2026 PERIODIC STRUCTURAL STABILITY ASSESSMENT OF COAL COMBUSTION RESIDUAL (CCR) SURFACE IMPOUNDMENT AT COLSTRIP STEAM ELECTRIC STATION

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2026 PERIODIC STRUCTURAL STABILITY ASSESSMENT OF COAL COMBUSTION RESIDUAL (CCR) SURFACE IMPOUNDMENT AT COLSTRIP STEAM ELECTRIC STATION

1.0 INTRODUCTION

1.1 BACKGROUND

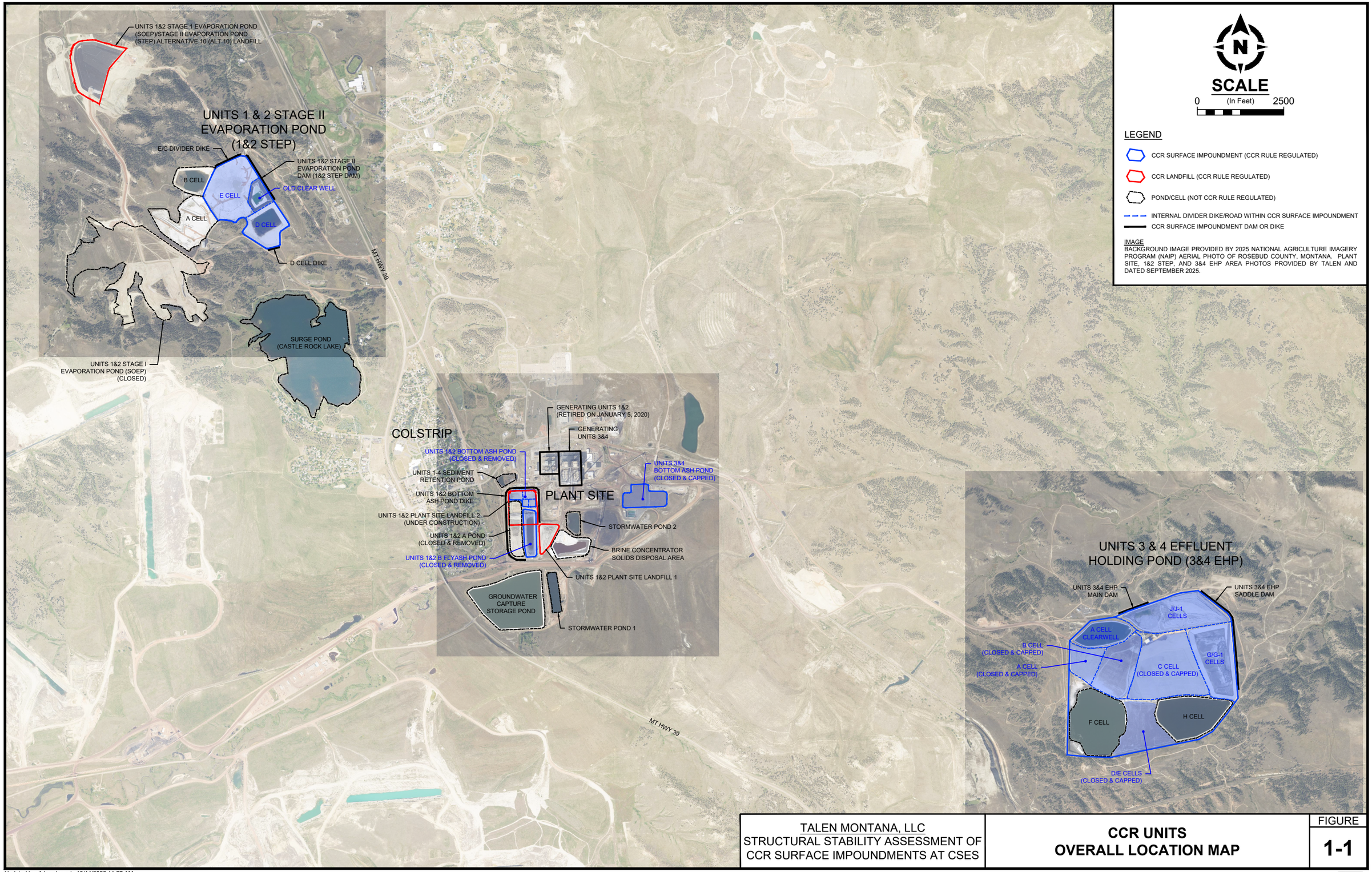
The United States Environmental Protection Agency (EPA) established regulations under Subtitle D of the Resource Conservation and Recovery Act (RCRA) to address the risks associated with disposal of Coal Combustion Residuals (CCRs) generated at electric utilities as well as independent power producers. These regulations became the *Disposal of Coal Combustion Residuals from Electric Utilities* final rule (CCR 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) of the CCR Rule requires the performance of a structural stability assessment for each CCR surface impoundment. A CCR surface impoundment is defined by CCR Rule §257.53 as "…*a natural topographic depression, man-made excavation, or diked area, designed to hold an accumulation of CCR and liquids, and the unit treats, stores, or disposes of CCR*." CCR Rule §257.73 paragraph (f) specifies that the initial structural stability assessment was to be performed by October 17, 2016, with a periodic update due every five years thereafter. The initial structural stability assessment for the CCR surface impoundments at Colstrip Steam Electric Station (CSES) was completed on October 7, 2016 (Hydrometrics, 2016). The last periodic update to the initial structural stability assessment was completed on October 7, 2021 (Hydrometrics, 2021).

