April 24, 2024

Ashley Garrison
Colorado Water Conservation Board
1313 Sherman Street, Room 718
Denver, CO 80203
Ashley.garrison@state.co.us

RE: Bolts Lake Reservoir Preliminary Design Pay Request No. 2 and Final Report
CWCB Contract Number POGG 2023-6461

Dear Ashley Garrison,

Enclosed is Pay Request No. 2 for the costs associated with the Bolts Lake Reservoir Preliminary Design Project. This Pay Request includes work from June 2023 to the present. The preliminary design and associated documents are completed and are discussed in the attached Final Report. The following is a summary of all current project charges:

Current Project Charges	$ 458,044.77
Previous Project Charges	$ 827,175.87
TOTAL	$ 1,285,220.64
CWCB Grant	$ 239,833.00
Less Previous Payments	$ 25,625.96
AMOUNT OF THIS REQUEST	$ 214,207.04

We request payment for the amount indicated above. We have included the invoices related to the requested payment identified above. If you have any questions, please call me at (970) 471-1152 or JHildreth@erwsd.org.

Sincerely,

Justin Hildreth

Justin Hildreth, PE
Water Resources Engineer

Attachments:
CWCB Progress Report
Preliminary Design Report, March 2024
Preliminary Design Plans, March 06, 2024
Preliminary Design Specifications Summary
2022/2023 Hydrogeologic Field Investigation Results and Groundwater Model Update – FINAL, December 27, 2023
BCF Borrow Report Site Evaluation, December 2023
Geotechnical Data Report, December 2023
Hydraulic Report, February 2024
30% Design – Cost Estimate Bolts Lake, February 16, 2024
Consultant Invoices

PROGRESS REPORT – JUNE 01, 2023, TO APRIL 1, 2024

Bolts Lake Preliminary Design

PAY REQUEST NO. 2 AND FINAL REPORT

Project cost expended during this pay period is summarized below:

Shannon & Wilson, Inc.	Amount
Professional Services Rendered Through 9/30/2023, Invoice 142507 dated 10/27/2023	$168,485.16
Professional Services Rendered Through 12/30/2023, Invoice 144440 dated 1/11/2024	$134,571.26
Professional Services Rendered Through 04/06/2024, Invoice 146557 dated 4/12/2024	$154,988.35
Current Project Charges	**$458,044.77**

Final Report and Project Update:
The project team completed the preliminary design for the Bolts Lake reservoir project. Bolts Lake is a 1200 Acre-Foot reservoir planned on District property south of Minturn, situated between Maloit Park and Tiguan Road (Figure 1). Ben Bolt constructed a reservoir around 1890 at this location which is a natural basin carved out by glaciers. Ben Bolt filled the reservoir with water diverted from Cross Creek and used it for recreation and fishing. Eventually, Empire Zinc Company, the operators of the Eagle Mine, purchased the property and operated the reservoir until 1996. The State of Colorado required its breach because it was classified as a high-hazard dam that did not meet the State's design and construction standards.

Shannon and Wilson prepared a feasibility study for the construction of the Bolts Lake reservoir and dam. Accompanying the report were conceptual level drawings for the dam, reservoir, Eagle River diversion and pipeline delivery system, as well as a conceptual level cost estimate. Upon completion of the feasibility, the District started working on the preliminary design in 2022 and was completed in February 2023. The CWCB grant application identified the following tasks and a summary of how each major task was completed is as follows:

- TASK 2, FIELD INVESTIGATIONS: The field investigations were completed in the fall and summarized in the BCF Borrow Site Evaluation and the Geotechnical Data Report. The field investigations included on-site soil boring and testing and evaluating the suitability of the clay soils at a potential clay borrow site in Wolcott, CO.

- TASK 3, DIVERSION AND DELIVERY SYSTEM ALTERNATIVES EVALUATION: Shannon and Wilson evaluated diversion and delivery system alternatives and recommended two delivery sources. The primary diversion will be the Bolts Lake Intake and Delivery Ditch, situated on the United States Forest Service (USFS) property and partially in Holy Cross Wilderness. The second delivery system will be a diversion from the Eagle River and pumped up to the reservoir site. The conceptual design contemplated a gravity flow ditch from the Eagle River but the length of the diversion, technical challenges, and environmental impacts are significantly higher than a pump station.

- TASK 4 SPILLWAY AND OUTLET WORKS DESIGN: Shannon and Wilson completed the preliminary design of the spillway and outlet works for the Bolts Lake Reservoir which is included in the Preliminary Design report.

- TASK 5, PRELIMINARY DESIGN REPORT: The preliminary design report was completed in February 2024.

- TASK 6, PRELIMINARY PLANS AND SPECIFICATIONS: The preliminary plans and specifications were completed in February 2024.

846 Forest Road Vail, Colorado 81657 Tel (970) 476-7480 Fax (970) 476-4089 erwsd.org

- TASK 7 ENVIRONMENTAL PERMITTING: District Staff and the design team held initial discussions and meetings on permit requirements, with Federal and State regulatory agencies.

- TASK 8, FEASIBILITY LEVEL COST ESTIMATE: Upon completion of the preliminary design, KMC Construction Consulting completed the preliminary design estimate of probable cost.

- TASK 9: PROJECT MANAGEMENT AND MEETINGS: Monthly project management meetings with District, Shannon & Wilson, and LRE Engineers staff to discuss recently completed work, future required work, site tours, permitting requirements, scheduling, and expenses.

- TASK 10: PROJECT ADMINISTRATION AND MANAGEMENT: District Staff attended regular monthly project meetings, paid invoices, and prepared reports to the District Boards and CWCB.

<u>Project Obstacles</u>
The preliminary design process identified several concerns that impact the reservoir design. The design issues and their resolution are summarized below:

Water source.
The 2022 feasibility study completed identified two potential sources of water to fill the reservoir, Cross Creek using the historical Bolts ditch diversion and a new gravity-fed ditch from the Eagle River. The Bolts ditch diversion is the most cost-effective method to fill the reservoir and will be the primary source. The feasibility study identified several concerns about the reliability of the ditch because of the difficult topography, and its location within the Holy Cross Wilderness Area which limits the equipment and materials that we use to operate and maintain the ditch. As a result, a secondary water source is required to ensure reliability and the feasibility study identified a gravity-fed ditch from the Eagle River. The preliminary design process determined that the ditch would be expensive to construct because of the following three issues:

- topography constraints,
- impacts on existing wetlands,
- Union Pacific Railroad crossing.

As a result, the Eagle River diversion is being changed to a pump station. The pump station has a shorter length and reduced disturbance area, will not impact the existing wetlands, and will not have to cross the Union Pacific Railroad.

Reservoir Liner
The geotechnical studies determined the existing soils at the reservoir site are highly permeable and a reservoir will need a liner system to adequately retain water. Generally, there are 2 types of liner systems, clay and geosynthetic. Clay material is not available at the site and will have to be trucked in. The District-owned property in Wolcott, 25 miles from the reservoir site, has clay material that can be used to line the reservoir. Once the District hires a contractor during the final design process, the selection of a liner system will involve a thorough evaluation of each option for cost-effectiveness, long-term durability, and water permeability.

<u>Meeting Summaries</u>
The design team and staff met on the first Wednesday of each month to discuss the project, assign tasks, and problem-solve design issues. Staff updated the District and Authority Boards at each monthly meeting on the progress of the design. The district staff presented a formal presentation to the District and Authority Boards every quarter. In addition, Staff updated the Town of Minturn and Town of Avon Town Councils several times on the progress of the preliminary design. The District Manager, Siri

Roman, presented the project to the U.S. House Committee on Natural Resources Subcommittee on Federal Lands regarding the project and the Bolts Ditch Act. The Bolts Ditch Act is a bill that seeks to amend the John D. Dingell, Jr. Conservation, Management, and Recreation Act to allow for additional entities to be eligible to complete the maintenance work on the Bolts Ditch headgate within the Holy Cross Wilderness, Colorado. The additional entities are the Eagle River Water and Sanitation District and the Upper Eagle Regional Water Authority.

Engineering Reports

The engineering reports are included with this Final Report submittal.

SUBMITTED TO:
Eagle River Water & Sanitation District/Upper Eagle Regional Water Authority
846 Forest Road
Vail, Colorado 81657

BY:
Shannon & Wilson, Inc.
5900 W. 38th Avenue
Wheat Ridge, Colorado 80212

(303) 825-3800
www.shannonwilson.com

PRELIMINARY REPORT

Bolts Lake Reservoir Preliminary Design

EAGLE COUNTY, COLORADO

March 2024

Shannon & Wilson No: 109771-001

SHANNON & WILSON

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Submitted To: Eagle River Water & Sanitation District/Upper Eagle Regional Water Authority
846 Forest Road
Vail, Colorado 81657
Attn: Jason Cowles, PE

Subject: PRELIMINARY REPORT, BOLTS LAKE RESERVOIR PRELIMINARY DESIGN, EAGLE COUNTY, COLORADO

Shannon & Wilson prepared this report and participated in this project as a consultant to the Eagle River Water & Sanitation District/Upper Eagle Regional Water Authority (the District). Our scope of services was specified in Contract Number 22.15.057 with the District, dated July 11, 2022. This report presents our preliminary design for the construction of a dam and reservoir at the subject site. The report was prepared by the undersigned as part of the authorized scope of services.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

Matthew T. Grizzell
Associate

Gregory R. Fischer, Ph.D., P.E.
President

MTG:GRF/jma/ajg

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ACRONYMS

8140 Partners	8140 Partners, LLC
AlpineEco	Alpine Ecological Resources, LLC
BLR	Bolts Lake Reservoir
BCF	Biosolids Containment Facility
CCR	Code of Colorado Regulations
CDPHE	Colorado Department of Public Health and the Environment
cfs	cubic feet per second
cm/s	centimeters per second
USACE	U.S. Army Corps of Engineers
CPW	Colorado Parks and Wildlife
CTP	Consolidated Tailings Pile
EA	Environmental Assessment
EAP	Emergency Action Plan
EIS	Environmental Impact Statement
EPA	U.S. Environmental Protection Agency
EPAT	Extreme Precipitation Analysis Tool
HDPE	high density polyethylene
LRE	LRE Water, Inc.
MCL	Maximum Contaminant Levels
NEPA	National Environmental Policy Act
OTP	Old Tailings Pile
OU3	Operable Unit 3 of the Eagle Mine Superfund Site
PMP	probable maximum precipitation
psi	pounds per square inch
SEO	Colorado Division of Water Resources State Engineer's Office
UCS	Unconfined Compressive Strength
USFWS	U.S. Fish and Wildlife Service
WSE	Water Surface Elevation

1 INTRODUCTION AND SCOPE OF WORK

This report presents the results of our preliminary design study for a new water supply reservoir and dams at the Bolts Lake Reservoir (BLR) site for the Bolts Reservoir Preliminary Design and Bolts Ditch Diversion and Pipeline Delivery System (the Project). This report was completed as part of our scope of work outlined in Contract Number 22.15.057 with the Eagle River Water & Sanitation District/Upper Eagle Regional Water Authority (the District), dated July 11, 2022. Separately, we have provided preliminary-level drawings for the dam, reservoir, and Bolts Ditch diversion and pipeline delivery system, as well as a preliminary-level cost estimate.

