	2.34
	D-D'	Outboard (East)	2.35
	E-E'	Outboard (East)	2.44
Southeast EHP Embankment	F-F'	Outboard (SE)	1.91
	G-G'	Outboard (SE)	2.07
Western EHP Embankment	H-H'	Outboard (West)	2.04
F-B Divider Dike	I-I'	F-Cell (South)	3.99
		B-Cell (North)	3.79
G-J Divider Dike	N-N'	G-Cell (South)†	2.83
C-H Divider Dike	Q-Q'	C-Cell (North)	1.98

† G-Cell modeled as dry, J-Cell modeled with Maximum Surcharge Pool condition with paste elevation of 3,285-ft and water surcharge elevation of 3,288-ft.

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 5-4: Results Summary – Seismic, Maximum Storage Pool

Embankment	Stability Section	Direction	Calculated Factor of Safety
Main Dam	A-A’	Outboard (North)	1.44
Saddle Dam	B-B’	Outboard (NW)	1.51
	C-C’	Outboard (East)	1.61
	D-D’	Outboard (East)	1.62
	E-E’	Outboard (East)	1.67
Southeast EHP Embankment	F-F’	Outboard (SE)	1.38
	G-G’	Outboard (SE)	1.53
Western EHP Embankment	H-H’	Outboard (West)	1.47
F-B Divider Dike	I-I’	F-Cell (South)*	2.70
		B-Cell (North)*	2.54
B-C Divider Dike	J-J’	B-Cell (West)*	2.42
	K-K’	B-Cell (West)*	2.18
G-J Divider Dike	N-N’	G-Cell (South)†	1.94
C-H Divider Dike	Q-Q’	H-Cell (South)*	2.27
		C-Cell (North)	1.41

* Factors of safety were conservatively calculated assuming the cell toward which the analysis was performed was dry (see discussion in Section 3.1.1.)

† G-Cell modeled as dry, J-Cell modeled with Maximum Storage Pool condition with paste elevation of 3,285-ft.

Table 5-5: Results Summary – Seismic, Existing Conditions

Embankment	Stability Section	Direction	Calculated Factor of Safety
Main Dam	A-A’	Inboard (South)	1.52
Saddle Dam	B-B’	Inboard (SE)	1.80
	C-C’	Inboard (West)	2.20
	D-D’	Inboard (West)	2.01
	E-E’	Inboard (West)	1.52
C-J Divider Dike	L-L’	J-Cell (North)	1.94
	M-M’	J-Cell (North)	2.72
G-J Divider Dike	N-N’	J-Cell (North)	2.18
C-G Divider Dike	O-O’	G-Cell(East)	1.54
	P-P’	G-Cell(East)	1.21

5.4 Loading Condition: Liquefaction

Liquefaction requirements are described in §274.73(e)(1)(iv). It has been determined that embankments and dikes of the Units 3 & 4 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 3 & 4 surface impoundments were designed and constructed using conservative approaches to stability. In particular, the embankment slopes are not steep and the highest embankments (EHP Main and Saddle Dams) employed zoned construction with drains and filters to prevent piping. Placement of fill appears to have been carefully controlled. Most of the 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.

Abramson, L.W., Lee, T.S., Sharma, S., and Boyce, G.M., 2002, Slope Stability and Stabilization Methods, 2nd ed., John Wiley & Sons, Inc.

American Society of Civil Engineers, 2013, Minimum Design Loads for Buildings and Other Structures (ASCE 7-10).

Bechtel, 1979. Power Corporation, 1979, “Second Stage Evaporation Pond Design Report.” Prepared by Bechtel Power Corporation, December 1979.

Bechtel, 1985a. “Effluent Holding Pond Slurry Cutoff Wall-Final Construction Report.” Prepared by Bechtel Power Corporation, February 1985.

Bechtel, 1985b. “Effluent Holding Pond Embankments-Construction Report.” Prepared by Bechtel Power Corporation, February 1985.

Boulanger, R.W. and Idriss, I.M., 2014, CPT and SPT-Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, California, Report No. UCD/CGM-14-01.

Geosyntec, 2016a. “EHP J Cell Cap and Liner System Design-Geotechnical Report.” Prepared by Geosyntec Consultants, January 2016.

Geosyntec, 2016b, EHP Drift Fence Design-Geotechnical Report. Prepared by Geosyntec Consultants, August 2016.

Geosyntec, 2016c. “Liner Construction Documentation Report: Colstrip Steam Electric Station.” Prepared by Geosyntec Consultants. October 2016.

Geosyntec, 2016d. “History of Construction Report: Colstrip Steam Electric Station, Colstrip, Montana.” Prepared by Geosyntec Consultants, October 2016.

Golder, 2001. “Report on Geotechnical Characterization, Mass Balance, Water Balance, and Conceptual Deposition Plan for Paste Fly Ash Units 3 & 4 Effluent Holding Pond Colstrip Steam Electric Station, Colstrip, Montana.” Prepared by Golder Associates, May 30, 2001.

Hydrometrics, 2016. “Structural Stability Assessment of Coal Combustion Residual (CCR) Disposal Units at Cosltrip Steam Electric Station.” Prepared by Hydrometrics, Inc., October 2016.

Hynes-Griffin, M.E., and Franklin, A.G., 1984. “Rationalizing the Seismic Coefficient Method,” Miscellaneous Paper No. G.L. 84-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Idriss, I.M. and Boulanger, R.W., 2008. “Soil Liquefaction During Earthquakes,” Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA.

Jorgensen, 2016b. “2015 Surface Impoundment Annual Inspection Report: Colstrip Steam Electric Station, Colstrip, Montana.” Prepared by Jorgensen Geotechnical, LLC, January 2016.

Petersen, M.D., A.D. Frankel, S.C. Harmsen, C.S. Mueller, K.M. Haller, R.L. Wheeler, R.L. Wesson, Y. Zeng, O.S. Boyd, D.M. Perkins, N. Luco, E.H. Field, C.J. Wills, & K.S. Rukstales, 2008, “Documentation for the 2008 Update of the United States National Seismic Hazard Maps,” U.S. Geological Survey Open-File Report 2008-1128.

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Yuehua, Rezaeian, Sanaz, Harmsen, S.C., Boyd, O.S., Field, Ned, Chen, Rui, Rukstales, K.S., Luco, Nico, Wheeler, R.L., Williams, R.A., and Olsen, A.H., 2014, Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091.

U.S. Geological Survey Seismic Design Maps Application, 2014, http://earthquake.usgs.gov/designmaps/us/application.php.

WAI, 2012. “Work Plan – Units 3 & 4 Stage 2 Dam Raise, PPL Montana, Colstrip Steam Electric Station.” Prepared by Womack & Associates, Inc., February 2012.

Youd, T.L., and I.M. Idriss, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: Tech. Report NCEER-97-0022.

APPENDIX A

Stability Model Cross-Section Figures

APPENDIX B

Slope Stability Analyses Reports