Talen Montana, LLC is the operator of the CSES located near Colstrip, Montana. Historically, CSES operated two pairs of coal-fired generating units known as Units 1&2 and Units 3&4. Units 1&2, originally placed in service in 1975, had a generating capacity of 330-megawatts-gross each (Hydrometrics, 2012). Units 1&2 owners include Talen Montana, LLC and Puget Sound Energy Inc. Units 1&2 were retired from service on January 5, 2020. Units 3&4, placed in service in 1983 and 1985, respectively, have a generating capacity of 805-megawatts-gross each (Hydrometrics, 2012). Units 3&4 owners include Talen Montana, LLC, PacifiCorp, Portland General Electric Company, and NorthWestern Energy.

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 CCR surface impoundments on and off the plant site that perform various functions to handle and permanently store CCRs. These CCR surface impoundments and their associated CCR units are regulated under the CCR Rule. Units 1&2 and Units 3&4 have their own respective surface impoundments with cells dedicated for CCR handling and storage. It is also important to note that because Units 1&2 have been retired, CCR surface impoundments associated with Units 1&2 continue to store existing CCRs but no longer accept new CCRs. The CCR surface impoundments at CSES regulated by the CCR Rule include:

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

Incised CCR units are exempt from the requirements of §257.73 paragraph (d)(1). Furthermore, 3&4 BAP was closed and capped with a certified final cover system in 2019 (Geosyntec, 2021). Therefore, the 3&4 BAP will not be further mentioned in this document. Some individual pond cells within the 1&2 STEP and 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 CCR surface impoundment. As will be discussed later in this document, each CCR surface impoundment at CSES is adequately sized to contain or safely route a probable maximum flood (PMF) 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 or 3&4 EHP. Therefore, these internal divider and saddle dikes do not have an impact on the overall stability of their respective CCR surface impoundments. It is also notable, and further discussed in Section 3.0 below, that 1&2 BAP, closed by removal in 2024, and B Pond, closure currently in progress, were impounded by the Units 1&2 Bottom Ash Pond Dike (1&2 BAP Dike). Closure of both ponds included removal of CCR and construction of a CCR compliant landfill, identified as Units 1&2 Plant Site Landfill 2 (Landfill 2), within their former footprint (Geosyntec, 2026a). The locations of each CCR unit and their respective CCR surface impoundments are displayed on Figure 1-1.



TALEN MONTANA, LLC
STRUCTURAL STABILITY ASSESSMENT OF CCR SURFACE IMPOUNDMENTS AT CSES

CCR UNITS
OVERALL LOCATION MAP

FIGURE
1-1

Updated by: AJourdonnais 10/14/2026 11:27 AM
C:\Users\ajourdonnais\Dropbox\Talen - 2026 CCR Unit Assessments\7 - CAD\DWG\2600701h002.dwg [FIGURE 1-1 (SSA)]

Hydrometrics, Inc.

1.2 PURPOSE

The purpose of this document is to provide a periodic structural stability assessment for each CCR surface impoundment at CSES in accordance with the specific requirements of 40 CFR §257.73 paragraph (d)(1) of the CCR Rule. As mentioned above, the initial structural stability assessment for the CSES CCR surface impoundments was completed on October 7, 2016 with a periodic assessment completed on October 7, 2021 as required by 40 CFR §257.73 paragraph (f)(3). The following structural stability assessments will provide documentation that design, construction, operation, and maintenance of each CCR surface impoundment at CSES is consistent with sound and generally accepted engineering practices.

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 BAP 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 the 2026 periodic structural stability assessment of 1&2 STEP Dam in accordance with §257.73 paragraph (d)(1) of the CCR 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 existing 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 shale. Thin lenses of limestone and carbon veinlets were also encountered. The coal seam was encountered on the north abutment at elevation 3290 feet above mean sea level (amsl) 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).

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

EMERGENCY SPILLWAY
CREST ELEVATION: 3274.1 FEET

UNITS 1&2 STAGE II
EVAPORATION POND
E/C DIVIDER DIKE

T2N, R41E, SEC 28
T2N, R41E, SEC 20

UNITS 1&2 STEP - B CELL

CREST ELEVATION: 3278 FEET

UNITS 1&2 STEP - E CELL

OLD CLEARWELL

UNITS 1&2 STAGE II
EVAPORATION POND
DAM (1&2 STEP DAM)

BR 3181

(TO I-94) HIGHWAY 39 (TO COLSTRIP)

BURLINGTON NORTHERN RR

Well

T2N, R41E, SEC 21
T2N, R41E, SEC 28

T2N, R41E, SEC 20
T2N, R41E, SEC 29

UNITS 1&2 STEP - A CELL

UNITS 1&2 STEP - D CELL

T2N, R41E, SEC 28
T2N, R41E, SEC 29

3290 3280 3270 3290 3270 3280 3240 3200 3185 3280 3240 3280 3295 3300

SCALE
(In Feet)
0 500

LEGEND

CCR SURFACE IMPOUNDMENT (CCR RULE REGULATED)

POND/CELL NOT REGULATED BY CCR RULE

INTERNAL DIVIDER DIKE/ROAD SEPARATING CCR UNITS

IMAGE

COLSTRIP WEST USGS 7½ MINUTE QUADRANGLE MAP AND 1&2 STEP AREA AERIAL PHOTO PROVIDED BY TALEN AND DATED SEPTEMBER 2025.