2 RELEVANT STUDY DOCUMENTS

Shannon & Wilson, LRE Water (LRE) and Alpine Ecological Resources (AlpineEco) prepared several documents for the Project. These documents are listed below and included in the references section of this report.

- Phase II Environmental Site Assessment (Shannon & Wilson, 2021a),
- Feasibility Study Geotechnical Data Report (Shannon & Wilson 2021b),
- Draft Dam Type Alternatives Report (Shannon & Wilson, 2021c),
- Borrow Source Study (Shannon & Wilson, 2021d),
- Wetlands and Water Feature Mapping (AlpineEco, 2021),
- Bolts Reservoir Due Diligence Groundwater Model Report (LRE, 2021),
- High Groundwater Mitigation Alternatives Memorandum (LRE, 2022a),
- Cross Creek Diversion Memorandum (LRE, 2022b),
- 2022/2023 Hydrogeologic Field Investigation Results and Groundwater Model Update Report (LRE, 2023a),
- Permitting Alternatives and Delivery Matrices (LRE, 2023b),
- Bolts Lake Firm Yield Analysis (2023c),
- Preliminary Design Geotechnical Data Report (GDR) (Shannon & Wilson, 2023a),
- BCF Borrow Site Evaluation Report (Shannon & Wilson, 2023b),
- Hydrology and Hydraulics Report (Shannon & Wilson, 2024).

In addition, we relied on previous studies by us and others, which are also cited in the references section of this report.

This report supersedes our 2022 Feasibility Report (Shannon & Wilson, 2022a).

3 SITE AND PROJECT HISTORY

3.1 Existing Dam and Reservoir

The BLR site is located approximately 2.5 miles south of Minturn (see Figure 1). The BLR was initially developed as a reservoir in the 1880s to provide recreation and water storage for the surrounding communities. The existing, abandoned BLR Dam was built at a relatively narrow water gap in the naturally occurring glacial moraine extending around the north and east sides of the BLR basin. The BLR was filled via a sluice diverting water from Cross Creek into the Bolts Ditch and the BLR.

Because of dam safety concerns, the Colorado Division of Water Resources (CDWR) State Engineer's Office (SEO) required lowering of the reservoir in 1992. Site observations suggest that the reservoir was drained via a trench excavated in native ground adjacent to the dam. The sluice at Cross Creek was abandoned in 2010 at the direction of the U.S. Department of Justice and the U.S. Forest Service.

In 2021, Shannon & Wilson and LRE Water completed a study on behalf of the District to evaluate the feasibility of developing the BLR site as a water supply reservoir (Shannon & Wilson, 2021a, b, c and d; Shannon & Wilson, 2022b). Based on the results of the feasibility study, the District purchased the BLR property in March 2022 to develop the reservoir. This report summarizes the additional information collected by the Shannon & Wilson Team to support preliminary design of the BLR.

3.2 Site History and Ownership

The BLR was formerly a part of a larger property owned by Battle North, LLC (Battle North), referred to as the "North Property." We understand that Battle North intends to develop parts of the North Property adjacent to the BLR with parks and trails.

The BLR is located adjacent to the abandoned Eagle Mine, a mining and milling complex located between Red Cliff and Minturn Colorado. The Eagle Mine produced lead, zinc, gold, silver, and copper from about 1905 to 1984. Due to mining impacts, including metal contamination in soils, surface water and groundwater, the U.S. Environmental Protection Agency (EPA) designated the Eagle Mine as a Superfund site (the Eagle Mine Site) in 1986 and placed it on the National Priorities List.

The North Property currently includes Operable Unit 3 (OU3) of the Eagle Mine Site. A 2006 Remedial Investigation report (ERM, 2006) determined that the BLR was not impacted by contaminants from the Eagle Mine and the BLR is therefore no longer included as a remedial area in OU3. Additional soil, groundwater, and surface water analytical results completed by Shannon & Wilson in 2021 are included in a Phase II ESA report (Shannon & Wilson, 2021a).

Additional detail regarding mining operations and remedial actions on the greater North Property can be found in the ERM Remedial Investigation report (ERM, 2006), the ERM Remedial Investigation Report Addendum (ERM, 2011), and the EPA Final Record of Decision (ROD) for OU3 (EPA, 2017).

4 PROJECT REQUIREMENTS

The District purchased the BLR site from Battle North, LLC to develop it as a water supply reservoir by constructing an earth-fill embankment dam near the existing, abandoned BLR Dam, along with a smaller saddle dam to achieve a higher reservoir pool elevation (and additional storage). The District desires a minimum reservoir size of 1,200 acre-feet. To achieve this storage goal, we developed a preliminary reservoir design with a water surface elevation (WSE) of 8150 feet and dam crest elevations of 8153 feet (to account for 3 feet of freeboard).

Based on yield studies and modeling by LRE (2023c) and conversations with the District, we understand that the BLR will be filled with two sources:

- A main source consisting of a rehabilitated Cross Creek Diversion and a new gravity fed ditch and pipeline alignment that will approximately follow the existing Bolts Ditch alignment, and
- A supplemental water source consisting of a pump station adjacent to the BLR that will draw water from the Eagle River.

We understand that the District desires seepage loss to be as low as practical given the potentially limited yield from the Cross Creek Diversion in dry years. During the feasibility study, the District set an annual seepage loss at 5% of the total capacity. A reservoir liner will be required to achieve this seepage loss target as discussed herein.

The work for this preliminary design did not include the following elements, among other tasks associated with final design:

- An Emergency Action Plan (EAP).
- Design of the inlet and outlet works.

- Detailing of liner to core transitions at dam abutments.
- Diversion or pump station outlet works.
- Instrumentation Plan.

5 REGIONAL GEOLOGY AND GEOMORPHOLOGY

The BLR site is located near the confluence of Cross Creek and the Eagle River on the east flank of the Sawatch Mountain Range. In general, local geology at the site is characterized by glacial sediments overlying northeast-dipping sedimentary bedrock units. Kirkham and others (2012) indicate that these deposits include late Pleistocene (on the order of 12,000- to 2.6-million-year-old) glacial till on the ridges surrounding the BLR site and late Pleistocene to Holocene (less than about 12,000-year-old) glacial moraine-dammed sediments in the BLR topographic basin.

Based on geologic mapping of the area (Tweto and Lovering, 1977; Kirkham and others, 2012), bedrock at the BLR site likely consists of Lower Mississippian-age (approximately 345- to 360-million-year-old) Leadville Dolomite and Lower Mississippian to Devonian-age (approximately 345- to 385-million-year-old) rocks of the Chaffee Group, including (from top to bottom) the Gilman Sandstone, Dyer Dolomite, and Parting Formation. Geologic mapping indicates that Ordovician to Upper Cambrian-age (approximately 500- to 445-million-year-old) rocks of the Harding Sandstone, Manitou Formation, and Peerless Formation may be present in the subsurface below the BLR site.

Kirkham and others (2012) mapped two inferred fault traces near the BLR site, including a fault trace generally paralleling the bottom of Cross Creek southwest of the BLR and a second fault trace generally paralleling the course of the Eagle River to the south of the BLR. Both fault traces are mapped as terminating under surficial soil deposits on the OTP and do not extend into the proposed BLR Dam footprint. As discussed in Section 7, these mapped faults are geologically inactive and not considered to present earthquake hazards.

The landforms in the BLR basin largely took shape during the Pinedale Glaciation (about 75,000 to 13,000 years ago) when the terminus of a valley glacier originating in the headwaters of Cross Creek and its tributaries flowed and spread out into the Eagle River Valley during up to three glacial advances. During the farthest advance of the Pinedale Glaciation, the advancing ice front split into two or more lobes where its advance encountered the bedrock hills south and west of the BLR (Pierce, 2003; Kirkham and others, 2012; and Tweto and Lovering, 1977).

The glacial lobe that occupied the BLR site deposited the up to 200-foot-high arcuate ridge ("terminal moraine") of boulder-strewn, granular glacial soils ("till") that now forms the downstream sides of the BLR basin. Relatively small, arcuate ridges of glacial debris ("end moraines") connecting the bedrock hills were later deposited as the ice front paused during retreat (Kirkham and others 2012; Tweto and Lovering, 1977). The downstream-most end moraine ridge forms the low, approximately east-west trending, broken ridge separating the BLR into two lake basins. The upstream-most end moraine forms the uneven, forested and boulder-covered ridge at the upstream limit of the BLR basin where the saddle dam will be aligned. This upstream-most end moraine ridge forms a drainage divide isolating the relatively small drainage basin of the BLR from the Cross Creek drainage to the west. Because there are no perennial streams flowing into the BLR basin, the Bolts Ditch was constructed to divert water from Cross Creek through the upstream-most end moraine to fill Bolts Lake.

The low elevation "water gaps" in the end moraine and terminal moraine likely formed as glacial melt-fed lakes breached and eroded through the moraines. The existing, abandoned BLR Dam was built across the downstream water gap in the terminal moraine. The proposed main dam alignment will again fill the water gap in the terminal moraine (see Figure 2).

