STRUCTURAL STABILITY ASSESSMENT OF COAL COMBUSTION RESIDUAL (CCR) DISPOSAL UNITS AT COLSTRIP STEAM ELECTRIC STATION

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STRUCTURAL STABILITY ASSESSMENT OF COAL COMBUSTION RESIDUAL (CCR) DISPOSAL UNITS AT COLSTRIP STEAM ELECTRIC STATION

1.0 INTRODUCTION

1.1 BACKGROUND

The United States Environmental Protection Agency (EPA) recently established regulations under Subtitle D of the Resource Conservation and Recovery Act (RCRA) to address the risks associated with disposal of Coal Combustion Residuals (CCRs) generated at electric utilities as well as independent power producers. These regulations became the *Disposal of Coal Combustion Residuals from Electric Utilities* final rule (Coal Ash Rule), which was signed by the EPA Administrator on December 19, 2014 and was published to the Federal Register on April 17, 2015 as Title 40 CFR §257 Subpart D (EPA, 2015). Section 257.73 paragraph (d)(1) requires the performance of a structural stability assessment for each surface impoundment.

Talen Montana, LLC is a partial owner and operator of the Colstrip Steam Electric Station (CSES) located near Colstrip, Montana. Other owners include: Puget Sound Energy Inc., Portland General Electric Company, Avista Corporation, PacifiCorp, and NorthWestern Energy. The CSES operates two pairs of coal-fired generating units known as Units 1&2 and Units 3&4. Units 1&2, placed in service in 1975, have a generating capacity of 330-megawatts-gross each. Units 3&4, placed in service in 1983 and 1985 respectively, have a generating capacity of 805-megawatts-gross each (Hydrometrics, 2012).

CCRs produced by the generating units at CSES are handled and disposed of in a closed-loop process aimed at minimizing impacts to water resources in the area. The closed-loop system is comprised of several surface impoundments on and off the plant site that perform various functions to handle and permanently store CCRs generated by Units 1&2 and 3&4. These surface impoundments and their associated CCR units are now regulated under the new Coal Ash Rule. The CCR units at CSES regulated by the Coal Ash Rule include:

- Units 1&2 Stage II Evaporation Pond (1&2 STEP) D and E/Old Clearwell Cells;
- Units 1&2 Bottom Ash Pond;
- Units 1&2 B Flyash Pond;
- Units 3&4 Effluent Holding Pond (EHP) A, B, C, D/E, G, and J Cells; and
- Units 3&4 Bottom Ash Pond which are incised.

Incised CCR units are exempt from the requirements of §257.73 paragraph (d)(1). Therefore, the 3&4 Bottom Ash Pond will not be further mentioned in this document. Individual pond cells within the 1&2 STEP, 1&2 Bottom Ash and B Flyash Ponds, as well as the 3&4 EHP are separated by internal divider or saddle dikes. Internal divider and saddle dikes have a crest elevation equal to or lower than the main dam or dike of their respective surface impoundment. As will be discussed later in this document, each surface impoundment at CSES is adequately sized to contain or safely route a probable maximum flood event. Failure of an internal divider or saddle dike would not result in a breach volume large enough to overtop the respective main dam or dike for the 1&2 STEP, 1&2 Bottom Ash Pond, or 3&4 EHP. Therefore, these internal divider and saddle dikes do not impact the overall stability of their respective surface impoundment. The locations of CCR units and their surface impoundments are displayed on Figure 1-1.

1.2 PURPOSE

The purpose of this document is to provide an initial structural stability assessment for each Coal Ash Rule regulated surface impoundment at CSES in accordance with the specific requirements of 40 CFR §257.73 paragraph (d)(1). The following structural stability assessments will provide documentation that the design, construction, operation, and maintenance of each surface impoundment is consistent with sound and generally accepted engineering practices.

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STRUCTURAL STABILITY ASSESSMENT
OF CCR DISPOSAL UNITS AT CSES

CCR UNITS
OVERALL LOCATION MAP

FIGURE
1-1

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1.3 REPORT ORGANIZATION

This report is organized as follows:

- Section 2.0 – Structural Stability Assessment of 1&2 STEP Dam;
- Section 3.0 – Structural Stability Assessment of 1&2 Bottom Ash Pond Dike;
- Section 4.0 – Structural Stability Assessment of 3&4 EHP Main and Saddle Dams;
- Section 5.0 – Structural Stability Assessment Certification; and
- Section 6.0 – References.

2.0 STRUCTURAL STABILITY ASSESSMENT OF 1&2 STEP DAM

This section provides an initial structural stability assessment of 1&2 STEP Dam in accordance with §257.73 paragraph (d)(1) of the Coal Ash Rule. As shown on Figure 2-1 below, 1&2 STEP Dam impounds the STEP cells.

2.1 FOUNDATION, ABUTMENT, AND EMBANKMENT STABILITY

Stability of 1&2 STEP Dam requires stable foundations and abutments [§257.73 paragraph (d)(1)(i)] and embankments that have been mechanically compacted to a density sufficient to withstand a range of loading conditions [§257.73 paragraph (d)(1)(iii)]. Stability of the foundation, abutments, and embankment of 1&2 STEP Dam are assessed in the following sections.

2.1.1 Summary of Foundation and Abutment Materials

According to the original design report for 1&2 STEP Dam prepared by Bechtel Power Corporation, the foundation material consists of shale, siltstone, and sandstone bedrock of the Fort Union Formation (Bechtel, 1979). Bedrock was predominantly overlain by clayey silt, alluvium, and fine to medium-grained gravel extending to a maximum depth of 35 feet in the valley bottom along the dam axis and thinning to depths of a few inches on the ridgetops near the abutments. Exploratory borings advanced along the proposed axis of the dam during the geotechnical investigation for design determined the general stratigraphic section to be, from top to bottom: a one foot thick remnant of the McKay coal seam, 60 feet of poorly to moderately cemented siltstone and sandstone, 25 feet of shale, and alternating moderately cemented siltstone and shale. Thin lenses of limestone and carbon veinlets were also encountered. The coal seam was encountered on the north abutment at elevation 3290 feet but was not encountered on the south abutment (Bechtel, 1979). Near surface rock found in the abutment areas was weathered and poorly cemented. Poorly cemented siltstone was encountered to a depth of 30 feet on the south abutment and from depths of 5 to 20 feet on the north abutment (Bechtel, 1979).