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.

TALEN MONTANA, LLC
STRUCTURAL STABILITY ASSESSMENT OF
CCR SURFACE IMPOUNDMENTS AT CSES

STAGE II EVAPORATION POND DAM LOCATION MAP

FIGURE
2-1

Updated by: AJourdonnais 9/24/2026 3:18 PM
C:\Users\ajourdonnais\Dropbox\Talen - 2026 CCR Unit Assessments\7 - CAD\DWG\2600701h003.dwg [STAGE II DAM]

Hydrometrics, Inc.

2.1.2 Embankment Construction

Included in Bechtel's original design report for 1&2 STEP Dam is a set of "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 (Geosyntec, 2015). The drawings in the design report show that the proposed embankment for 1&2 STEP Dam was to be constructed as 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 zone 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 amsl 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 obtained from designated borrow areas located near the 1&2 STEP Dam. Zone 1 soils were moisture conditioned to a specified range of 0 to +2 percent of optimum and compacted in 12-inch lifts to a minimum 95 percent of maximum dry density as determined by ASTM D1557, 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 amsl results in a maximum height of 88 feet above existing ground level. 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 D1557, 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. Zone 3 soil materials were specified as processed sand and gravel while Zone 5 soil materials were specified as processed sand. Zone 3 and Zone 5 soil materials were required to be sourced from an approved quarry and to meet provided gradations for each. 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 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 Upstream Filter Blanket

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 D1557, 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, which includes the upstream face of 1&2 STEP Dam, has eliminated the need for this filter blanket.

2.1.3 Foundation, Abutment, and Embankment Stability Summary

Prior to the implementation of the CCR Rule regulations, formal engineer's inspections of 1&2 STEP Dam were performed every five years. The 1&2 STEP Dam has been inspected annually since CCR Rule implementation in 2015. A review of available inspection reports from 1999, 2005, 2009, and 2014 through 2025 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, 2014a, and 2021; Jorgensen, 2016a, 2017, 2018, 2019, 2020, 2021a, 2022, 2023, 2024, 2025; Tetra Tech, 2026).

Monitoring instrumentation installed in the 1&2 STEP Dam embankment and foundation includes two slope inclinometers and six vibrating wire piezometers (VWPs). From the time they were installed in 2015, these instruments have been monitored at least semi-annually. The monitoring frequency was increased to monthly in 2015 to comply with CCR Rule requirements (Jorgensen 2016a, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025; Tetra Tech, 2026). Review of inclinometer monitoring data presented in the annual engineer's inspection reports over the past five years shows very little, if any, movement of the embankment on its foundation or abutments over the monitoring period (Jorgensen, 2022, 2023, 2024, and 2025; Tetra Tech, 2026). All piezometers but two have been dry since their installation in 2009. One VWP near the toe of the embankment detected slight amounts of groundwater in June 2011 and again in the spring of 2025 (Jorgensen, 2016a; Tetra Tech, 2026). Another VWP near the embankment crest detected a slight amount of groundwater in the spring of 2025 (Tetra Tech, 2026). These two instruments have been dry since. 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. Under these conditions, the development of an adverse pore water pressure situation within the dam embankment is very unlikely.

A slope stability assessment of the 1&2 STEP Dam embankment, required by the CCR Rule, was performed by Jorgensen Geotechnical, LLC in 2016 and 2021 (Jorgensen, 2016b and 2021b), and by Hydrometrics in 2026 (Hydrometrics, 2026a). The results of these slope stability analyses determined that the 1&2 STEP Dam embankment exceeds required factors of safety against failure for the range of loading conditions required by the CCR 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 a geosynthetic liner system installed in E Cell and the Old Clearwell. The slopes of the Old Clearwell and E Cell are lined with 100-mil high density polyethylene (HDPE) geomembrane while D Cell liner system is comprised of a geocomposite drainage net, a HDPE geomembrane, and a cushion geotextile (Geosyntec, 2015). 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). A recent visual inspection noted that the liner systems on the upstream slope appeared to be in good condition with a good vegetative cover between the liner and dam crest (Hydrometrics, 2026b). The inspection also noted good grass coverage on the downstream slope that appeared to be well maintained. The most recent formal engineer's inspection, performed in 2025, reported that, "no conditions were observed that would indicate an active or impending structural stability concern (Tetra Tech, 2026)."

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 geosynthetic liner and grass cover employed as a 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 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 amsl, 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 of the CCR Rule 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 amsl to an elevation of 3274.6 feet amsl, 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.