6 SUBSURFACE CONDITIONS

Subsurface conditions observed in subsurface explorations at the BLR site broadly consisted of glacial deposits overlying bedrock. Glacial deposits include glacial till in the moraine ridge surrounding the north and east side of the BLR basin, and complexly interbedded glacial till, outwash, and glaciolacustrine (glacial lake) deposits in the subsurface of the BLR basin. Our subsurface observations are generally consistent with geologic mapping in the area (Kirkham and others, 2012; Tweto and Lovering, 1977). The following sections describe our data sources and individual geologic units. Groundwater conditions are described in Section 8.

6.1 Subsurface Data Sources

The Shannon & Wilson Team conducted a field exploration program to support this preliminary design effort between October 12, 2022 and November 7, 2022. The preliminary design subsurface exploration program consisted of drilling and sampling six geotechnical borings and installing six VWPs. Shannon & Wilson previously completed two additional field exploration programs, including:

- A feasibility study subsurface exploration program between June 7, 2021, and June 25, 2021, consisting of drilling and sampling six geotechnical borings (including two drilled and logged by LRE), excavating ten test pits, and installing three monitoring wells (Shannon & Wilson, 2021b).
- A subsurface exploration program and limited geotechnical characterization under subcontract to 8140 Partners to support their 2011 preliminary BLR design effort. This effort included drilling eleven borings, installing four monitoring wells, and excavating two test pits (Shannon & Wilson, 2011).

Shannon & Wilson also considered older boring logs and interpretation of subsurface soil conditions and depth to bedrock from ERM (2006, 2011, and 2012).

Data from all Shannon & Wilson subsurface explorations, including laboratory testing, is included in our GDR (Shannon & Wilson, 2023a). The locations of the previous and current Shannon & Wilson borings and test pits, and previous borings drilled by ERM, used in our characterization are included in Figure 2.

In addition to the explorations indicated above, Shannon & Wilson completed an engineering geology reconnaissance during the 2021 feasibility study to characterize rock mass conditions in visible rock exposures along the south side of the BLR. Mapping efforts focused primarily on characterizing discontinuities (e.g., separations on bedding planes and systematic fractures or "joints"). These mapping observations are summarized in our GDR (Shannon & Wilson, 2023a).

We prepared six generalized subsurface profiles illustrating our borings and select groundwater measurements conditions at the BLR site. A plan view showing the locations of the profile lines is included as Figure 2, and the individual subsurface profiles are included as Figures 3 through 8. We prepared an interpreted bedrock elevation contour map based on bedrock elevation intercepts in borings (Shannon & Wilson, 2011; Shannon & Wilson, 2021b, Shannon & Wilson, 2023a, and ERM, 2011) and surface outcrop locations. This bedrock elevation contour map is included as Figure 9.

6.2 Subsurface Soil Units

Glacial soil types observed in borings and distinguishable by laboratory testing are described in the following sections. In general, the subsurface soil units indicated poor correlation between individual borings. This observation is consistent with the complex and locally variable depositional environments expected to occur at glacial margins. We did not attempt to correlate glacial soil units on the subsurface profiles included as Figures 3 through 8. We did, however, observe an apparent increase in relative density of soils beneath some parts of the BLR. The relatively dense sediments may correspond to glacially overridden sediments that were deposited ahead of a glacial advance or during an older glacial advance and retreat.

6.2.1 Glacial Till Deposits

Till consists of sediments deposited by glacial ice. Because glacial erosion processes result in particle sizes ranging from clay size to boulder size, the resulting till deposits are characteristically poorly sorted. Till deposits are typically unstratified and often display a characteristic "matrix-supported" texture of gravel, cobbles, and boulders floating in a matrix of finer sediments. Till deposits are present in the glacial moraines surrounding the west, north, and east sides of the BLR topographic basin and in the subsurface beneath the BLR topographic basin. In general, we observed till deposits to consist of medium dense to very dense, silty sand with gravel and variable amounts of cobbles and boulders. Based on borings in the proposed cut envelope, cobbles and boulders are likely to comprise up to up to about 20% of till with local areas exhibiting higher proportions. We observed boulders ranging in size from 1 foot to about 6 feet in diameter in the cut envelope.

6.2.2 Glacial Outwash Deposits

Glacial outwash deposits consist of stratified sands and gravels deposited by meltwater rivers issuing from glacial ice. Outwash deposits typically exhibit fluvial (river-deposited) sedimentary structures, which may include horizontal stratification and cross bedding. We observed outwash deposits to consist of very loose to very dense, clean to silty, sand with gravel. Outwash deposits were identified in our subsurface explorations primarily by gradation analyses and occasionally by indications of planar bedding or cross bedding in samples. Outwash samples may locally contain cobbles and boulders.

6.2.3 Glaciolacustrine Deposits

Glaciolacustrine soils are deposited in ephemeral (temporary) lakes formed as meltwater streams are dammed by terminal or end moraines, advancing ice, or landslides. Glaciolacustrine deposits typically consist of fine-grained (clay and silt) soils that may exhibit horizontal planar laminations or "varves," formed by deposition of alternating annual layers of relatively fine and coarse sediments. Glaciolacustrine deposits may also include "dropstones" consisting of gravel, cobble, or boulder sized clasts dropped onto the lake bottom by melting icebergs.

Where observed in explorations, sediments of interpreted glaciolacustrine origin consisted of loose to very dense, trace sandy to sandy silts, and very stiff to hard, trace sandy to sandy, low plastic clays and silty clays. In test pits TP-05, TP-08 and TP-09, we identified clay and silt deposits ranging from 2.5 to 8.0 feet in depth that we interpreted as glaciolacustrine deposits. We observed varved deposits in test pit TP-09 and boring SW-105, including 45-degree laminations in boring SW-105 that may have rotated due to glacial "dozing" or landsliding.

6.3 Subsurface Bedrock Units

Bedrock encountered in borings was consistent with mapping and descriptions of Leadville Dolomite/Limestone, rocks of the Chaffee Group, and potentially rocks of the Harding Sandstone. Based on televiewer data and outcrop mapping, we commonly observed bedrock dips to the northeast at angles ranging from about 10 to 25 degrees. Bedrock units encountered in borings are described in the following sections.

6.3.1 Leadville Dolomite/Limestone

Several borings encountered limestone and dolomite bedrock of the Leadville Dolomite/Limestone. As described by Tweto and Lovering (1977) and Kirkham and others (2012), Leadville Dolomite/Limestone includes distinct limestone and dolomite facies, with limestone described north of Cross Creek and dolomite to the south. We encountered limestone facies bedrock in borings SW-MW-201, SW-MW-202, SW-MW-203, and SW-302. In these borings, Leadville Limestone generally consisted of medium strong to very strong (unconfined compressive strength [UCS] ranging from approximately 3,600 to 36,000 pounds per square inch [psi]), light gray to gray limestone with closely to moderately (approximately 2.5- to 24-inch) spaced bedding plane discontinuities.

We encountered dolomite facies bedrock in borings SW-107, SW-109, SW-205, SW-300, SW-301, SW-303, SW-305, and SW-306. In these borings, Leadville Dolomite generally consisted of medium strong to very strong, light to dark gray dolomite with characteristic zones of white, recrystallized layers (e.g., "zebra stone") and intervals with brecciated, angular black chert layers. Leadville Dolomite exhibited very close to close (1- to 8-inch) spaced bedding plane discontinuities and joints, and intervals of quartz or calcite-lined solution vugs up to about one inch in diameter. Rocks of the Leadville Dolomite are also exposed in the hill forming the proposed south reservoir rim, where this unit consisted of rounded outcrops of medium brown and gray, coarsely crystalline to micritic dolomite with bedding plane discontinuities and two to three sets of steeply dipping fracture sets forming rounded, deeply weathered and blocky outcrops. Where encountered in borings, Leadville Dolomite/Limestone is at least 64 feet thick, but is reported to be between 100 and 140 feet thick in the region by Tweto and Lovering (1977).

6.3.2 Gilman Sandstone

We interpreted dolomitic sandstone and sandstone-dolomite breccia observed in borings SW-300, SW-301 and SW-305 to correspond to the Gilman Sandstone. In these borings, Gilman Sandstone consisted of medium strong to very strong (UCS of 3,600 psi to 36,000 psi), light gray to gray, fine to medium-grained dolomitic sandstone and angular sandstone-dolomite breccia. Gilman Sandstone also included dolomite interbeds and exhibited very close to moderate (1-inch to 24-inch) spaced bedding plane discontinuities and joints. Where encountered in borings, the sandstone bearing intervals interpreted as Gilman Sandstone were about 11 to 17 feet thick. This is consistent with descriptions of Tweto and Lovering (1977) who indicate Gilman Sandstone thickness varies over short distances and ranges from about 10 to 50 feet thick in the region.

6.3.3 Dyer Dolomite

Borings SW-300, SW-301, and SW-305 encountered dolomite bedrock underlying the Gilman Sandstone consistent with descriptions of Dyer Dolomite (Tweto and Lovering, 1977; Kirkham and others, 2012). In these borings, Dyer Dolomite typically consisted of medium strong to strong (UCS ranging from approximately 3,600 to 14,500 psi), dark gray, finely crystalline, massive to occasionally laminated dolomite with very close to wide (1-inch to 6-foot-spaced) bedding plane discontinuities and joints. Our borings did not extend to the bottom of the Dyer Dolomite. Boring SW-300 encountered Dyer Dolomite on the order of 70 feet thick, which would be consistent with Tweto and Lovering (1977) who suggest that the Dyer Dolomite is typically 75 to 80 feet thick.

In boring SW-305, Dyer Dolomite displayed reduced rock quality designation (RQD) and was characterized by weak to medium strong (estimated UCS ranging from 700 to 7,200 psi) dolomite at elevations below about 8100 feet. In this interval, Dyer Dolomite was characterized by highly fractured rubble zones, clay-filled discontinuities, and solution cavities or very wide discontinuities up to about 2 feet high Boring SW-305 was terminated due to binding and unstable conditions in this interval. Similarly, the upper approximately 35 feet of boring SW-300 (approximate elevation 8090 to 8055) encountered a zone of low RQD in the Dyer Dolomite including a void or very wide (up to about 18-inch-high), open solution cavity at approximate elevation 8055 feet. The void features in borings SW-300 and SW-305 exhibited high hydraulic conductivities as evidenced by excessive water takes with no buildup of injection pressure while attempting to packer test these intervals.