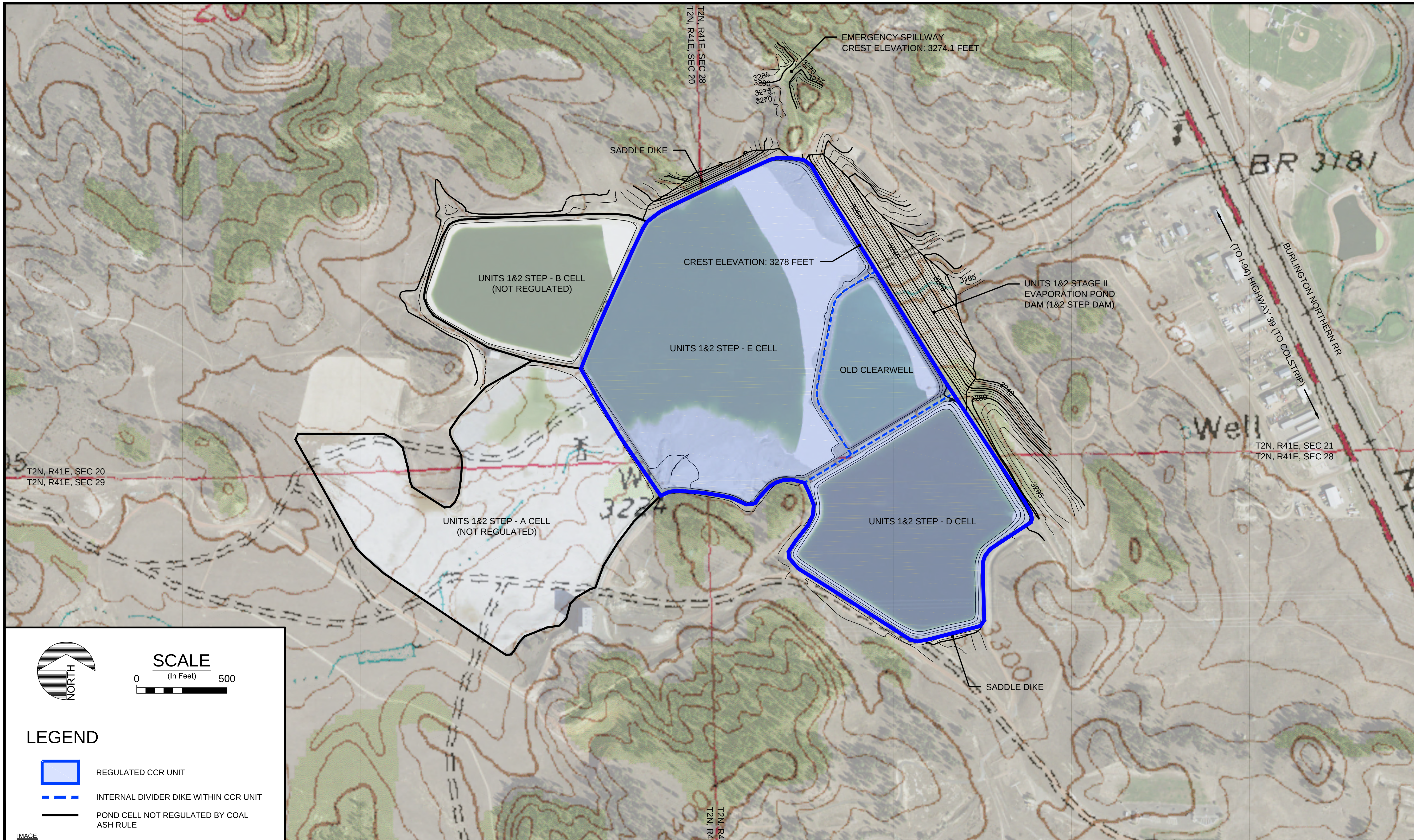

IMAGE
COLSTRIP WEST USGS 7½ MINUTE QUADRANGLE MAP AND 2015 NATIONAL AGRICULTURE IMAGERY PROGRAM (NAIP) AERIAL PHOTO (TILE 6031)

REFERENCE NOTE:
STEP AREA CONTOURS SHOWN REFERENCE DECEMBER 2014 AERIAL SURVEY CONDUCTED BY AERIAL DESIGN DATA OF NORTH HUNTINGTON, PA AND PRESENTED IN *MASTER PLAN SUMMARY REPORT: COLSTRIP STEAM ELECTRIC STATION* PREPARED BY GEOSYNTEC CONSULTANTS OF COLUMBIA, MD.

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STRUCTURAL STABILITY ASSESSMENT
OF CCR DISPOSAL UNITS AT CSES

STAGE II EVAPORATION POND DAM LOCATION MAP

FIGURE
2-1

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In 2010, Womack and Associates, Inc. (WAI) prepared a supplementary geotechnical investigation report for 1&2 STEP Dam (WAI, 2010a). The Womack and Associates report findings generally correlate well with the foundation and abutment material properties presented in the Bechtel design report.

2.1.2 Embankment Construction

Included in Bechtel's original design report for 1&2 STEP Dam is a set "issued for construction" drawings (Bechtel, 1979). Soil materials specified for use in the Bechtel design report were largely validated by WAI's 2010 geotechnical investigation (WAI, 2010a). This indicates that construction of the 1&2 STEP Dam embankment was generally in accordance with the requirements stated in the design report. Construction of 1&2 STEP occurred between 1987 and 1988 (Maxim, 2006a). The drawings in the design report show that the proposed embankment for 1&2 STEP Dam was to be constructed of a zoned rolled earth fill with a central core, exterior shell, drainage layers, and an upstream filter blanket on the north abutment. The following sections summarize the materials and methods prescribed for construction of 1&2 STEP Dam in the Bechtel design report.

2.1.2.1 Foundation Preparation

Prior to placement of 1&2 STEP Dam zoned embankment fill, organic and other unsuitable materials were to be stripped to a minimum depth of 12 inches. Excavation was to be conducted to remove organic and unsuitable materials where they were encountered at a depth greater than 12 inches. The rock foundation for the core trench was to be prepared by removing loose, soft, and broken material. Uneven foundation surfaces were to be leveled by slush grouting. If coal seams were encountered they were to be covered with a four inch layer of slush grout. After stripping and prior to placement of exterior shell material, the existing ground was to be scarified to a depth of 9 inches, moisture conditioned to within 2 percent of optimum, and compacted by a minimum of four passes of a 50- to 60-ton rubber tired roller (Bechtel, 1979).

2.1.2.2 Central Core

The central core for 1&2 STEP Dam has a top width of 16 feet at an elevation of 3274.6 feet and extends downward to a core trench. Each exterior face of the core slopes outward from the centerline at 1H:3V from the top down to the core trench. The core trench was excavated a minimum of two feet into bedrock and varies in width from 20 to 55 feet. Below the central core trench a grout curtain was drilled to a depth of 80 feet along the dam axis for seepage control. Design drawings specify Zone 1 soil materials for the central core which consist of inorganic silt and clay with more than 50 percent passing a No. 200 U.S. Standard Sieve. Zone 1 soil materials were moisture conditioned to within +2 percent of optimum and compacted in 12 inch lifts to a minimum of 95 percent of maximum dry density as determined by ASTM D-1557, Method D (Bechtel, 1979).

2.1.2.3 Exterior Shell

The exterior shell for 1&2 STEP Dam surrounds the central core and features a 20-foot wide crest and 3H:1V upstream and downstream slopes which daylight to existing ground. The 1&2 STEP Dam crest elevation of 3278 feet results in a maximum height of 88 feet above existing ground. Design drawings specify Zone 2 soil materials for the exterior shell which consist of inorganic silt and clay overburden, as well as soft and friable siltstone, sandstone, and shale. The crest of 1&2 STEP Dam was capped with a layer of baked shale to provide an erosion resistant surface for vehicle traffic. Design drawings show a transition of finer to coarser soil material from the central core outward to the upstream and downstream slope faces. Zone 2 soil materials were moisture conditioned to within ±2 percent of optimum and compacted in 12-inch lifts to a minimum of 95 percent of maximum dry density as determined by ASTM D-1557, Method D (Bechtel, 1979).

2.1.2.4 Drainage Layers

1&2 STEP Dam has chimney, inclined, and horizontal blanket drainage layers (Bechtel, 1979). The purpose of these drainage layers is to collect and route seepage to a toe drain system consisting of perforated pipe installed in a drainage trench running along the toe of the dam. The toe drain flows to a drain in the valley bottom, referred to as the “valley drain,” which discharges to a concrete sump approximately 540 feet downstream of the dam toe.

Seepage water collected in the valley drain sump is pumped back to the ponds (Hydrometrics, 2014a). The chimney drain is installed between the exterior shell and the downstream face of the central core. The inclined drain is installed between the downstream face of the core trench and the foundation. The chimney and inclined drains are separated from the central core and core trench by a five foot thick transition layer provided to prevent contamination of the drains with fine grained soils. The horizontal blanket drain, which connects the chimney and inclined drains to the toe drain, is placed between the exterior shell and the downstream embankment foundation.