Additional analysis, performed in 2024, determined that stormwater resulting from a PMF event with back-to-back 100-year storms would not reach the emergency spillway crest and would be fully contained within the STEP facility in its current configuration (Geosyntec, 2024a). Therefore, available data indicates that the STEP facility has adequate capacity to safely store a full PMF event without need for the additional discharge capacity provided by the emergency spillway.

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 deficiencies were discovered during this periodic assessment [§257.73 paragraph (d)(2)].

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

The following section provides the 2026 periodic structural stability assessment of 1&2 Bottom Ash Pond Dike (1&2 BAP Dike) in accordance with §257.73 paragraph (d)(1) of the CCR Rule. As shown on Figure 3-1 below, 1&2 BAP Dike historically impounded Units 1&2 Bottom Ash Pond (1&2 BAP) and B Flyash Pond (B Pond), which were CCR units, and A Pond which was not a regulated CCR unit. A Pond was closed and drained in 2015 (Hydrometrics, 2015) and capped in 2019 (Geosyntec, 2025a). In 2026 B Pond and 1&2 BAP were drained and stored CCR was removed by excavation in order to construct Landfill 2. It is also notable that the bottom liner and leachate collection system for Landfill 2 has been installed and certified as complete as of August 2026 (Geosyntec, 2026a and 2026b). The 1&2 BAP Dike remains in place as part of the Landfill 2 structure. However, no appreciable amount of liquid is currently impounded by 1&2 BAP Dike as it transitions from a CCR surface impoundment to Landfill 2 (Hydrometrics, 2026b). Once complete, Landfill 2 will be a CCR landfill as defined in §257.53 of the CCR Rule.

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)]. 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 that were “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 (Bechtel, 1974). However, in late 2009 WAI conducted geotechnical investigations of the 1&2 BAP Dike embankment and foundation to fill in data gaps (WAI, 2010b and WAI, 2010c). Soil borings were performed in 1&2 BAP Dike west of form 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 consist 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

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C:\Users\ajourdonnais\Dropbox\Talen - 2026 CCR Unit Assessments\7 - CAD\DWG\26007701h003.dwg [1&2 BAP & B POND]

SCALE
(In Feet)
0 250

LEGEND

- CCR SURFACE IMPOUNDMENT (CCR RULE REGULATED)
- CCR LANDFILL (CCR RULE REGULATED)
- POND/CELL NOT REGULATED BY CCR RULE

IMAGE
COLSTRIP EAST USGS 7½ MINUTE QUADRANGLE MAP AND PLANT SITE AREA AERIAL PHOTO PROVIDED BY TALEN AND DATED SEPTEMBER 2025.

REFERENCE NOTE:
UNITS 1&2 BOTTOM ASH POND DIKE 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.

UNITS 1&2
(RETIRED ON JANUARY 5, 2020)

UNITS 3&4

UNITS 1-4
SEDIMENT
RETENTION POND

UNITS 1&2 BOTTOM
ASH POND DIKE

CREST ELEVATION: 3265 FEET

UNITS 1&2 BOTTOM ASH POND
(CLOSED & REMOVED)

UNITS 1&2 PLANT SITE LANDFILL 2
(UNDER CONSTRUCTION)

STORMWATER POND 2

UNITS 1&2
B FLYASH POND
(CLOSED & REMOVED)

UNITS 1&2 A POND
(CLOSED & REMOVED)

UNITS 1&2 PLANT
SITE LANDFILL 1

BRINE CONCENTRATOR
SOLIDS DISPOSAL AREA

BURLINGTON NORTHERN RR

3240 3245 3250 3260 3265 3270 3280

T2N, R41E, SEC 34

T1N, R41E, SEC 3

TALEN MONTANA, LLC STRUCTURAL STABILITY ASSESSMENT OF CCR SURFACE IMPOUNDMENTS AT CSES	**UNITS 1&2 BOTTOM ASH POND DIKE LOCATION MAP**	FIGURE **3-1**

Hydrometrics, Inc.

Formation (WAI, 2010c). Corrected standard penetration test (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 37 feet below the embankment crest north of the Bottom Ash 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 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 its 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 the 1&2 BAP Dike (WAI, 2010b and 2010c). Overall, the 1&2 BAP Dike is approximately 4,000 linear feet in length with a maximum height of approximately 25 vertical 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 300-foot length along the southeast corner. 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 1&2 BAP Dike. This area is currently the site of Units 1&2 Plant Site Landfill 1, which is a CCR landfill.

The properties of the 1&2 BAP Dike core and shell soil materials are discussed in the following sections. Information regarding the original construction of 1&2 BAP Dike, including earthwork 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 that slope 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 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 1&2 BAP was 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 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). 1&2 BAP Dike has a minimum crest elevation of 3265 feet amsl. Shell soil materials encountered in borings were typically coarser than core soil materials (WAI, 2010b and 2010c). Shell soil materials found in borings west of former A Pond and northwest of 1&2 BAP were largely heterogeneous and ranged from silty clay to gravelly sand (WAI, 2010b). Shell soil materials found in borings north and east of 1&2 BAP 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 CCR Rule regulations, formal engineer's inspections of 1&2 BAP Dike were performed every five years. Engineer's inspections have been conducted annually since 2015. Available inspection reports from 2009 and 2014 through 2025 reported no observation of unusual movement of 1&2 BAP Dike on its foundation (Hydrometrics, 2009a, 2014a, and 2021; Jorgensen, 2016a, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025; Tetra Tech, 2026). 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. As mentioned previously, it is also of note that 1&2 BAP Dike no longer impounds any appreciable volume of liquid.