Dyer Dolomite in boring SW-301 also exhibited rubble zones, zero RQD, and an approximate 1-foot-high void or open fracture. However, packer testing in this interval exhibited a relatively low hydraulic conductivity (estimated at 2.8x10-5 cm/s) suggesting relatively less continuity of fractures and voids.

6.3.4 Harding Sandstone or Taylor Pass Member of Manitou Formation

Rock core retrieved from near the bottom of boring SW-206 appears to be consistent with descriptions of the Harding Sandstone by Tweto and Lovering (1977) and Harding Sandstone or Taylor Pass Member of the Manitou Formation as described by Kirkham and others (2012). In this boring, bedrock consisted of medium strong to strong (UCS estimated at 3,600 to 14,500 psi), maroon to light green quartzite.

6.3.5 Dotsero Formation

Borings SW206 and SW-303 encountered dolomite and flat-pebble dolomitic conglomerate that appear to be consistent with descriptions of Dotsero Formation by Kirkham and others (2012)[1]. In borings, rocks interpreted as Dotsero Formation consisted of medium strong to very strong (UCS of 3,600 psi to 36,000 psi), gray, red-brown, and maroon dolomite, breccia, flat-pebble dolomitic conglomerate, sandstone with very close to close spaced joints.

Figure 9 presents an interpolated bedrock contour elevation map based on borings completed at the BLR site and other data as described above.

7 GEOLOGIC HAZARDS AND SEISMICITY

Rogers and others (1998) identified four potentially active faults within about 20 miles of the BLR site[2]. The most recent prehistoric deformations attested for these faults ranges from within the last 15,000 years (Gore Range Frontal Fault) to within the last 1.6 million years (unnamed faults northwest of Leadville). Because the nearest potentially active fault is located 13 miles from the BLR site, it is our opinion that the potential for ground surface fault rupture at the BLR site is low.

Liquefaction, seismically-induced compression, and lateral spreading may occur in loose, cohesionless soils when subjected to earthquake ground shaking. As noted in Section 12.8 of this report, we assumed a seismic peak horizontal ground acceleration (PGA) of 0.27g based on the USGS Unified Hazard Tool web application (USGS, 2023). This PGA value is for a 5,000-year return period based on the most recent seismic return-period guidance provided by the SEO (CDWR, 2007). Based on the high relative density typically observed in cohesionless subsurface soils encountered in borings at the BLR site, it is our opinion that the risk of these hazards is low. Based on a review of an abandoned mine database in the

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[1] This lithology is also consistent with descriptions of Tweto and Lovering (1977), who previously assigned this unit to the Peerless Formation.
[2] The fault traces mapped west of the BLR basin by Kirkham and others (2012) are not considered to be seismogenic.

area, abandoned underground mine workings are not documented to exist in or below the Project site (Sares and others, 2020).

8 GROUNDWATER OBSERVATIONS AND MODELING

8.1 Groundwater Observations

LRE conducted two groundwater site investigations at the BLR, including the due diligence investigation and modeling effort (LRE, 2021) and the 2022/2023 investigation and model update study (LRE 2023a). The LRE (2021) effort included installing three monitoring wells (SW-MW-201 through SW-MW-203), aquifer slug testing on 11 new and existing wells, and measuring water levels at 27 new and existing wells. In addition, LRE recovered historical water level data from existing LevelTroll500 pressure transducers in wells BL-MW-2, BL-MW-4, and CTP-MW3. The transducers had accessible pressure data from December 2010 through August 2019 (BL-MW-2) and through December 2019 (BL-MW-4 and CTP-MW3). The locations of the new and existing monitoring wells are shown on Figure 2.

The preliminary design effort by LRE (2023a) included installation of additional instrumentation (including volumetric water content probes and shallow monitoring wells in the OTP) and additional monitoring of these and other accessible groundwater instrumentation in and around the BLR site. LRE (2023a) also updated their regional groundwater flow model based on this groundwater monitoring data as well as additional borings, piezometer measurements, and geophysical data completed by Shannon & Wilson (2023a). Details of instrumentation and groundwater monitoring details are included in the LRE (2023a) report.

8.2 Groundwater Modeling

8.2.1 Model Construction and Calibration

LRE developed a numerical groundwater flow model for the Cross Creek and Eagle River basins, specifically focusing on the area surrounding the proposed BLR. Developed in two phases, the model is a tool to help understand the existing groundwater flow regime and assess potential impacts from the BLR. Initial findings and detailed methodology are documented in the Bolts Lake Reservoir Due Diligence Groundwater Model Report (LRE, 2021), with subsequent updates and data in the 2022/2023 Hydrogeologic Field Investigation Results and Groundwater Model Update memorandum (LRE, 2023a). Objectives of the Groundwater Flow Model include:

- Simulate hydrologic conditions at the BLR site;

- Assess the impact of BLR seepage on the surrounding groundwater regime, including the CTP;
- Evaluate seepage under various liner conditions; and
- Explore strategies for mitigating high groundwater conditions.

The model was calibrated to groundwater elevation measurements taken in 2021 from 27 monitoring wells located at and near the BLR and the CTP and 7 additional groundwater elevation measurements from monitoring wells in previous site investigations. Aquifer hydraulic conductivity, riverbed conductance and elevation, and recharge rates were modified to achieve calibration. Overall, the calibration was determined to not have a strong bias, the root mean squared error of residual was 11.45 feet, which is within industry standard error (10% of the total head change), and therefore appeared to be suitable for meeting the Project objectives.

Key insights on the groundwater conditions of the proposed BLR site include:

- **There is a high-permeability, alluvial channel feature along the Eagle River to the east of the BLR, which acts as a drain controlling the groundwater flow direction.** Although the channel feature was identified in previous investigations, the permeability measured by LRE (2021) is higher than expected based on material description in previous reports. This feature acts as an area of low groundwater potential driving the natural groundwater flow direction in the vicinity of the BLR to the east and toward the Eagle River channel.
- **Natural recharge rates in the OTP and Bolts Lake area are high and control annual and inter-annual groundwater fluctuations.** Soil moisture and groundwater level measurements in the OTP taken in the spring and summer 2023 (LRE, 2023a) indicate average annual recharge rates between 0.02 and 0.05 ft/day with higher rates during the spring and summer runoff. High recharge rates concentrated in the spring and summer result in seasonal and long-term fluctuations in the groundwater table elevations.
- **There are significant seasonal and long-term fluctuations in groundwater levels in the BLR area.** Long-term monitoring at BL-MW-2 indicates that there is the potential for nearly 10 feet of annual and inter-annual fluctuations in groundwater levels depending on hydrologic conditions with a maximum observed water level elevation of 8109 feet (LRE, 2021).
- **The lateral and terminal moraine ridges along the north and east side of the reservoir act as a barrier to flow due to a combination of the bedrock geometry and low-permeability glacial moraine deposits**. In combination with the low groundwater potential along the Eagle River channel, these areas further direct the natural groundwater flow towards the east and results in a steepening of groundwater gradients through the terminal moraine.

One of the uncertainties in the first iteration of the model was the nature of the bedrock ridge between the BLR and the CTP (LRE, 2021). The observed groundwater levels indicated that the ridge acts as a barrier to groundwater flow, as evidenced by the significant drop in groundwater levels from the up-gradient (reservoir area) to the down-gradient (CTP) sides of this feature. Originally the ridge was modeled with a bedrock high in the subsurface, which produced a suitable calibration to water level observations. However, direct (borehole) evidence and indirect observations (geophysics) indicated that bedrock elevations were lower than simulated and rose abruptly further to the west. The model was updated to reflect the observed bedrock geometry and modeled the lateral moraine with a lower permeability.

The combination of the modified bedrock geometry and recharge rates based on field observation resulted in an improved model calibration. The root mean squared error of the calibrated model was reduced from 15.8 feet to 11.45 feet, and the range of residuals was reduced from 55.2 feet to 45.8 feet. Additionally, the original model had some bias in the lower model elevation that was resolved with the modifications.

8.2.2 Groundwater Model Uncertainties

Key remaining uncertainties in the LRE (2023a) groundwater flow model include:

- **The effect of bedrock on the groundwater conditions on the flow regime is not considered yet may be an important factor.** The model assumes an impermeable bedrock unit; however, packer testing and surface geophysical surveys indicate that the bedrock is fractured with zones of high permeability and there is a spatially variable hydraulic connection with the glacial and alluvial units. Borings, geophysical reports, and packer testing data are included in the GDR (Shannon & Wilson, 2023a).
- **The model does not consider or capture seasonal or inter-annual fluctuations.** The model is in steady state and is calibrated to 2021 conditions and does not capture transient hydraulic conditions and the seasonality in the water budget.

8.2.3 Predictive Model Simulations

LRE simulated the impact of reservoir seepage on the groundwater regime using forward model simulations and modeled the proposed BLR using the river package. LRE tracked the simulated seepage to evaluate changes in the groundwater flow regime using particle tracking.

Assuming no reservoir liner and a constant reservoir elevation head of 8,139 feet, the groundwater model predicted seepage losses of more than 1,700 acre-feet per year, which is more than the proposed storage of the proposed BLR.

To evaluate seepage losses with a constructed liner, LRE (2023a) simulated a constant reservoir elevation head of 8,139 feet and added a 3-foot thick liner. Considering a liner hydraulic conductivity of 0.00283 to 0.000283 ft/day (1 x 10^{-6} to 1 x 10^{-7} cm/s), the simulated reservoir seepage rate was 256 to 30 acre-ft per year, respectively. Under steady-state seepage conditions, the model predicted the following:

- For the above two hydraulic conductivities, the simulated maximum groundwater mounding beneath the CTP is predicted to be less than approximately 0.1 feet, and the seepage does not meaningfully change the existing groundwater flow paths.
- When simulated seepage from the proposed BLR increases beyond 256 acre-ft/year, the model predicts the potential for groundwater mounding at the southern portion of the CTP and the potential for changing the existing groundwater flow paths beneath the CTP. The spatial extent and magnitude of mounding and potential for new flow paths to develop beneath the CTP increase as the seepage loss increases.
- Groundwater mounding will occur beneath the reservoir and the potential for developing new groundwater flow paths increases if reservoir seepage exceeds natural groundwater recharge at the existing BLR site.