The horizontal blanket drain was specified to be constructed with Zone 3 soil materials encased in Zone 5 soil materials. The chimney and inclined drains are specified to be constructed with Zone 5 soil material. The transition zone is specified to be constructed with Zone 1A soil material. Zone 1A soil materials were specified as Zone 1 or 2 soils with less than 80 percent passing a No. 200 U.S. Standard Sieve. Placement and compaction of Zone 1A soil materials was to be accomplished in the same manner as Zone 1 soils. Zone 3 soil materials were specified as processed sand and gravel while Zone 5 soil material was specified as processed sand. Zone 3 and 5 soils were to be conditioned to a moisture content between 8 and 12 percent and compacted in 12-inch lifts by a minimum of 4 passes with a 50- to 60-ton rubber tired roller.

2.1.2.5 <u>Upstream Filter Blanket</u>

The left abutment of 1&2 STEP Dam required an upstream filter blanket due to a coal seam that was encountered during the geotechnical investigation for design (Bechtel, 1979). The filter blanket was to be a five foot thick layer of Zone 1 material. Zone 1 filter blanket soils were to be moisture conditioned to ±2 percent of optimum and compacted to 90 percent of maximum dry density as determined by ASTM D-1557, Method D. Compaction of each 12-inch thick filter blanket lift was to be accomplished by two track walking passes with a bulldozer. Installation of geosynthetic liner systems in the upstream STEP cells has removed the need for this filter blanket.

2.1.3 Foundation, Abutment, and Embankment Stability Summary

Prior to the implementation of the Coal Ash Rule regulations, engineer's inspections of 1&2 STEP Dam were performed every five years. A review of available inspection reports from 1999, 2005, 2009, and 2014 revealed no observations of unusual movement of the embankment on its foundation. These inspection reports had no observations of cracking, sloughing, sliding, scarping, erosion, or unusual movement in the crest, upstream slope, or downstream slope of the embankment and reported good contact between the embankment and abutments. Furthermore, no low areas were noted in the 1&2 STEP Dam crest (Maxim, 1999a and 2006a; Hydrometrics, 2009a and 2014a).

Monitoring of instrumentation installed in the 1&2 STEP Dam embankment and foundation, which includes two slope inclinometers and six vibrating wire (VW) piezometers, has been conducted at least semi-annually since its installation (Jorgensen, 2016a). Review of inclinometer monitoring data presented in the Jorgensen report shows very little, if any, movement of the embankment on its foundation or abutments over the monitoring period. All piezometers but one have been dry since their installation in 2009. One piezometer near the toe of the embankment detected groundwater, thought to be surface infiltration, during an abnormally wet spring in 2011 (Jorgensen, 2016a). The largely dry piezometer readings are expected due to the installation of geosynthetic liners in the STEP cells. The piezometer data indicates a low phreatic surface, if one is present at all, within the 1&2 STEP Dam embankment which makes development of adverse pore water pressure conditions within the dam embankment very unlikely.

A slope stability assessment of the 1&2 STEP Dam embankment, required by the Coal Ash Rule, was recently performed by Jorgensen Geotechnical, LLC (Jorgensen, 2016b). The results of this slope stability analysis determined that the 1&2 STEP Dam embankment exceeds required factors of safety against failure for a range of loading conditions required by the Coal Ash Rule.

2.2 SLOPE PROTECTION

Adequate slope protection is needed to protect against surface erosion, wave action, and adverse effects of sudden drawdown [§257.73 paragraph (d)(1)(ii)]. 1&2 STEP Dam has no low level outlet; therefore, sudden drawdown is not a concern. Upstream slope protection against surface erosion and wave action for 1&2 STEP Dam is provided by geosynthetic lining of the upstream cells. The downstream slope of the 1&2 STEP Dam embankment is protected against surface erosion by a 6-inch layer of topsoil seeded with grass (Bechtel, 1979). The most recent inspection reported that the liner on upstream slope appeared to be in good condition with a good vegetative cover between the liner and dam crest (Hydrometrics, 2014a). The inspection report also noted good grass coverage on the downstream slope with the exception of weeds around several rodent holes. The inspection report made recommendations to fill in the rodent holes with soil and then monitor for continued rodent activity. An ongoing rodent control program has been implemented by CSES staff to address this recommendation (PPL, 2015).

2.3 SLOPE VEGETATION

Vegetated slopes of dikes should not exceed a height of six inches above the slope of the dike, except where an alternate form of slope protection has been provided [§257.73 paragraph (d)(1)(iv)]. However, Case Number 15-1219 *Utility Solid Waste Activities Group, et al. vs. Environmental Protection Agency* filed on June 14, 2016 in the U.S. Court of Appeals removed the six inch height requirement for slope vegetation and revised §257.73(d)(1)(iv) to simply require that embankment slopes must be vegetated or protected by an alternative means. As discussed in Section 2.2 above, the upstream and downstream slopes of 1&2 STEP Dam embankment meet this requirement with a combination of geosynethic liner and grass cover employed as means of slope protection.

2.4 SPILLWAY CAPACITY

The spillway must be properly configured and have the capacity to manage flow during and following the peak discharge from a specified flood event [§257.73 paragraph (d)(1)(v)]. As 1&2 STEP Dam has a high hazard potential classification, the required design inflow flood event for this impoundment is a probable maximum flood (PMF) [(d)(1)(v)(B)]. The design inflow flood used for the impoundment exceeds this requirement.

1&2 STEP Dam has an emergency spillway approximately 400 feet northwest of the left abutment. The emergency spillway is at an elevation of 3274.6 feet, and is an unlined, uncontrolled earth channel with a bottom width of 25 feet, 2H:1V side slopes, and a length of 100 feet (Hydrometrics, 2014a). The spillway design analysis used exceeded the requirements in the CCR rules by using a combination of a 100 year flood followed by the probable maximum flood (PMF) and determined that this combination would raise the pond elevation about 4.6 feet from the maximum operating level of 3270 feet to an elevation of 3274.6 feet, the selected spillway elevation (Bechtel, 1979). However, at its current elevation, the spillway has only 3.4 feet of freeboard before the 1&2 STEP Dam embankment crest is overtopped. Therefore, additional analysis was required.

Following an update to the hydrometeorological report (HMR) for this region, an independent check of 1&2 STEP Dam flood routing was performed for the 1988 inspection report (Chen-Northern, 1988a). A 72-hour PMF event was used for the analysis and determined that the STEP cells would hold all but 501 acre-feet. A spillway analysis in that report determined that excess flood water would pass through the spillway at a peak flow rate of 111 cubic feet per second at a flow depth of 0.8 feet. The peak flow depth of 0.8 feet is well below the 3.4 feet of freeboard provided by the spillway. Therefore, available data indicates that 1&2 STEP Dam and its emergency spillway have adequate capacity to safely pass a full PMF event routed through the STEP.

2.5 LOW LEVEL HYDRAULIC STRUCTURES

The 1&2 STEP Dam does not have a low level outlet or any other type of hydraulic structure passing through its embankment [§257.73 paragraph (d)(1)(vi)].

2.6 INUNDATED DOWNSTREAM SLOPES

There are no water bodies adjacent to 1&2 STEP Dam which could potentially inundate the downstream slope [§257.73 paragraph (d)(1)(vii)].

2.7 IDENTIFICATION OF STRUCTURAL STABILITY DEFICIENCIES

No structural stability deficiencies were discovered during this initial assessment [§257.73 paragraph (d)(2)].