Monitoring of instrumentation installed in the 1&2 BAP Dike embankment and foundation, which currently includes three VWPs, was conducted at least semi-annually from the time the instruments were installed until 2015. 1&2 BAP Dike had six piezometers for a period; however, one was destroyed during the capping of A Pond in November 2019, another experienced equipment malfunction first noted in 2017 (Jorgensen, 2018 and 2020), and a third was destroyed during Landfill 2 construction. From 2015 to present the instruments have been monitored on a monthly basis to comply with CCR Rule requirements (Jorgensen, 2016a, 2017, 2018, 2019, 2020, 2021a, 2022, 2023, 2024, 2025; Tetra Tech, 2026). In general, monitored piezometers located in the 1&2 BAP Dike have been observed to be dry. A single instance of groundwater detection occurred in a piezometer in May 2025. However, it was reported as an anomaly that was likely caused by operator or instrument error (Tetra Tech, 2026). Piezometer data indicates that the 1&2 BAP Dike's central core is effective in maintaining a low phreatic surface. Given that 1&2 BAP Dike no longer impounds any significant quantity of liquid, there is an extremely low potential for development of adverse pore water pressure conditions that could pose a threat to embankment stability.

Slope stability assessments of the 1&2 BAP Dike, required by the CCR Rule, were performed by Jorgensen Geotechnical, LLC in 2016 and 2021 (Jorgensen, 2016b and 2021b) and Hydrometrics, Inc. in 2026 (Hydrometrics, 2026a). The results of these slope stability analyses have determined that 1&2 BAP Dike exceeds required factors of safety against failure for the range of loading conditions required by the CCR 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 1&2 BAP Dike has transitioned over time. The upstream slopes were protected by six inches of topsoil and vegetation following original construction (Bechtel, 1974). Since that time, various geosynthetic liner systems have been installed along the upstream slope of 1&2 BAP Dike which provide protection against surface erosion and 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 west side of 1&2 BAP Dike was regraded in 2019 to construct a drainage ditch that conveys stormwater runoff to Units 1-4 Sediment Retention Pond. Following construction of the ditch, the disturbed areas were regraded to the original 2H:1V slope and revegetated. During a recent visual inspection of the 1&2 BAP Dike area it was noted that the downstream embankment slope has a good grass cover (Hydrometrics, 2026b). It was also noted that a portion of the embankment on the north side of 1&2 BAP Dike had been disturbed by Landfill 2 construction activities (Hydrometrics, 2026b). It is our understanding that this area will be regraded and vegetated as part of Landfill 2 construction.

3.3 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)]. As stated in Section 2.3 above, the six-inch height requirement has been removed from this rule. 1&2 BAP Dike meets this requirement with the grass covered downstream slopes and geosynthetic lined upstream slopes discussed in the previous section.

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 low downstream hazard potential classification (Hydrometrics, 2026c), the required inflow design flood event for this CCR surface impoundment is a 100-year flood [§257.73 paragraph (d)(1)(v)(B)(3)].

1&2 BAP Dike has no spillways, outlets, or other means of rapidly lowering water levels. Water and CCR waste levels have been historically maintained by pumping. The historic maximum operating level for B Pond and 1&2 BAP was 3260 feet amsl, which provided at least five feet of freeboard below the minimum embankment crest elevation of 3265 feet amsl. Run-on is diverted away from the 1&2 BAP Dike area, leaving only precipitation falling within the footprint of the ponds as the only water received during a given storm event. Because 1&2 BAP Dike previously had a downstream hazard potential classification of high, the general and local PMF events were analyzed (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 by 2.42 feet to elevation 3262.42 amsl which leaves over 2.5 feet of freeboard before overtopping of 1&2 BAP Dike would occur. It is also important to note that the current transition of the 1&2 BAP Dike area from a CCR surface impoundment, impounding liquid and CCR, to a CCR landfill, storing dry CCR waste, will result in a reconfiguration of stormwater management infrastructure. The stormwater controls for Landfill 2 are presented in the design documents (Geosyntec, 2026b). Analysis of Landfill 2 stormwater controls was conducted for a PMF event, which has a much greater magnitude than the 100-year storm event required by the CCR Rule, and determined that all generated runoff will be contained within the CCR site (Geosyntec, 2024a). Therefore, 1&2 BAP Dike meets CCR Rule requirements for stormwater capacity.

3.5 LOW LEVEL HYDRAULIC STRUCTURES

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 deficiencies of 1&2 BAP Dike were noted during this periodic assessment [§257.73 paragraph (d)(2)].