8.2.4 High Groundwater Management Alternatives

LRE (2023a) evaluated two alternatives for capturing and re-routing shallow groundwater in the area immediately upgradient of the proposed Bolts Lake. The primary objective is to maintain groundwater levels below the elevation of the engineered liner while allowing for the deepening of the reservoir to maximize storage volume. The alternatives included:

1. **Permanent, Passive Groundwater Drain** - This system could consist of an earthen trench excavated to the desired groundwater elevation with permeable backfill and/or a perforated pipe at the bottom of the trench to outlet groundwater to the Eagle River. We anticipate that a passive groundwater drain would present considerable constructability challenges. A passive drain would likely need to be on the order of 1,000 to 1,500 feet long to intercept groundwater flowing into the BLR basin. The alignment of this drain may need to cross some areas of the OTP property. The trench may need to be more than 30 feet deep in higher elevation areas west of the BLR and would likely require installation of dewatering wells for construction in addition to excavation shoring, and removal of large boulders to place a drainage element (such as a perforated pipe) "in the dry."
2. **Permanent, Active Dewatering System** - Installation of an effective dewatering system could consist of installing and maintaining multiple, permanent dewatering wells and groundwater outlet piping that would drain into the Eagle River. A permanent dewatering system would also require long-term operations and maintenance to ensure that subsurface groundwater wells, pumping equipment, and outlet appurtenances would function as designed when the reservoir pool is lowered. Alternatively, the

District could develop a temporary groundwater dewatering plan including installation of new relief wells and other measures for each low-pool operation event.

The model showed that under both hydrologic scenarios, the lateral drain and dewatering wells can effectively maintain water levels at or below 8,105 feet upgradient of the BLR. The simulated discharge rates required to lower the groundwater table are similar in both scenarios, ranging from approximately 30 gpm to 180 gpm, respectively. While the simulation assumes four dewatering wells in each scenario, it is likely that fewer wells would be necessary under 2021/2022 hydrologic conditions.

The choice between the dewatering alternatives should be based on factors such as constructability, costs, operations and maintenance, and discharge management and permitting. The model suggests that four dewatering wells may be sufficient to adequately lower groundwater levels. Given the length and depth of the drain, and the potential for encountering boulders during construction, dewatering wells are likely the more cost-effective option and are assumed in this preliminary design. However, it is important to note that the wells might have higher operations and maintenance costs, and there is significant uncertainty related to the actual yield and corresponding aquifer response compared to the model simulation. Additional evaluation of this option (including installation of a pumping well, observation wells and performing aquifer pumping test; and chemical analysis of well effluent to characterize treatment needs) will be required for final design.

8.3 Design Groundwater Surface

Shannon & Wilson used the results of the LRE (2023a) groundwater model for preliminary design grading models. Our grading model accounted for groundwater elevation contours developed by LRE (2023a) assuming an active dewatering well system upgradient of the BLR and the highest observed groundwater surface observed in 2011. The design groundwater surface is shown on Figure 10.

9 ENVIRONMENTAL CONSIDERATIONS

9.1 Soil and Groundwater Contamination

The BLR site is surrounded by property owned by the Battle North and referred to as the North Property, which includes Operable Unit 3 (OU3) of the Eagle Mine Site. While the North Property includes several remedial features of the Eagle Mine Site, the 2006 Remedial Investigation report (ERM, 2006; ERM 2011a) determined that the BLR was not impacted by contaminants from the Eagle Mine and was therefore no longer included as a remedial area in OU3. A previous Phase I ESA noted that areas within the BLR basin may have been impacted by mine waste, which was a concern because of the proposed excavation and reuse of soil within the BLR basin (Pinyon Environmental, Inc., 2020). The Phase I ESA also noted that past groundwater sampling within the BLR basin had identified dissolved manganese concentrations in groundwater samples above the regulatory standards.

During the feasibility study, Shannon & Wilson collected additional soil, groundwater, and surface water samples as part of a Phase II ESA. The sampling methodology and analytical results for these samples are included in the Phase II ESA report (Shannon & Wilson, 2021a). The metals concentrations detected in soil samples collected from the test pits within the proposed BLR basin were below applicable soil cleanup values for residential uses. The 2013 Battle Mountain Feasibility Study (ERM, 2011b) states: “A very small amount of waste rock and soil was noted in a road bed within the Bolts Lake Area. This waste material is not located within the proposed Bolts Lake Reservoir.” Portions of this access road will be below the embankment for the saddle dam. If this road material needs to be excavated, it will need to be managed and disposed of properly.

Metal concentrations detected in the groundwater samples collected during the Phase II ESA were below the below the Colorado Department of Public Health and Environment (CDPHE) groundwater standards and/or EPA maximum contaminant levels (MCLs) except for manganese detected in groundwater samples collected from borings BL-MW-1, BL-MW2, and SW-MW-201. The manganese detected in these three samples exceeded the CDPHE drinking water standard. The manganese detected in the sample collected from boring BL-MW2 slightly exceeded the CDPHE agricultural standard. (There is no EPA MCL for manganese.)

Based on the concentrations of metals detected in the Phase II ESA surface water and groundwater samples collected at and near the BLR site, the BLR site does not appear to be a source of metals observed in the surface water of the Eagle River.

Any groundwater extracted for construction dewatering or low-pool operation of the proposed BLR will need to be managed properly. This will include obtaining a Colorado Discharge Permit System (CPDS) discharge permit, which will likely in turn require treatment measures to reduce manganese concentrations to allow discharge into the Eagle River. Additional hydrogeologic and environmental studies will be required during final design to evaluate the likely discharge rate, chemistry, and corresponding treatment requirements of groundwater that would be produced by a dewatering system. Additional study will also be required during final design evaluate how dewatering activities will impact groundwater on the adjacent OTP site. Additional discussion of dewatering requirements is included in Section 8.2.4 of this report.

10 ENVIRONMENTAL PERMITTING CONSIDERATIONS

Construction of the Project will require compliance with multiple federal, state, and local regulatory requirements.

10.1 Federal Permits

Rebuilding of the Bolts Ditch Headgate on Cross Creek and portions of the Bolts Ditch and pipeline will impact National Forest lands, as well as streams and wetlands within the Eagle River Basin. As such, the Project will require federal permitting to meet Clean Water Act (33 U.S.C. §1251 et seq.), National Environmental Policy Act (NEPA, 42 U.S.C. §4321 et seq.), Endangered Species Act (16 U.S.C. §1531 et seq.), and Wilderness Act (16 U.S.C. §1131-1136) compliance requirements. The Clean Water Act grants regulatory authority to the EPA and U.S. Army Corps of Engineers (USACE) to regulate impacts to Waters of the United States, which are navigable water bodies such as lakes and rivers, their "reasonably permanent" tributaries, and wetlands that are "indistinguishable" from navigable water bodies or their tributaries (EPA, 2023). Depending on the extent and nature of impacts to Waters of the United States, the USACE can require compensatory mitigation to offset impacts. The NEPA review process is applicable to projects that require a federal permit decision and will impact federal assets (lands or buildings) or otherwise protected resources, habitats, and species. NEPA requires that the federal permit decision consider the environmental effects of the Project, including hearing of public comments, alternatives to the Project, and quantification of impacts. Under NEPA, the public process is documented in the preparation of an Environmental Assessment (EA) or an Environmental Impact Statement (EIS) by a lead federal agency. The Wilderness Act allows Congress to protect special areas within the National Forest System as designated "wilderness areas" where "the earth and its community of life are untrammeled by man, where man himself is a visitor and does not remain." Wilderness Areas are under the jurisdiction of the U.S. Forest Service (USFS) and have additional requirements for any proposed project that may impact those areas.

10.1.1 Clean Water Act Section 404 Permit

Based on preliminary plans, the Project will have minimal impact on Waters of the United States, including wetlands, as defined by the Clean Water Act. A wetland delineation study completed for the Project area identified the presence of aquatic habitats and wetlands within and near the proposed water diversion, outfall, and conveyance facilities (see Figure 11). A jurisdictional determination from USACE is pending (AlpineEco, 2021). However, we anticipate only a small portion of aquatic habitats and wetlands in the Project area are likely under USACE jurisdiction, of which less than 1/10-acre are likely to be impacted by the Project. Therefore, the Project will be readily permittable with a streamlined

Nationwide Permit 39 from the USACE under Section 404€ of the Clean Water Act and will not likely require any compensatory mitigation.

10.1.2 NEPA, Wilderness Act, and Endangered Species Act

Given that most of the Project's federally regulated impacts will affect National Forest and Wilderness Areas under the jurisdiction of the USFS, a Standard Form 299 (SF299) permit application to the USFS for approval of the Project will be required. The permit application will trigger the NEPA review process led by the USFS. Due to the Project's impacts on the Holy Cross Wilderness, the USFS will also require additional documentation of alternatives and impacts to the primitive character of Wilderness Areas under the Minimum Requirements Alternatives Framework (MRAF).

Other consulting federal agencies will likely be involved in the NEPA review process, such as the U.S. Fish and Wildlife Service (USFWS), USACE, and the EPA. Under Section 7 of the Endangered Species Act (16 USC §1531 et seq.), the lead agency would be required to initiate consultation with the USFWS to ensure that threatened and endangered species are not adversely affected by the Project. In addition to Section 7 consultation, under the Fish and Wildlife Coordination Act (16 USC §661 et seq.), the USFWS is authorized to evaluate of impacts to fish and wildlife from water resource development projects. The USACE has authority over permits issued under Section 404 of the Clean Water Act, discussed previously. The EPA has final review and approval authority under the Clean Water Act, including veto authority, over Section 404 Permit decisions made by the USACE.

The NEPA process will also invite public comments from citizens, as well as state and federal agencies that may be incorporated as special conditions in a final permit issued by the USFS. Under the Fish and Wildlife Coordination Act, federal agencies are required to consult with local fish and wildlife management agencies. Colorado Parks and Wildlife (CPW) will work with the federal agencies in the evaluation of impacts and recommended mitigation measures. The issues of concern include water quality impacts on aquatic life and potential impacts on wildlife habitat, including game migration corridors. Though the CPW has no permitting authority, their comments will be influential in federal, state, and local permitting decisions.