3.0 STRUCTURAL STABILITY ASSESSMENT OF 1&2 BOTTOM ASH POND DIKE

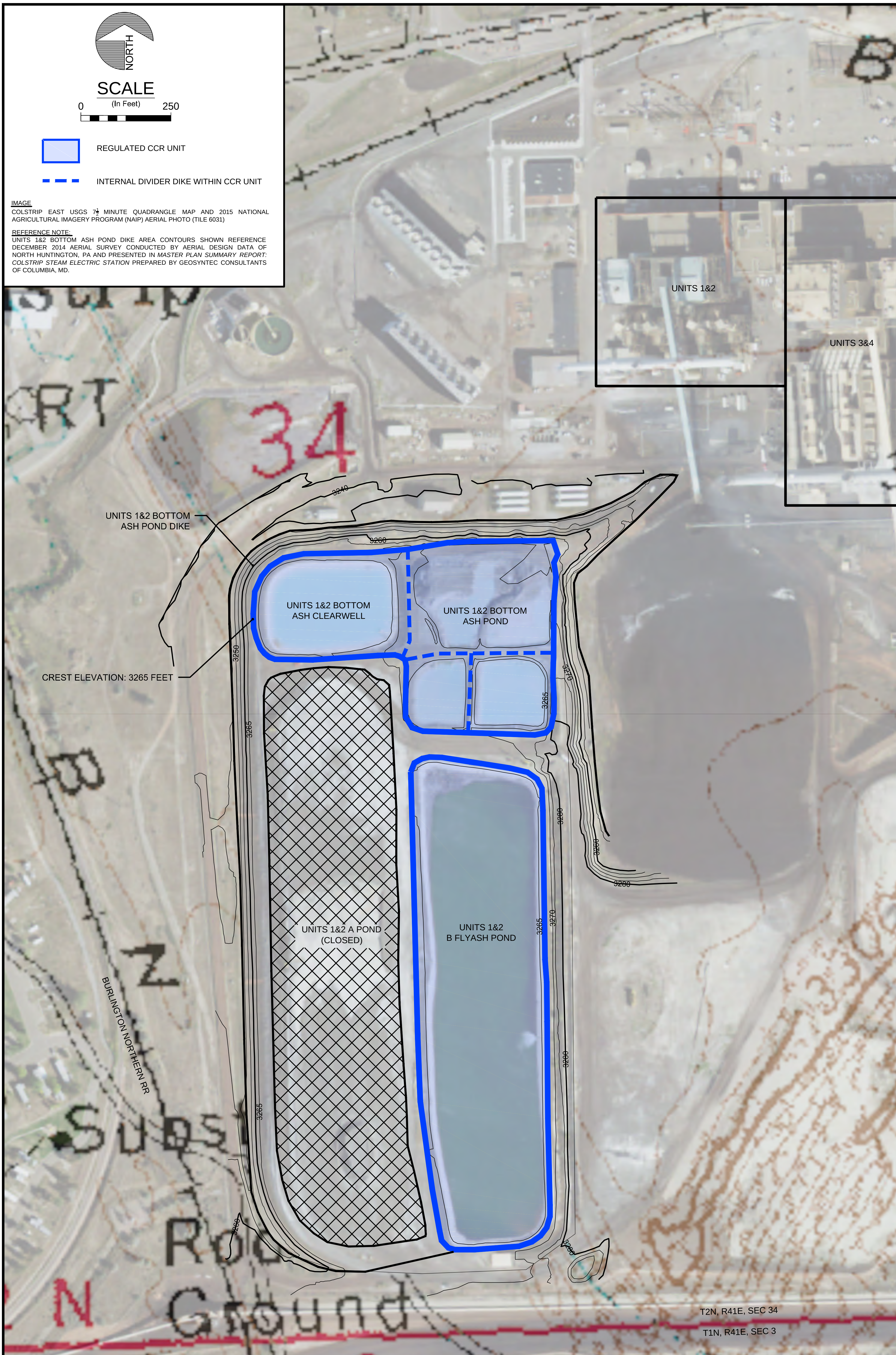
This section provides an initial structural stability assessment of 1&2 Bottom Ash Pond (BAP) Dike in accordance with §257.73 paragraph (d)(1) of the Coal Ash Rule. As shown on Figure 3-1 below, 1&2 BAP Dike impounds Units 1&2 Bottom Ash Pond and B Flyash Pond (B Pond).

3.1 FOUNDATION, ABUTMENT, AND EMBANKMENT STABILITY

Stability of the 1&2 BAP Dike requires stable foundations and abutments [§257.73 paragraph (d)(1)(i)] and embankments that have been mechanically compacted to a density sufficient to withstand a range of loading conditions [§257.73 paragraph (d)(1)(iii)]. However, the 1&2 BAP Dike was originally constructed as a continuous embankment that circles the impoundment. Therefore, 1&2 BAP Dike has no abutments to assess.

3.1.1 Summary of Foundation Materials

Beyond design drawings prepared by Bechtel Power Corporation (Bechtel, 1974) “Issued for Construction” in 1974, there is no existing data on actual construction of the 1&2 BAP Dike as the original design report could not be located. However, in late 2009 WAI conducted geotechnical investigations of the 1&2 BAP Dike embankment and foundation in an attempt to fill in data gaps (WAI, 2010b and WAI 2010c). Soil borings were performed in 1&2 BAP Dike west of former Units 1&2 A Pond (A Pond), northwest of the Bottom Ash Clearwell, and north and east of the Bottom Ash Pond. Borings in the west and northwest portion of the 1&2 BAP Dike indicated that foundation materials consisted of very stiff to hard, inorganic clay alluvium (WAI, 2010b). The boring performed west of former A Pond encountered a six foot thick layer of wet, loose to medium dense silty sand alluvium at a depth of 33 to 39 feet below the ground surface. Borings in the north and east portion of the 1&2 BAP Dike encountered foundation materials composed of weathered claystone and shale bedrock of the Fort Union Formation (WAI, 2010c). Corrected SPT blow counts exceeded 100 blows per foot in the foundation material. A three foot thick lens of sandy silt alluvium overlying bedrock was encountered at 37 feet below the embankment crest north of the Bottom Ash



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UNITS 1&2 BOTTOM ASH POND DIKE LOCATION MAP

FIGURE
3-1

Pond. This lens was not thought to be continuous or representative of all foundation materials (WAI, 2010c).

3.1.2 Embankment Construction

As mentioned in the previous section, little is known about the original construction of the 1&2 BAP Dike. Bechtel's design drawings, included in Appendix C, indicate that the 1&2 BAP Dike was constructed as a zoned earth embankment sometime in the mid-1970s (Bechtel, 1974). Over time, changes have been made to the configuration of the cells and internal divider dikes within 1&2 BAP Dike, but the Dike itself has remained unchanged since initial construction. The zoned earth embankment was to be constructed of a central core and exterior shell. This construction method was confirmed by the WAI geotechnical investigations of 1&2 BAP Dike (WAI, 2010b and WAI, 2010c). Overall, the 1&2 BAP Dike is approximately 4,000 feet in length with a maximum height of approximately 25 feet. The Bechtel design drawings show a 20-foot wide core trench keyed into bedrock at maximum sections of the 1&2 BAP Dike including: a 400 foot length along the northwest corner and a 300 foot length along the southeast corner. However, it is important to note that fill has since been placed adjacent to 1&2 BAP Dike south and east of B Pond to an elevation above the elevation of dike. It is also important to note that A Pond was closed in 2015 resulting in a large portion of the west side of the 1&2 BAP Dike no longer impounding any water (Hydrometrics, 2015). The properties of the core and shell soil materials are discussed in the following sections. Information regarding the original construction of the 1&2 BAP Dike, including material specifications and placement methods, could not be found.

3.1.2.1 <u>Central Core</u>

Typical sections in the Bechtel design drawings show the central core for the 1&2 BAP Dike extending downward through the full depth of the embankment. The central core has a 20 foot top width and exterior faces sloping outward at 1H:3V down to the base (Bechtel, 1974). The top of the central core is capped with a foot of road base for erosion protection. As mentioned above, the central core is keyed into bedrock in a 20-foot wide core trench along the northwest and southeast corners of the 1&2 BAP Dike. Central core soil material encountered in a boring on the west side of former A Pond was plastic sandy silty clay with medium stiffness (WAI, 2010b). Central core soil materials found in borings north and east of the Bottom Ash Pond is composed of stiff to hard, slightly plastic silty clay (WAI, 2010c).