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

This section provides the 2026 periodic 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 CCR 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

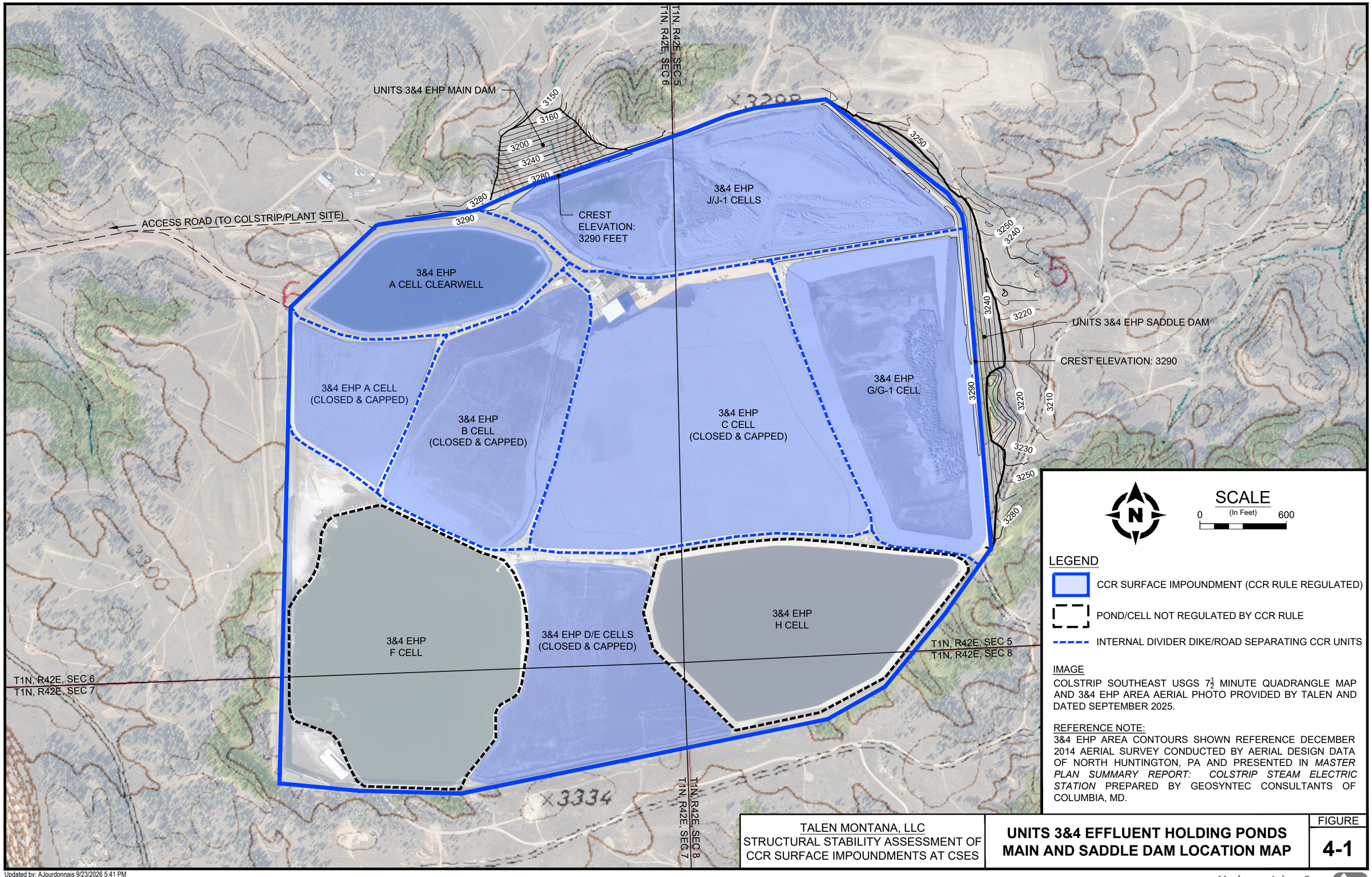
Stability of the Main and Saddle Dams require stable foundations and abutments [§257.73 paragraph (d)(1)(i)] and embankments that have been mechanically compacted to a density sufficient to withstand a range of loading conditions [§257.73 paragraph (d)(1)(iii)]. Stability of the foundation, abutments, 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 the Main Dam during the design process (Bechtel, 1982). Additionally, the results of Bechtel's subsurface investigation for design reasonably agree with the findings of WAI's 2009 geotechnical investigation of the Main Dam (WAI, 2010d). Below elevation 3165 amsl, the foundation material for the 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 in thickness to an approximate top elevation of 3200 feet amsl 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 maximum depth of approximately 28 feet in the valley bottom along the Main Dam alignment.



IMAGE

COLSTRIP SOUTHEAST USGS 7½ MINUTE QUADRANGLE MAP AND 3&4 EHP AREA AERIAL PHOTO PROVIDED BY TALEN AND DATED SEPTEMBER 2025.

REFERENCE NOTE:

3&4 EHP 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.

TALEN MONTANA, LLC
STRUCTURAL STABILITY ASSESSMENT OF CCR SURFACE IMPOUNDMENTS AT CSES

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

FIGURE
4-1

Updated by: AJourdonnais 9/23/2026 5:41 PM
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Hydrometrics, Inc.