10.2 State Permits

10.2.1 Clean Water Act Section 401 Water Quality Certification

Section 401 of the Clean Water Act requires that any federal permits for the Project cannot be approved unless a Water Quality Certification is either issued or waived by the State of Colorado, administered by CDPHE Water Quality Control Division. If water quality impacts by the Project are minimal, CDPHE may waive certification, or may require an antidegradation analysis to determine whether any decrease in water quality caused by project construction would be within certain limits established by the CDPHE Water Quality Control Commission. Under the State of Colorado 401 Certification Regulation, applicants for Nationwide Permits do not need to submit documentation to CDPHE (CDPHE, 2023).

10.2.2 Stormwater and Discharge Permits

Construction of the Project will result in disturbance of more than one acre, which will require the general contractor to obtain a Construction Stormwater Discharge permit and will require a detailed Stormwater Management Plan and erosion control plan.

10.2.3 State Engineer Approval

Prior to construction, the Project will be subject to approval by the CDWR Dam Safety Branch, who is responsible for review of engineering designs, specifications, and construction oversight for jurisdictional-size dams. In Colorado, Jurisdictional Dams are those having a statutory heigh greater than ten feet to the spillway crest, creating a reservoir with more than 100 AF of water, or covering more than 20 acres at the high waterline. Given the inherent hazards in the unlikely event of dam failure, Jurisdictional Dams also require regular maintenance and monitoring once constructed, including development of an Emergency Action Plan (EAP) to respond to such failures.

10.2.4 Waters of the State Enforcement

Following the Sackett vs. EPA decision by the Supreme Court of the United States in May 2023, CDPHE issued Clean Water Policy 17 in July 2023 which asserts its intention to "exercise enforcement discretion for discharges of dredged or fill material into state waters that are not subject to federal Section 404 permitting[…]." While no state-level regulatory or permitting process is currently in place, the Colorado state legislature and CDPHE are in the process of developing a permitting framework to regulate impacts to state waters. Given that this forthcoming regulatory framework will be untested, it presents risk to any project that may impact non-jurisdictional wetlands and may even delay implementation of future projects.

10.2.5 Other State Permits

Additional state agencies that may require permits include the CDPHE Air Pollution Control Division and the Colorado Department of Transportation for any impacts to air quality or highways, respectively.

10.3 Local Permits

Eagle County Guidelines and Regulations for Matters of State Interest (the "1041" Regulations) apply to the designation and regulation of any area or activity of state interest wholly or partially in the unincorporated areas of Eagle County, whether on public or private land. The Town of Minturn has also codified 1041 regulatory authority over any area or activity of state interest wholly or partially in the Town limits. As with the NEPA review process, the County and/or Town would be the lead agency responsible for documenting the potential impacts of a project approval, including environmental, social, economic, and other relevant impacts. The permit application requirements share significant overlap with the requirements in an EA or EIS.

Given that this Project will involve activity on lands located in both unincorporated Eagle County and the Town of Minturn, both of whom designate 1041 authority municipal water projects in their respective regulatory codes, it will be subject to 1041 regulation. Section 16-25-40(8) of the Town of Minturn municipal code allows for exemption from 1041 regulations if there is an intergovernmental agreement (IGA) with another governmental entity specific to the Project. It is our understanding that the District and the Town of Minturn are presently developing an IGA specific to the Project.

Both Eagle County and the Town of Minturn will also require a floodplain development permit and/or a "no-rise" certification be obtained prior to construction within any area of special flood hazard. The Project is located outside of a published FEMA study area, so the County and the Town Floodplain Administrators will need to make a necessary judgment to determine the flood hazard resulting from the Project. Initial consultation with both the County and Town finds that that a no-rise certification will be sufficient in this case as the Project's proposed diversions and conveyances will have minor impact, if any, on either the Cross Creek or Eagle River's floodway and floodplain. Accordingly, the District will be able to successfully obtain the necessary no-rise certification.

11 WATER DIVERSION AND PIPELINE DELIVERY SYSTEM

11.1 Diversion System Alternatives

There are several alternatives that were identified to convey water into the Bolts Lake that can physically and legally meet the future District demands. The alternatives are as follows:

- Eagle River Gravity,
- Eagle River Gravity and In-Line Pump Station,
- Eagle River Pump Station,

- Cross Creek Gravity,
- Eagle River and Cross Creek Gravity, and
- Cross Creek Gravity and Eagle River Pump Station

These alternatives are further discussed in a forthcoming LRE memorandum (LRE, 2024). Based on LRE analyses (LRE 2023c) and guidance from the District, the Cross Creek Gravity and Eagle River Pump Station alternative was selected for preliminary design. Preliminary design drawings of the Cross Creek Diversion, Pipeline, and Eagle River Pump Station alternative are issued under separate cover.

12 DAM AND RESERVOIR DESIGN CONSIDERATIONS

12.1 Influence of Geologic and Environmental Factors

In our opinion, there are no geologic hazards that would preclude the construction of a dam at this site. However, because the subsurface beneath the proposed reservoir and main embankment dam consists primarily of permeable sand and gravel sediments up to approximately 100 feet thick, construction of the BLR will require a liner to reduce uneconomically feasible seepage losses and mounding of groundwater at the downgradient CTP.

Provided that adequate seepage control is achieved, there appears to be only limited environmental concerns with the site for construction because Battle North will manage and dispose of any contaminated soils encountered during construction in accordance with an agreement of consent. While groundwater extracted as part of the proposed BLR construction activities may need to be treated and discharged properly in accordance with local, state, and federal regulations and permitting requirements, we do not anticipate that this will be an impediment to construction of the Project.

12.2 Preliminary Dam Section and Zoning

To construct a reservoir onsite will require the construction of a main embankment dam (approximately 1,650-foot-long by up to 58-foot-high embankment) and smaller saddle dam (approximately 615-foot-long by up to 30-foot-high south embankment) with dam crest elevations of 8153 feet (which will provide 3 feet of freeboard above the proposed 8150 feet WSE). Unless otherwise stated in the report, the use of the word "dam" refers to both the main embankment dam and the smaller embankment saddle dam.

Earth-fill embankment dams are generally designed with different material zones based on the following factors:

- The type and availability of local soil and rock;
- Influence of foundation conditions; and
- Seepage considerations, including maximum allowable seepage losses from the reservoir.

An evaluation of factors in the selection of the recommended dam section and zoning are presented in the Dam Type Alternatives Evaluation Report (Shannon & Wilson, 2021c). The key conclusions from that report, as well as our further studies, are discussed below.

We recommend an earthen embankment dam consisting of an upstream sloping clay core and filter/drain material covered on both sides by a granular shell material excavated from the reservoir basin. In other locations, a geosynthetic filter will be used to reduce the potential for piping to develop in the clay liner. See Exhibit 12-1 and the accompanying preliminary plans for our recommended embankment dam section.

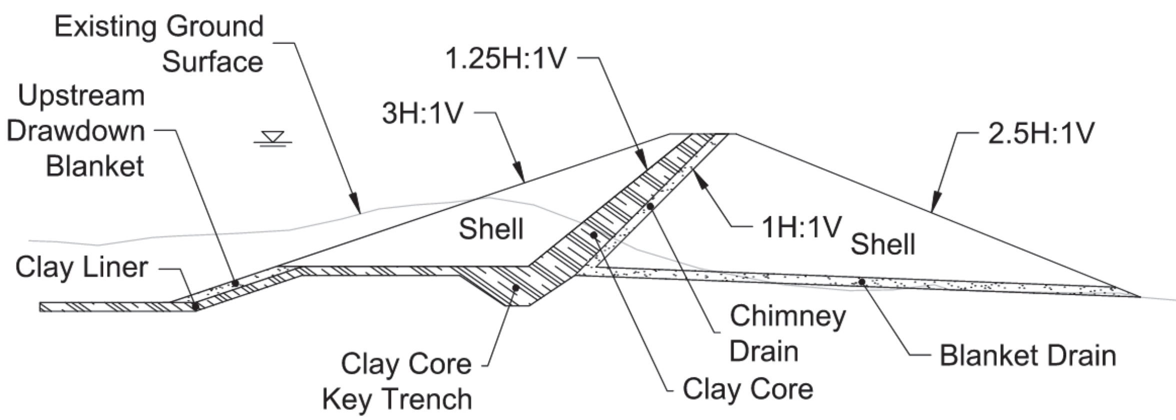

Exhibit 12-1: Preliminary Dam Section and Zoning (not to scale).

The clay core will require construction of a transition section at the dam abutments and a key trench beneath the dam. While the key trench beneath the dam will not be able to penetrate into an impermeable layer, it will lengthen any seepage path and reduce the potential for toe seepage. In addition, the inclined clay core should be constructed with an accompanying inclined chimney filter and drain connected to a horizontal blanket drain. An inclined clay core will allow a steeper downstream embankment slope that will in turn increase reservoir capacity by moving the dam alignment to the east. While seepage through the liner is anticipated to flow generally downward toward the unconfined groundwater table present downgradient of the dam alignment, the horizontal blanket drain is recommended to provide a preferential seepage path to efficiently remove seepage that flows through the clay core, as well as decrease the likelihood that reservoir leakage can saturate and reduce stability of the downstream shell.

The preliminary design includes profiles along the proposed dams where we recommend the existing glacial moraine ridges be graded at relatively gentle slopes (less than 10H:1V) to facilitate placement and compaction of the dam. The embankment foundation profile along the dams was established to extend below native grade so that the clay core key trench will be excavated into native materials. The foundation profile should be adjusted during final design as necessary to excavate any remnants of the former earthen BLR dam that was breached in the 1990s.

12.3 Camber

We anticipate that the primarily cohesionless subsurface soils encountered in borings below the dam alignments will settle elastically as the embankment is constructed. We anticipate that post-construction foundation settlement will be negligible. Similarly, if compacted properly, the embankment materials should not settle significantly following construction (self-weight settlement).