3.1.2.2 Exterior Shell

Typical sections in the Bechtel design drawings show the exterior shell, noted as "random earth," for the 1&2 BAP Dike surrounding the central core and extending downward from each side of the 20 foot wide embankment crest at 2H:1V upstream and downstream slopes which daylight to the original ground surface (Bechtel, 1974). The 1&2 BAP Dike has a minimum crest elevation of 3265 feet. Shell soil materials encountered in borings were typically coarser than core soil materials (WAI, 2010b and WAI, 2010c). Shell soil materials found in borings west of A Pond and northwest of the Bottom Ash Clearwell were largely heterogeneous and ranged from silty clay to gravelly sand (WAI, 2010b). Shell soil materials found in borings north and east of the Bottom Ash Pond were slightly heterogeneous with classifications ranging from sandy silt to gravelly clay (WAI, 2010c).

3.1.3 Foundation, Abutment, and Embankment Stability Summary

Prior to implementation of Coal Ash Rule regulations, engineer's inspections of 1&2 BAP Dike were performed every five years. Available inspection reports from 2009 and 2014 reported no observation of unusual movement of the dike on its foundation (Hydrometrics, 2009b and 2014b). In addition, there were no observations of cracking, sloughing, sliding, scarping, or unusual movement in the crest, upstream slope, or downstream slope of the embankment. While they do not impact overall stability of 1&2 BAP Dike, the inspection reports also note good abutment contact between internal divider dikes and 1&2 BAP Dike. It is also important to note that the recent removal from service of A Pond in 2015 resulted in a large portion of the west side of 1&2 BAP Dike no longer impounding water or CCR (Hydrometrics, 2015).

Monitoring of instrumentation installed in the 1&2 BAP Dike embankment and foundation, which includes six piezometers, has been conducted at least semi-annually since the instruments were installed (Jorgensen, 2016a). Only one piezometer, installed near the crest of the northwest corner of 1&2 BAP Dike, has recorded a water level over the monitoring period. The recorded water level in this piezometer is still indicative of a low phreatic surface within the 1&2 BAP Dike embankment and it is noteworthy that there was a significant drop in water level corresponding with the closure of A Pond. The remaining piezometers have been dry. Piezometer data indicates that the core is effective in maintaining a low phreatic surface within the 1&2 BAP Dike embankment and that there is a low potential for development of adverse pore water pressure conditions that could threaten embankment stability.

A slope stability assessment of the 1&2 BAP Dike, required by the Coal Ash Rule, was recently performed by Jorgensen Geotechnical, LLC (Jorgensen, 2016b). The results of this slope stability analysis determined that the Dike embankment exceeds required factors of safety against failure for a range of loading conditions required by the Coal Ash Rule.

3.2 SLOPE PROTECTION

Adequate slope protection is required to protect against surface erosion, wave action, and adverse effects of sudden drawdown [§257.73 paragraph (d)(1)(ii)]. 1&2 BAP Dike has no low level outlet; therefore, sudden drawdown is not a concern. Upstream slope protection for 1&2 BAP Dike varies between each impounded cell. Geosynthetic liner systems in B Pond and the Bottom Ash Clearwell provide upstream slope protection against surface erosion and wave action along a majority of the dike to the northwest and southeast. There is no upstream slope protection provide in the bottom ash cells; however, the large volume of bottom ash solids compared to a relatively small volume of decant water results in little potential for erosion due to wave action. Downstream slope protection against surface erosion for 1&2 BAP Dike is provided by six inches of topsoil and vegetation (Bechtel, 1974). The most recent inspection report noted minor erosion rills on an unprotected portion of the upstream slope caused by crest runoff (Hydrometrics, 2014b). The report also noted generally good vegetative cover on the downstream slope of 1&2 BAP Dike. Two exceptions to this were noted where crest runoff caused erosion rills in two areas locally sparse vegetation on the downstream slope. Several rodent holes were also noted. Recommendations were made to repair noted erosion rills on the upstream and downstream embankment slopes, backfill the rodent holes with soil, and to provide continued monitoring for burrowing animal activity. To address these recommendations the erosion and animal burrows have since been repaired by CSES staff and a rodent control program has been implemented (PPL, 2015).

3.3 SLOPE VEGETATION

Vegetated slopes of dikes not exceed a height of six inches above the slope of the dike, except where an alternate form of slope protection has been provided [§257.73 paragraph (d)(1)(iv)]. As stated in Section 2.3 the six inch height requirement has been removed from this rule. With one minor exception the 1&2 BAP Dike meets this requirement with grass covered downstream slopes and geosynthetic lined upstream slopes which are discussed in the previous section. The exception, also discussed in the previous section, is a small unprotected portion of the upstream slope formed by the bottom ash ponds.

3.4 IMPOUNDMENT CAPACITY

The spillway must be properly configured and have the capacity to manage flow during and following the peak discharge from a specified flood event [§257.73 paragraph (d)(1)(v)]. As 1&2 BAP Dike has a high hazard potential classification, the required inflow design flood event for this impoundment is a probable maximum flood (PMF) [paragraph (d)(1)(v)(B)].

1&2 BAP Dike has no spillways, outlets, or other means of rapidly lowering water levels. Water and CCR waste levels within the individual pond cells are maintained by pumping. The maximum operating level for the B Pond and the Bottom Ash Pond is 3260 feet, which provides at least five feet of freeboard below the minimum embankment crest elevation of 3265 feet. Run-on is diverted away from the 1&2 BAP Dike area. This means that precipitation falling within the footprint of the ponds is the only water received during a given storm event. The general and local PMF events were previously analyzed for 1&2 BAP Dike (Hydrometrics, 2014b). The analysis determined that the general PMF event, the more severe of the two cases, would result in 29 inches of rainfall over a 72-hour period. From the maximum operating pool, this PMF event would raise the water surface 2.42 feet to elevation 3262.42 which leaves over 2.5 feet of freeboard before overtopping of the 1&2 BAP Dike would occur. While there is no spillway, the results of this analysis strongly suggest that the 1&2 BAP Dike meets the Coal Ash Rule requirements as the ponds are capable of safely storing a 72-hour PMF with a sufficient amount of excess freeboard remaining.

3.5 LOW LEVEL HYDRAULIC STRUCTURES

The 1&2 BAP Dike does not have a low level outlet or any other type of hydraulic structure passing through its embankment [§257.73 paragraph (d)(1)(vi)].

3.6 INUNDATED DOWNSTREAM SLOPES

There are no water bodies adjacent to 1&2 BAP Dike which could potentially inundate the downstream slope [§257.73 paragraph (d)(1)(vii)].

3.7 IDENTIFICATION OF STRUCTURAL DEFICIENCIES

No structural stability deficiencies were noted during this initial assessment [§257.73 paragraph (d)(2)].

4.0 STRUCTURAL STABILITY ASSESSMENT OF UNITS 3&4 EHP MAIN AND SADDLE DAMS

This section provides an initial structural stability assessment of Units 3&4 EHP Main (Main Dam) and Saddle Dams (Saddle Dam) in accordance with §257.73 paragraph (d)(1) of the Coal Ash Rule. As shown on Figure 4-1 below, the Main and Saddle Dams impound the 3&4 EHP cells.