The upper third of the left and right dam abutment material for the Main Dam embankment is composed of a baked shale unit beginning at an approximate elevation of 3220 feet amsl. 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 consists 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 Units 3&4 EHP Saddle Dam

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 Saddle Dam geotechnical investigations 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 the bedrock layer is a 10-foot-thick seam of coal belonging to 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 amsl in the north and 3225 feet amsl in the south. The baked shale unit 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 design and construction 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 a 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 amsl. The second phase of construction, conducted in 2012, raised the Main and Saddle Dam embankments to a final elevation of 3290 feet amsl (WAI, 2012). The following sections summarize the materials and methods used to construct the Main and Saddle Dam embankments.

4.1.2.1 Foundation Preparation

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 and 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 amsl, 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 amsl 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 Central Core

Bechtel's as-built drawings show the central cores of both the Main and Saddle Dams having top widths of 33.4 feet at an elevation of 3256 feet amsl that 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 was excavated a minimum of two feet into bedrock and five feet 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 existing valley bottom. 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 modifications were made to the central core of either dam during the 2012 embankment raise (WAI, 2012).

4.1.2.3 Exterior Shell

Bechel'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 amsl and were later raised to a final elevation of 3290 feet amsl. The final crest elevation of 3290 feet amsl results in respective maximum embankment heights of 138 feet for the Main Dam and 66 feet for the Saddle Dam. The Zone 2 soil materials used to construct the exterior shells below elevation 3260 feet amsl 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 exterior 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 amsl (WAI, 2012). Bottom ash was placed over existing upstream slopes below an elevation of 3260 feet amsl to serve as a filter blanket and to maintain the 3H:1V slope during the embankment raise (WAI, 2012).

4.1.2.4 Drainage Layers

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

4.1.3 Foundation, Abutment, and Embankment Stability Summary

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

4.1.3.1 Units 3&4 EHP Main Dam

Prior to implementation of CCR Rule regulations, formal engineer's inspections of the Main Dam were performed every five years. Beginning in 2015 the Main Dam has been inspection annually to comply with CCR Rule requirements. A review of the available inspection reports from 1999, 2005, 2009, and 2014 through 2025 revealed no observations of unusual movement of the Main Dam embankment on its foundation (Maxim, 1999b and 2006b; Hydrometrics, 2009c, 2014c, and 2021; Jorgensen 2016a, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025; Tetra Tech, 2026). These inspection reports have 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 currently includes four slope inclinometers, six standpipe piezometers, and 13 VWPs. From the time of installation until 2015 these instruments were monitored at least semi-annually. In 2015 the monitoring frequency was increased to monthly to comply with CCR Rule requirements. Review of the inclinometer monitoring data presented in annual inspection reports from the past five years shows very little, if any, movement of the Main Dam embankment on its foundation or abutments over the monitoring period (Jorgensen 2022, 2023, 2024, and 2025; Tetra Tech, 2026). Review of piezometer data shows that groundwater levels within the Main Dam embankment have changed very little over the monitoring period and indicates that the central core and downstream drainage layers, as well as the J/J-1 Cell liner system, are effective at maintaining a low phreatic surface within the embankment.

Slope stability analyses of the Main Dam embankment were performed by Jorgensen Geotechnical, LLC in 2016 and 2021 (Jorgensen, 2016c and 2021c), and by Hydrometrics in 2026 (Hydrometrics, 2026a). These analyses determined that the Main Dam embankment exceeds required factors of safety against failure for a range of loading conditions required by the CCR Rule.

4.1.3.2 Units 3&4 EHP Saddle Dam

As with the Main Dam, available formal engineer's inspection reports from 1999, 2005, 2009, and 2014 through 2025 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 of the embankment detected by existing instrumentation and that slope stability analyses indicated high factors of safety against failure (Hydrometrics, 2000). However, the operational water level in adjacent G Cell was ultimately restricted to an elevation of 3237.5 feet amsl 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). A recent visual inspection noted no observations of potential structural weakness, including cracking, subsidence, uncontrolled seepage on the crest, upstream slope, or downstream slope of the Saddle Dam embankment (Tetra Tech, 2026). The Tetra Tech report did note minor surficial erosion of the Saddle Dam crest road but noted that it was typical for earthen embankments and would not impact the structural performance of the embankment (Tetra Tech, 2026).

Saddle Dam monitoring instrumentation includes four standpipe piezometers, 15 VWPs, and six slope inclinometers. Four of the slope inclinometers were installed in the Saddle Dam embankment following the 2012 Phase 2 embankment raise (WAI, 2013). During early monitoring of these instruments, three of the four new inclinometers reported very little, if any, movement of the Saddle Dam embankment since they were installed. However, one inclinometer 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 resulted from minor secondary settlement in the saturated baked shale foundation, which was a response to an increase in overburden pressure caused by the 2012 embankment raise (Jorgensen, 2016a). The

Jorgensen report additionally concluded that the movement was not a cause for concern but should continue to be monitored for changes. Monitoring efforts have been on-going over the past five years and include monthly readings of instrumentation combined with periodic visual inspections (Jorgensen, 2022, 2023, 2024, and 2025; Tetra Tech, 2026). Very little, if any, movement of the Saddle Dam embankment on its foundation was observed over the monitoring period.