However, to account for variation in thickness of overburden soils and in accordance with good practice, we recommend a camber be built into the crest of the dams to ensure that the freeboard will not be diminished by post-construction settlement of the dam and the foundation. Because the relatively free-draining cohesionless soil materials encountered by borings in the dam foundations are anticipated to settle essentially elastically as load is applied, we recommend that final design of the dam include a camber of about 2% of the embankment height (about 6 inches). Camber should also account for the variation in soil thickness beneath the dam crest ranging from less than 20 feet at the right abutment to relatively thick soils at the left abutment. Linear equations should be used to vary the camber and make it proportional to the height of the embankment and variation of soil thickness underlying the embankment.

12.4 Access Road

The preliminary design includes a minimum 13-foot-wide access road around the perimeter of the reservoir traversing the top of the liner and the dam crests. The roadway will be accessed from the left abutment of the main embankment dam.

12.5 Dam Classification

A preliminary layout for a dam and reservoir at the BLR site is shown on Figure 12. Per the CDWR Rules and Regulations for Dam Safety and Dam Construction (2 CCR 402-1), Rule 4.6.1, we anticipate that the BLR Dam will be jurisdictional (CDWR, 2020). Depending on the configuration and storage volume chosen, the jurisdictional height is anticipated to on the order of 50 feet. Per 2 CCR 402-1, Rule 7.4.2.1.1, the dam crest for a 50-foot-high dam must be at least 18 feet wide (CDWR, 2020).

A preliminary inundation map was completed as part of the Project. Based on this study and because of the proximity to the Town of Minturn, we expect that the proposed dam will meet the criteria for a high hazard rating per CCR Rule 4.13.1 and will therefore require the completion of a final inundation map and EAP for final design of the BLR. Similarly, an extreme hydrologic hazard rating is expected per CCR Rule 4.15. As a result, the critical rainfall per CCR Rule 7.2.1 will be the PMP.

12.6 Dam Materials

As described in Section 5 of this report, the soils within the proposed excavation envelope are anticipated to consist primarily of medium dense to very dense, variably silty, gravelly sand with variable amounts of cobbles and boulders. Based on borings and test pits within the excavation envelope, we estimate the following volume proportions of various grain sizes in cut materials excavated from within the preliminary design cut envelope.

- Boulders could account for on the order of 10% of the entire excavation volume. Where encountered in borings, boulder intervals within the excavation envelope ranged up to 6 feet in length. Cobbles could account for an additional approximately 5% to 10% of excavation volume. Cobbles and boulders will likely be subangular to rounded and composed of high to very high strength rock types (granites, gneisses, quartzites) that could be crushed to likely yield aggregates and select fill materials. Cobbles and boulders are more likely to be encountered on glacial moraine ridges (obvious topographic ridges around perimeter of lake) where some individual borings encountered upwards of 30% cobbles and boulders. Boulders will also likely be encountered in the bottom of the BLR topographic basin as well as evidenced by boulders and cobbles encountered in test pits and borings.

- Low plastic clay (CL) and silty clay (CL-ML) with varying amounts of sand could account for on the order of about 5% to 10% of excavation volume. Plastic soils are more likely to be encountered in the basin bottom and will contain organics in upper foot or two. Lateral variability in borings and test pits suggest that plastic soils are not likely to form laterally continuous deposits that can easily be segregated for use as liner or dam core. Additional exploration and laboratory testing will be required during final design to better evaluate the extent and hydrogeologic properties of clay soils and whether they have strength and permeability properties compatible for use in dam and liner construction.

Exhibit 12-2 includes a plot of available grain size analysis within the preliminary design cut envelope (Shannon & Wilson, 2011, 2021b, and 2023a). This exhibit is not intended to show the proportion of grain sizes because tested samples were not chosen randomly and may not reflect bulk gradation properties. The grain size data in the table also does not include information regarding cobbles or boulders as it is not practical to recover or test samples containing cobbles and boulders.

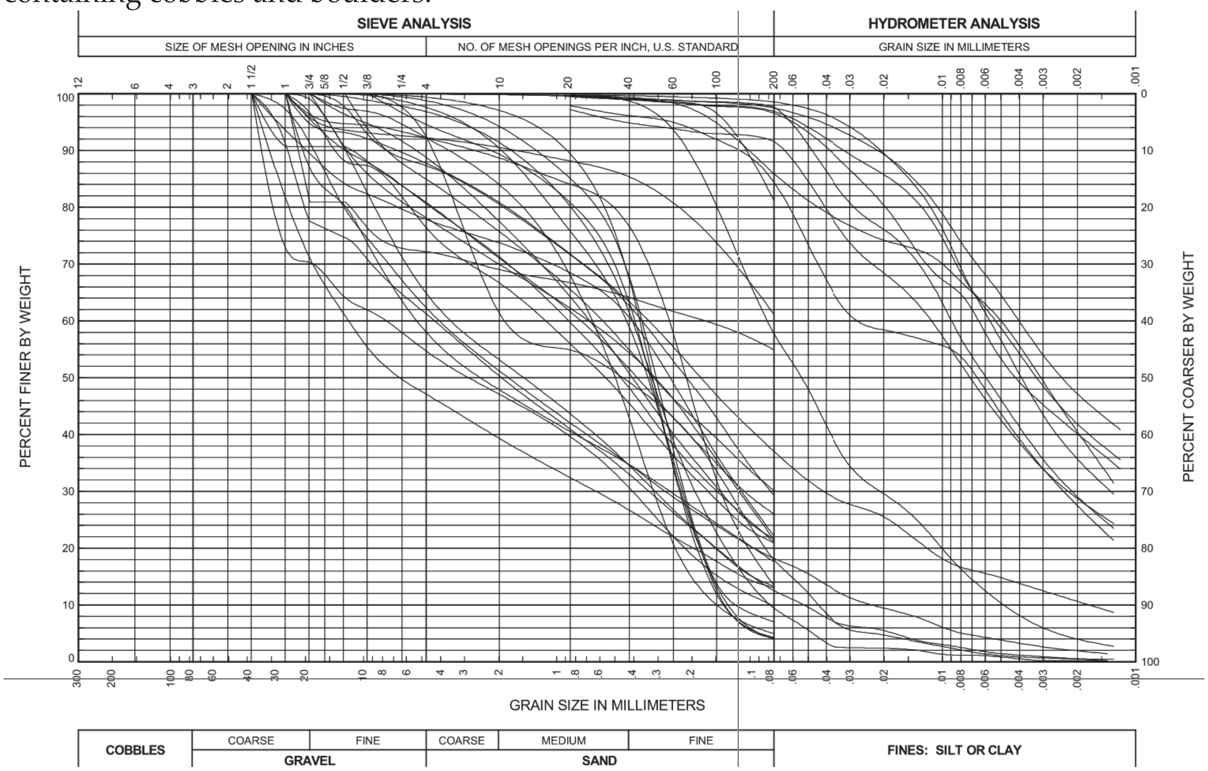

Exhibit 12-2: Grain Size Distributions of Soil Samples in Anticipated Excavation Zone

We anticipate that the excavation envelope will yield adequate volumes of sand and gravel with frictional strength and hydraulic conductivity properties suitable for use in constructing granular embankment shells of a dam. Other select materials such as chimney and blanket filter drain sands, drawdown blanket gravels, and potentially spillway riprap could likely be obtained from the excavation envelope with additional crushing, screening, and/or washing.

As described above, clay soils suitable for use in an interior core or reservoir liner are not anticipated to be present in adequate volumes in the anticipated excavation envelope of the BLR. Therefore, we anticipate that suitable clay soils will need to be imported for construction of the BLR. A Shannon & Wilson (2021d) borrow source study identified the Biosolids Containment Facility (BCF) site near Wolcott, Colorado as a potential clay borrow source for potential earthen dam cores and a compacted clay liner.

A preliminary subsurface exploration program and laboratory testing program for this site is described under separate cover (Shannon & Wilson, 2023b). This study suggests that while the BCF site is likely to provide clay material suitable for use in the clay core and liner, it is not likely to provide adequate quantities of suitable clay for construction of the entire BLR project. BLM and Eagle County parcels adjacent to the BCF parcel are likely to have similar suitable clay soils that could be mined to recover the balance of the required clay for construction of the BLR. Additional study will be required during final design to evaluate the properties, quantity, and permitting/environmental requirements to exploit clay deposits on adjacent lands for borrow and transport to the BLR (Shannon & Wilson, 2023b).

12.7 Seepage Loss and Reservoir Liner Considerations

As indicated previously, the BLR will experience significant seepage losses if the reservoir pool is in contact with relatively highly permeable silty and sandy soils in the reservoir subgrade. Without measures to limit seepage, construction of the BLR would result in unacceptable water losses for the District. Significant seepage may also result in downgradient groundwater mounding beneath the southern portion of the CTP repository and potentially change the groundwater flow paths beneath the CTP.

In our opinion, there are essentially two methods that could be used to reduce the rate of reservoir seepage at the BLR site: (1) subsurface cut-offs and/or (2) impervious blankets or liners. Based on our preliminary evaluation, it is our opinion that a liner is the more constructable and economical solution (Shannon & Wilson, 2021c). Based on cost and long-term operational considerations, we recommend a liner be designed and constructed for the Project.

Our preliminary design study considered two different liner types – (1) a compacted clay liner and (2) an engineered geosynthetic liner. Regarding the former, our borrow evaluation and borrow source study reports detail our evaluation of a potential clay borrow source (Shannon & Wilson, 2021d and Shannon & Wilson, 2023b). We also considered the use of a geosynthetic liner and obtained preliminary cost information, which suggested that a geosynthetic liner may result in a higher cost. In addition, we understand that the District prefers use of a compacted clay liner over a geosynthetic liner. Based on these considerations, a clay liner is included in the preliminary design.

Calculations by LRE (2023a) outlined in Section 8.2.3 of this report using a regional groundwater model indicate that a 3-foot-thick clay liner with a saturated hydraulic conductivity on the order of 1.0×10^{-7} cm/s or less will be required to limit annual seepage losses to about 2.5% of the design storage volume (or 30 acre-feet/year). LRE (2023a) also indicates that a 3-foot-thick clay liner with a saturated hydraulic conductivity of 1.0×10^{-6} cm/s would result in annual seepage losses of about 22% of design storage volume (or 256 acre-feet/year) and downgradient groundwater mounding beneath the CTP.