4.1 FOUNDATION, ABUTMENT, AND EMBANKMENT STABILITY

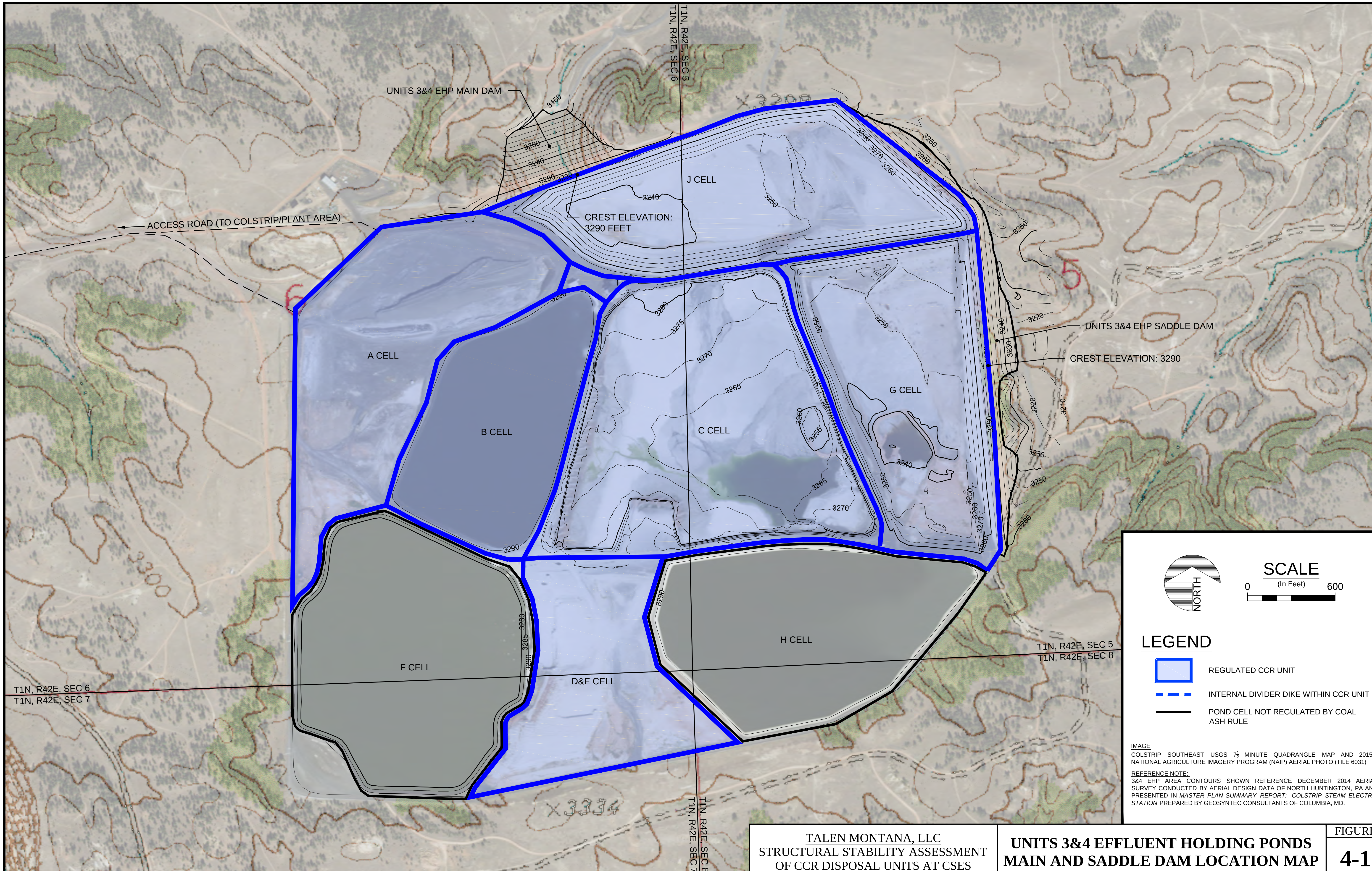
Stability of the Main and Saddle Dams require stable foundations and abutments [§257.73 paragraph (d)(1)(i)] and embankments that have been mechanically compacted to a density sufficient to withstand a range of loading conditions [§257.73 paragraph (d)(1)(iii)]. Stability of the foundation, abutments, and embankments of the Main and Saddle Dams is assessed in the following sections.

4.1.1 Summary of Foundation and Abutment Materials

A summary of foundation and abutment materials for the Main and Saddle Dams is provided below.

4.1.1.1 Units 3&4 EHP Main Dam

Bechtel Power Corporation conducted an extensive investigation of the foundation and abutment soils underlying Main Dam during the design process (Bechtel, 1982). Additionally, the results of Bechtel's subsurface investigation for design reasonably agree with the findings of WAIs 2009 geotechnical investigation of the Main Dam (WAI, 2010d). Below elevation 3165 feet the foundation material for Main Dam consists of low permeability, dense, interbedded sandstone, siltstone, and claystone bedrock of the Fort Union Formation. A very poorly to poorly cemented fine to medium grained sandstone unit averaging 40 to 60 feet thick to an approximate elevation of 3200 feet overlies the bedrock unit. Above the sandstone unit is a layer of interbedded and weathered sandstone, siltstone, and claystone with interspersed carbonaceous shale or coal beds. A deposit of colluvium and alluvium overburden consisting of silty sand, sandy silt, and silty clay was encountered to a



TALEN MONTANA, LLC
STRUCTURAL STABILITY ASSESSMENT
OF CCR DISPOSAL UNITS AT CSES

UNITS 3&4 EFFLUENT HOLDING PONDS
MAIN AND SADDLE DAM LOCATION MAP

FIGURE
4-1

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Hydrometrics, Inc.
Consulting Scientists and Engineers

maximum depth of approximately 28 feet in the valley bottom along the Main Dam alignment.

The upper third of the left and right dam embankment abutment material for the Main Dam is composed of a baked shale unit beginning at an approximate elevation of 3220 feet. The baked shale unit is highly permeable and consists of weathered, baked claystone, siltstone, and sandstone within a matrix of loose, dry silt and fine grained sand. Below the baked shale unit, the middle third of the abutments consist of the weathered, interbedded sandstone, siltstone, and claystone unit mentioned above. The bottom third of the abutments contacts the very poorly to poorly cemented sandstone unit also mentioned in the previous paragraph. The valley bottom overburden is also found along the abutments.

4.1.1.2 <u>Units 3&4 EHP Saddle Dam</u>

As with the Main Dam, Bechtel conducted an extensive investigation of the foundation and abutment soils underlying the Saddle Dam during the design process (Bechtel, 1982). The results of geotechnical investigations of the Saddle Dam performed by Hydrometrics in 2000 (Hydrometrics, 2000) and WAI in 2009 (WAI, 2009) generally correlate well with Bechtel's subsurface investigation findings for design. The foundation material for the Saddle Dam consists of low permeability, moderately hard, interbedded sandstone, siltstone, and claystone bedrock of the Fort Union Formation. Overlying bedrock is a 10-foot seam of coal of the McKay Coal. A 10 to 30-foot thick layer of weathered, interbedded sandstone, siltstone, and claystone occurs over the coal seam. Deposits of colluvium and alluvium consisting of silty sand, sandy silt, and silty clay were encountered in low lying areas to maximum depths of 10 feet along the Saddle Dam alignment.

The left and right abutment material for the Saddle Dam is composed of a baked shale unit, up to 60 feet thick, beginning at an approximate elevation of 3210 feet in the north and 3225 feet in south which overlies the weathered sandstone, siltstone, and claystone unit mentioned above. The baked shale unit is highly permeable and consists of weathered, baked claystone, siltstone, and sandstone within a matrix of loose, dry silt and fine grained sand.

4.1.2 Embankment Construction

Design and construction of the embankments for the Main and Saddle Dams are well documented in reports prepared by Bechtel Power Corporation (Bechtel, 1982 and 1985). The embankments for the Main and Saddle Dams are both rolled zoned earth fill consisting of central core, exterior shell, and downstream drainage layers. A slurry cutoff wall was constructed to bedrock below the core trench of each dam embankment and around the perimeter of the 3&4 EHP area to mitigate seepage. The first phase of construction, performed between 1983 and 1984, completed the dam embankments to an elevation of 3260 feet. The second phase of construction, conducted in 2012, raised each dam embankment to a final elevation of 3290 feet (WAI, 2012). The following sections summarize the materials and methods used to construct the Main and Saddle Dam embankments.