Additional instrumentation in the Saddle Dam foundation and embankment consists of four standpipe piezometers and 15 VWPs. Ten piezometers are located outboard of the foundation cutoff wall while nine are inboard. From installation until 2015, the piezometers were monitored at least semi-annually. Beginning in 2015, the piezometers were monitored monthly to comply with CCR Rule requirements. Review of piezometer over the last five years shows that groundwater levels inboard of the Saddle Dam embankment cutoff wall have generally decreased. This decrease is to be expected as the former G Cell, located adjacent to the Saddle Dam, remained dry for several years. Additionally, G Cell was closed in 2024 and its former footprint was regraded and lined to construct G-1 Cell (Geosyntec, 2024b and 2025b). Only one of the ten piezometers located outboard of the embankment cutoff wall has observed measurable groundwater over the past five years. The groundwater level in this piezometer was notably consistent over the monitoring period. Piezometer data from the last five years shows that a low phreatic surface is being maintained inboard of the cutoff wall and that the cutoff wall itself is functioning as intended as outboard piezometers are predominantly dry. This strongly suggests that development of adverse pore water conditions in the Saddle Dam embankment is highly unlikely.

Slope stability analyses of the Saddle Dam embankment were performed by Jorgensen Geotechnical, LLC in 2016 and 2021 (Jorgensen, 2016b and 2021b), and by Hydrometrics in 2026 (Hydrometrics, 2026a). These analyses determined that the Saddle Dam embankment exceeds required factors of safety against failure for the range of loading conditions required by the CCR 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)]. 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 soil-cement installed during Phase 1 embankment construction to a crest elevation of 3260 feet amsl (Bechtel, 1985). Following the Phase 2 embankment raise to a crest elevation of 3290 feet amsl, dried paste was placed on the

upstream slopes of the Main and Saddle Dams as a cushion layer for future installation of geosynthetic liner systems in J-1 and G-1 Cells (Hydrometrics, 2014c). The J-1 Cell liner was competed in 2016 and currently provides upstream slope protection for the Main Dam. The G-1 Cell liner was completed in 2025 and currently provides upstream slope protection for the Saddle Dam. The lower portion of the downstream slopes of the Main and Saddle Dams, from their respective toes to elevation 3260 feet amsl, 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 from elevation 3260 amsl to elevation of 3290 feet amsl, along with a fair cover of vegetation, predominantly composed of grass and wild mustard, provide erosion protection for the upper portion of the downstream slopes of the Main and Saddle Dams. The coarse, angular baked shale fragments, similar in nature to riprap, provide adequate protection against erosion whether vegetated or not. A recent visual inspection noted good vegetative coverage on the grassed lower portions of the Main and Saddle Dam downstream slopes and increasing vegetative coverage of the baked shale on the upper portions of the downstream slopes (Hydrometrics, 2026b).

4.3 SLOPE VEGETATION

Vegetated slopes of embankments shall not exceed a height of six inches above the slope of the embankment, 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 amsl and coarse, angular baked shale fragments and vegetation above elevation 3260 feet amsl. As mentioned in Section 4.2, the upstream slopes of the Main and Saddle Dams are protected by geosynthetic liner systems.

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 downstream potential hazard classification of significant. Therefore, the appropriate inflow design flood event for this CCR surface impoundment is the expected runoff from a 1,000-year precipitation event [§257.73 paragraph (d)(1)(v)(B)(2)].

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 amsl. The flood volume from the design event was predicted to raise the water level from a

maximum operating pool of 3280 feet amsl to 3283.1 feet amsl, leaving almost seven feet of freeboard. The Bechtel design proposed a spillway at elevation 3283.1 feet amsl, but this was never constructed. To address an update to the HMR for this area, and to be current with updated regulations, an independent check of the flood routing was performed in the 1988 Phase I inspection of the Main and Saddle Dams (Chen-Northern, 1988). A 72-hour general PMF event was analyzed and resulted in a discharge of 29 cubic feet per second over the proposed spillway at elevation 3283.1 feet amsl. A 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 CCR Rule requirements for flood routing capacity for CCR surface 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 bodies of water adjacent to the Main or Saddle Dams which could potentially inundate their downstream slopes [§257.73 paragraph (d)(1)(vii)].

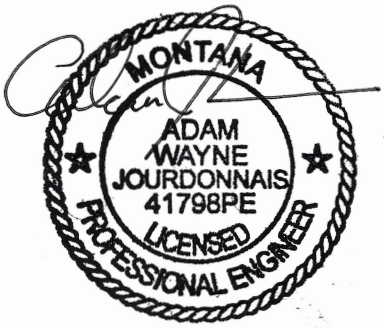
4.7 IDENTIFICATION OF STRUCTURAL DEFICIENCIES

No structural deficiencies were noted during this periodic 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 based, in part, on review of reference documentation identified in Section 6.0 of this report. This structural stability assessment concludes that the design, 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 engineering practices.



10/15/2026

Adam W. Jourdonnais, P.E.

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