4.1.2.1 <u>Foundation Preparation</u>

Foundation preparation for the Main and Saddle Dams included stripping and excavation to remove organics, unsuitable material, and alluvial/colluvial overburden from the core trench and embankment abutments (Bechtel, 1985). Abutments were machine cleaned and moisture conditioned prior to placement of the respective embankment fills. Core trench contact areas were machine cleaned, hand cleaned, and blown out with forced air prior to placement of central core material. Baked shale encountered in the foundations outside of the core trenches on the upstream side was moisture conditioned and rolled with a vibratory tamping foot roller until a tight and dense surface was provided. Exposed baked shale in the Main Dam abutments above an elevation of 3230 feet was cleaned and then covered with a four inch layer of lean concrete. A 7 to 8 foot layer of fractured, blocky siltstone near an elevation of 3230 feet in the Main Dam abutments was hand cleaned and treated with dental concrete. All foundation contact areas outside of the abutments and core trench were moisture conditioned prior to placement of embankment fill.

4.1.2.2 <u>Central Core</u>

Bechtel's as-built drawings show the central cores of the Main and Saddle Dams have top widths of 33.4 feet at an elevation of 3256 feet and extend downward to a core trench. Upstream and downstream exterior faces of each central core slope outward at 1H:3V down to the core trench. Each core trench is excavated a minimum of two feet into bedrock and five into baked shale where it was encountered (Bechtel, 1985). The core trench for the Main Dam varies in width from 20 feet to a maximum of 105 feet at the bottom of the valley. The core trench for the Saddle Dam varies in width from 20 feet to a maximum of 45 feet in the two lowest areas. The slurry cutoff wall mentioned above extends five feet vertically into each of the core trenches. Zone 1 soil material used for each central core and core trench consists of low to medium plasticity clayey silts and silty clays averaging 78 to 86 percent passing a No. 200 U.S. Standard Sieve. Zone 1 soils used for the central core were moisture conditioned as necessary and compacted using a variety of self-propelled vibratory tamping foot rollers (Bechtel, 1985). No changes were made to the central cores of each dam during the 2012 embankment raise (WAI, 2012).

4.1.2.3 <u>Exterior Shell</u>

Bechtel's as-built drawings show that the exterior shells for the Main and Saddle Dams surround their respective central cores and feature a 20 foot wide crest and 3H:1V upstream and downstream slopes which daylight to existing ground (Bechtel, 1985). As mentioned above, the exterior shells of each dam were originally constructed to a crest elevation of 3260 feet and were later raised to a final elevation of 3290 feet. The final crest elevation of 3290 feet results in a maximum embankment height of 138 feet for the Main Dam and 66 feet for the Saddle Dam. The Zone 2 soil materials used for the exteriors shells below elevation 3260 consist of inorganic silty and clayey overburden, friable siltstone, sandstone and silty shale averaging 60 to 70 percent passing a No. 200 U.S. Standard Sieve. Zone 2 soils used in each shell were moisture conditioned as necessary and compacted with a variety of self-propelled vibratory tamping foot rollers (Bechtel, 1985). Processed baked shale was used for the embankment raise above an elevation of 3260 feet (WAI, 2012). Bottom ash was placed over existing upstream slopes below an elevation of 3260 feet to serve as a filter blanket and to maintain the 3H:1V slope during the embankment raise (WAI, 2012).

4.1.2.4 <u>Drainage Layers</u>

The Main and Saddle Dams each have chimney, horizontal blanket, and inclined drain layers (Bechtel, 1982 and 1985). The Main Dam has an abutment drain in the east abutment to collect seepage passing through a layer of high permeability baked shale (Bechtel, 1982 and 1985). The purpose of these drains is to collect and route seepage to a toe drain system consisting of perforated pipe installed in a drainage trench running along the toe of the each dam. The toe drains, and abutment drain in the Main Dam, flow to valley drains which discharge to concrete sumps. Seepage water collected in the valley drain sumps is pumped back to the 3&4 EHP (Hydrometrics, 2014c). The chimney drains are installed between the exterior shell and the downstream face of the central core. The inclined drains are installed between the downstream face of the core trench and the foundation. Chimney and inclined drains are separated from the central core and core trench by a five-foot thick layer of transition material to prevent contamination with fine grained soils. The horizontal blanket drains, which connect the chimney and inclined drains to the toe drain, are placed between the exterior shell and the downstream embankment foundations.

Zone 1A soil materials, consisting of select sandy silts and sandy clays averaging 35 to 45 percent passing a No. 200 U.S. Standard Sieve, were used to construct the transition between the central core and the chimney and inclined drains (Bechtel, 1985). Zone 3 soil material, consisting of well graded processed 1-inch minus gravel and sand, was used to construct the horizontal blanket drain and as backfill around the toe and valley drainage systems (Bechtel, 1985). Zone 4 soil materials, consisting of baked shale processed to 4-inch minus particle sizes, were used as cover material over the toe drains. Zone 5 soil material, consisting of well graded processed 3/8-inch minus sand, was used to construct the chimney, inclined, and abutment drains (Bechtel, 1985). Additionally, Zone 5 soil materials were used as backfill around blanket drains as well as around toe and valley drain piping. Zone 1A soils were moisture conditioned as necessary and compacted with a vibratory sheepsfoot roller. Zone 3 and 5 soils were compacted with vibratory smooth-drum roller.

4.1.3 Foundation, Abutment, and Embankment Stability Summary

A summary of the foundation, abutment, and embankment stability assessment for the Main and Saddle Dam is presented below.

4.1.3.1 <u>Units 3&4 EHP Main Dam</u>

Prior to implementation of Coal Ash Rule regulations, engineer's inspections of the Main Dam were performed every five years. A review of available inspection reports from 1999, 2005, 2009, and 2014 revealed no observations of unusual movement of the embankment on its foundation (Maxim, 1999b and 2006b; Hydrometrics, 2009c and 2014c). These inspection reports had no observations of cracking, sloughing, sliding, scarping, erosion, or unusual movement in the crest, upstream slope, or downstream slope of the embankment and reported good contact between the embankment and abutments.

Monitoring of instrumentation installed in the Main Dam embankment and foundation, which includes two slope inclinometers and 12 piezometers, has been conducted at least semi-annually since its installation (Jorgensen, 2016a). Review of the inclinometer monitoring data presented in the Jorgensen report shows very little, if any, movement of the Main Dam embankment on its foundation or abutments over the monitoring period. Review of piezometer data shows that water levels within the Main Dam embankment have changed very little over the monitoring period and indicates that the central core and downstream drainage layers are effective at maintaining a low phreatic surface within the embankment. It is also important to note that a geosynthetic liner system is currently being installed on the upstream slope of the Main Dam as part of a project to line adjacent J Cell (Hydrometrics, 2016). It is likely that this liner system will further depress or eliminate the existing phreatic surface within the Main Dam embankment.

A slope stability analysis of the Main Dam embankment was recently performed by Jorgensen Geotechnical, LLC (Jorgensen, 2016c). This analysis determined that the Main Dam embankment exceeds required factors of safety against failure for a range of loading conditions required by the Coal Ash Rule.

4.1.3.2 <u>Units 3&4 EHP Saddle Dam</u>

As with the Main Dam, available engineer's inspections from 1999, 2005, 2009, and 2014 were reviewed. During the 1999 inspection, several stress cracks were observed near the right (south) abutment of the Saddle Dam coupled with the observation of a significant volume of downstream seepage (Maxim, 1999b, Hydrometrics, 2000). This discovery prompted an investigation and additional monitoring (Hydrometrics, 2000). The investigation report concluded that the cracks were not a threat to embankment stability as there was very little movement detected by instrumentation and that slope stability analyses indicated high factors of safety against failure (Hydrometrics, 2000). However, the operational water level in the adjacent G Cell was ultimately restricted to an elevation of 3237.5 feet to limit the seepage. The Hydrometrics' report recommended continued monitoring of these cracks which occurred until 2012 with little observed change (Hydrometrics, 2014c). In 2012, the cracks were covered with approximately 30 feet of compacted baked shale during the 2012 Phase 2 embankment raise (WAI, 2012). The most recent engineer's inspection report noted no observations of cracking, sloughing, sliding, scarping, erosion, or unusual movement in the crest, upstream slope, or downstream slope of the Saddle Dam embankment and reported good contact between the embankment and abutments. Furthermore, no low areas were noted in the Saddle Dam crest (Hydrometrics, 2014c).

Four inclinometers were installed in the Saddle Dam embankment following the 2012 Phase 2 embankment raise (WAI, 2013). Three of the four inclinometers have reported very little, if any, movement of the Saddle Dam embankment since they were installed. One inclinometer has detected movement within the Saddle Dam embankment significant enough to warrant additional monitoring. As a response, two additional inclinometers were installed in 2015 to further characterize this movement (Jorgensen, 2016a). Following a period of additional monitoring, it was concluded that the movement results from minor secondary settlement in the saturated baked shale foundation, which is a response to an increase in overburden pressure caused by the 2012 embankment raise (Jorgensen, 2016a). The Jorgensen report additionally concludes that the movement is not a cause for concern but should continue to be monitored for changes. Monitoring efforts are on-going and include monthly readings of instrumentation combined with periodic visual inspections.

Additional instrumentation in the Saddle Dam foundation and embankment consists of 19 piezometers (Jorgensen, 2016a). Nine piezometers are located outboard of the foundation cutoff wall while 10 are inboard. Water levels detected by the piezometers have been recorded at least semi-annually since they were installed. Review of piezometer monitoring data shows that the outboard piezometers have all been dry over the monitoring period. Water levels detected by inboard piezometers appear to respond to water levels in unlined cells in the EHP facility, specifically C Cell. The piezometer data indicates that the central core and downstream drainage layers of the Saddle Dam embankment are effectively maintaining a low phreatic surface within the embankment.

A slope stability analysis of the Saddle Dam embankment was recently performed by Jorgensen Geotechnical, LLC (Jorgensen, 2016c). This analysis determined that the Saddle Dam embankment exceeds required factors of safety against failure for a range of loading conditions required by the Coal Ash Rule.

4.2 SLOPE PROTECTION

Adequate slope protection is required to protect against surface erosion, wave action, and adverse effects of sudden drawdown [§257.73 paragraph (d)(1)(ii) requires]. The Main and Saddle Dams do not have low level outlets; therefore, sudden drawdown is not a concern. Upstream slope protection against wave action and surface erosion for both the Main and Saddle Dams was originally provided by a layer of soil-cement installed during Phase 1 embankment construction to an elevation of 3260 feet (Bechtel, 1985). Following the Phase 2 embankment raise to an elevation of 3290 feet, dried paste has been placed on the upstream slopes of the Main and Saddle Dams as a cushion layer for future installation of geosynthetic liner systems in Cells J and G (Hydrometrics, 2014c). A liner is currently being installed in J Cell which will provide upstream slope protection for the Main Dam (Hydrometrics, 2016). G Cell is scheduled to be lined in the future (Geosyntec, 2015), which will provide upstream slope protection for the Saddle Dam. G Cell contains no water in the interim (Hydrometrics, 2016). The downstream slopes of the Main and Saddle Dams, from their respective toes to an elevation of 3260 feet, are protected from surface erosion by a 6-inch layer of topsoil and grass (Bechtel, 1985). Baked shale placed for the Phase 2 embankment raise to an elevation of 3290 feet is bare. However, the coarse, angular baked shale fragments, similar in nature to riprap, provide adequate protection against erosion. The most recent engineer's inspection report noted good vegetative coverage on the grassed portions of the downstream slopes of both the Main and Saddle Dam (Hydrometrics, 2014c). No slope protection issues were noted for the Main Dam. Several rodent holes were noted in the downstream slope of the Saddle Dam embankment as well as the presence of scattered woody brush. The report recommended filling noted holes with soil followed by continued monitoring for burrowing animal activity as well as the removal of woody vegetation. As a response to these recommendations, the woody vegetation was sprayed with herbicide or physically removed by CSES staff and an ongoing rodent control plan was implemented (PPL, 2015).

4.3 SLOPE VEGETATION

Vegetated slopes of dikes shall not exceed a height of six inches above the slope of the dike, except where an alternate form of slope protection has been provided [§257.73 paragraph (d)(1)(iv)]. As stated in Section 2.3 the six inch height requirement has been removed from this rule. The downstream slopes of the Main and Saddle Dams meet this requirement with the combination of grass coverage below elevation 3260 feet and coarse, angular baked shale fragments placed above elevation 3260 feet. The upstream slope of the Main and Saddle Dams will be protected by geosynthetic liner systems in J and G Cells respectively. The J Cell liner is currently being installed and the lining of G Cell is scheduled to occur in the future. G Cell is dry in the interim.

4.4 IMPOUNDMENT CAPACITY

The spillway must be properly configured and have the capacity to manage flow during and following the peak discharge from a specified flood event [§257.73 paragraph (d)(1)(v)]. The Main and Saddle Dams have a significant hazard potential classification. Therefore, the appropriate inflow design flood event for this impoundment is the expected runoff from a 1,000-year precipitation event [specified by paragraph (d)(1)(v)(B)].

There are no spillways for either the Main or Saddle Dam. The EHP facility was originally designed to contain a 24-hour probable maximum precipitation (PMP) event with a concurrent 100-year flood event (Bechtel, 1982) at its current crest elevation of 3290 feet. The flood volume from the design event was predicted to raise the water level from a maximum operating pool of 3280 to 3283.1 feet, leaving almost seven feet of freeboard. The Bechtel design proposed a spillway at an elevation of 3283.1 feet, but this was never constructed. To address an update to the hydrometeorological report (HMR) for this area and to be current with updated regulations, an independent check of flood routing was performed in the 1988 Phase I inspection of the Main and Saddle Dams (Chen-Northern, 1988b). A 72-hour general PMF event was analyzed and resulted in a discharge of only 29 cfs over the proposed spillway at elevation 3283.1 feet. A recent review of this analysis shows that this small excess flood volume would be safely stored in the impoundment in the absence of a spillway given the large amount of excess freeboard provided in the Bechtel design (Hydrometrics, 2014c). This analysis of available data suggests that the 3&4 EHP impoundment has adequate capacity to store the entire volume of a PMF and concurrent 100-year flood event with sufficient remaining freeboard. This exceeds Coal Ash Rule standards for flood routing capacity for impoundments with a downstream hazard potential classification of significant.

4.5 LOW LEVEL HYDRAULIC STRUCTURES

The Main and Saddle Dams do not have low level outlets or any other type of hydraulic structure passing through their respective embankments [§257.73 paragraph (d)(1)(vi)].

4.6 INUNDATED DOWNSTREAM SLOPES

There are no water bodies adjacent to Main or Saddle Dams which could potentially inundate their downstream slopes [§257.73 paragraph (d)(1)(vii)].

4.7 IDENTIFICATION OF STRUCTURAL DEFICIENCIES

No structural stability deficiencies were noted during this initial assessment [§257.73 paragraph (d)(2)].

5.0 STRUCTURAL STABILITY ASSESSMENT CERTIFICATION

CERTIFICATION

I, Adam Jourdonnais, a registered Professional Engineer in the State of Montana, certify that this ***Structural Stability Assessment Report*** for the Colstrip Steam Electric Station was conducted in accordance with the requirements of ***40 CFR 257.73(d)(1) Structural Stability Assessment.*** This certification is made in compliance with the specific requirements of §257.73(d)(3). This certification is based in part on review of reference documentation identified in Section 6 of this report. This structural stability assessment concludes that the design, construction, operation, and maintenance of Units 1&2 STEP Dam, Units 1&2 Bottom Ash Pond Dike, and Units 3&4 Effluent Holding Pond Main and Saddle Dams are consistent with recognized and generally accepted good engineering practices.

MONTANA
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PROFESSIONAL ENGINEER

10-7-2016

Adam W. Jourdonnais, P.E.

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