As documented in our BCF borrow study report (Shannon & Wilson, 2023b), we completed remolded flex wall permeability testing of four clay samples from the BCF borrow site. The results indicated an average saturated hydraulic conductivity of approximately 6.7 x 10^{-7} cm/s with the most conductive test result indicating a hydraulic conductivity of 1.7 x 10^{-6} cm/s. For the above average hydraulic conductivity, we estimate seepage losses much greater than the District's design seepage criterion and the limit for impacting the CTP. These test results suggest that blending of BCF clays from various parts of the site may be necessary to yield a material with more consistent hydraulic conductivity.

12.7.1 Steady State Seepage

We evaluated the seepage characteristics of the proposed dam and reservoir by constructing a 2-dimensional model through the main embankment dam using the Seep/W module of GeoStudio (GEOSLOPE International, Ltd., 2021). We estimated soil hydraulic properties using correlations to soil type encountered in our subsurface exploration programs and values from the LRE Water (2023) groundwater modeling report. We completed our analysis for the following scenarios assuming steady state seepage conditions would develop:

- Empty reservoir – the boundary conditions on either end of the model were set to closely match existing groundwater levels (8,094 feet upstream and 8,025 feet downstream).
- Full reservoir and clay liner, no chimney drain or blanket drain in dam – empty reservoir boundary conditions with an additional upstream boundary condition representing the full reservoir surface of 8,150 feet assigned to the upstream model surface.
- Full reservoir and clay liner, with a blanket drain in dam – empty reservoir boundary conditions with an additional upstream boundary condition representing the full reservoir surface of 8,150 feet assigned to the upstream model surface and the inclusion of a chimney drain and a blanket drain in the dam.

The above scenarios were run with two additional variables: 1) with and without a clay liner on the bottom of the reservoir, and 2) with two different hydraulic conductivity values ($1x10^{-7}$ cm/s and $1x10^{-6}$ cm/s) for the clay liner and core.

Our models indicate that without a clay liner on the bottom of the reservoir, the groundwater elevation will rise in response to a full reservoir condition. The models also indicate little to no change in existing groundwater elevation with a low permeability ($1x10^{-7}$ cm/s) clay liner. The addition of the chimney and blanket drains, along with a low permeability liner ($1x10^{-7}$ cm/s) also has little impact on the existing groundwater table. However, a clay liner/core with the relatively higher hydraulic conductivity value ($1x10^{-6}$

cm/s) shows that the groundwater will rise in response to a full reservoir and may reach the elevation of the downstream toe.

12.7.2 Reservoir Liner Uplift and Groundwater Considerations

Without groundwater control, uplift pressures are possible on the base of the liner system when the reservoir is operated at low pool levels. As discussed in Section 8.2.4 of this report, LRE (2021, 2022a, and 2023a) groundwater modeling indicates that groundwater control to accommodate construction and low-pool-level operation could likely be achieved by (1) construction of a permanent, passive groundwater diversion structure such as an upgradient interceptor drain, or (2) construction of a permanent, active dewatering system such as a dewatering well field. These options are discussed in additional detail in Section 8.2.4 of this report.

The preliminary design assumes use of dewatering wells as they are likely a more operationally flexible and less expensive to construct. Our elevation grading of the reservoir liner subgrade considered a design groundwater surface modeled and provided by LRE (2023a) assuming a dewatering well field upgradient of the reservoir operating under the highest observed groundwater levels in 2011. The design groundwater elevation is included as Figure 10.

12.8 Embankment and Reservoir Slope Stability

12.8.1 Methodology

We analyzed the global stability of the proposed dam embankment and reservoir slopes using the Slope/W module of GeoStudio (GEOSLOPE International, Ltd., 2021). Based on recommendations provided in US Army Corps of Engineers (USACE) Design and Construction of Levees (2000) and Slope Stability (2003), selected cross-sections were analyzed under the following conditions:

- End-of-Construction – this condition considers effective stresses and drained shear strengths in granular soils and total stresses and undrained shear strengths to account for excess pore pressures that will develop in the clay blanket and core during placement and compaction.
- Long-Term (Steady Seepage) – this condition considers drained shear strengths and effective stresses in all soils to simulate stresses occurring after post-construction pore pressures dissipate in the compacted clay liner, blanket, and core.
- Rapid Drawdown – this case considers effective stresses and drained shear strengths in granular soils and total stresses and undrained shear strengths in the saturated clay blanket and core. This case considered removal of the buttressing effect of the reservoir pool by assuming an instantaneous reservoir drawdown.

- Earthquake (pseudostatic) - this case considers effective stress and drained shear strength parameters in granular soils and total stress and undrained shear strengths in the clay liner and core with the addition of a horizontal seismic acceleration (k_h).

As indicated above, for this preliminary design, we completed pseudostatic analyses to determine an initial estimate of earthquake slope stability. Because the current guidance does not specify a return period for calculating a PGA, we used the previous Code of Colorado Regulations (CDWR, 2007), which indicates that embankments should be evaluated for stability under an earthquake with a minimum 5,000-year return period. A PGA of 0.27 for the 5,000-year return period was obtained from the Unified Hazard Tool web application (2023). This peak ground acceleration was modified for site class C to obtain a modified PGA of 0.36. We multiplied the modified PGA by ½ to account for wave scattering in the embankment, thereby obtaining a k_h value of 0.18 that was used in our pseudo-static analysis.

12.8.2 Slope Stability Results

The results of our preliminary stability analyses indicate that the downstream embankment can be constructed at slopes of 2.5H:1V (horizontal to vertical) or flatter. Upstream embankment and reservoir side slopes should be 3H:1V or flatter, as discussed below.

While low permeability clay is ideal for use as a liner, these compacted clay soils can develop excess pore pressures and become subject to undrained failure under rapid drawdown conditions. In our opinion, the rapid drawdown failure mode is less likely to occur on the reservoir side slopes because the liner will be thin and underlain by higher permeability, unsaturated native soils. As such, we anticipate that, while the liner may become saturated, it will drain relatively quickly because of double drainage through the bottom and top sides of the relatively thin clay liner layer.

We completed preliminary analyses in SEEP/W that indicated porewater pressures would dissipate relatively quickly under drawdown on the order of 1 foot per day. Nevertheless, consistent with typical practice, we evaluated the slopes for a rapid (instantaneous) drawdown condition. Under such analyses, the reservoir slopes had low factor of safety (FS) values. Methods to increase stability include (a) flattening slopes, (b) reinforcing slopes with geosynthetic layers, (c) increasing the strength of the liner material, and (d) using an upstream drawdown blanket as discussed in EM-1110-2-1901 (USACE, 1993). Because of the desire of the District to maximize the reservoir size, flatter slopes were not viewed favorably. Other methods to increase the strength of the clay layer would also introduce potential concerns. For preliminary design, we determined that a 3-foot-thick upstream drawdown blanket (comprised of well-graded gravel that could be likely processed on site from excavated materials) provided the best method to meet the goals of the Project. In addition to increasing the stability of the reservoir slopes to a FS of at least 1.3, the upstream drawdown blanket provides protection against wave action and reduces the potential for desiccation of the clay from freeze/thaw and excessive heat.

The following exhibit summarizes the FS values for our various analysis cases at three cross-sections based on the height of dam above the clay liner. The upstream and downstream slope of each cross section was analyzed, as appropriate for the loading condition. These sections also consider the reservoir rim slopes as the section is the same (i.e., 3 feet of granular fill overlying 3 feet of clay liner overlying granular soil).

Exhibit 12-3: Summary of Global Stability Analysis

Station	Condition	Upstream FS	Downstream FS	Minimum FS[1]
36+00	End-of-Construction	2.1	1.8	1.3
	Long-Term	2.1	1.8	1.4
	Rapid Drawdown	1.7	NA	1.2
	Earthquake – shallow[2]	0.9	1.2	1.1
	Earthquake – deep[3]	1.3	NA	1.1
40+00	End-of-Construction	2.1	1.8	1.3
	Long-Term	2.1	1.8	1.4
	Rapid Drawdown	1.7	NA	1.2
	Earthquake – shallow[2]	0.9	1.2	1.1
	Earthquake – deep[3]	1.3	NA	1.1
41+00	End-of-Construction	2.1	1.8	1.3
	Long-Term	2.1	1.8	1.4
	Rapid Drawdown	1.8	NA	1.2
	Earthquake – shallow[2]	0.9	1.2	1.1
	Earthquake – deep[3]	1.5	NA	1.1

NOTES:

1 Based on USACE, 2000 Table 6-1b and USACE, 2003 Table 3-1

2 Shallow failure surfaces occurred withing the dam embankment material above the clay liner.

3 Deep failure surfaces occurred through or beneath the clay liner.

NA = not applicable

As indicated in the above exhibit, shallow failure surfaces in the upstream slope do not meet the minimum FS requirements. This is not uncommon in slope stability analyses under high k_h values. As such, often FS requirements are replaced by estimating permanent deformations resulting from the earthquake conditions using a Newmark Sliding Block Analysis or other equivalent method. These methods are useful as they estimate deformations that occur when the base acceleration exceeds the yield acceleration (i.e., a

FOS of less than 1.0). The final designer will need to coordinate with the State Engineer's Office to determine the level of seismic stability that will be required for the Project.

Ice-related risks and wave action should be evaluated during final design to determine if additional slope protection should be provided.

12.9 Rock Outcrop

Based on preliminary evaluation of discontinuity orientations measured in Leadville Limestone bedrock outcrops and borehole televiewer data collected on the south side of the BLR basin, we anticipate that cuts in bedrock can be excavated at angles of 1H:1V. Kinematic analysis and rigid block limit-equilibrium analyses indicate that planar failures with inadequate factors of safety could occur on inclined joints and bedding planes at cut angles steeper than 1H:1V. Due to the relatively high strength of the outcrops exposed in this area, we anticipate that bedrock excavation will require blasting.

While the rock can be excavated at a relatively steep slope, there is a risk of significant seepage losses through the fractured rock mass in the reservoir subsurface as indicated by the variable hydraulic conductivity values indicated by packer testing (Shannon & Wilson, 2023a). Seepage through the open fractures and potential dissolution features in this carbonate rock mass also has the potential to further dissolve and enlarge fractures in dolomite, which will tend to increase seepage rates over time.

Seepage through bedrock could potentially be reduced by pressure grouting, a process by which a bentonite cement slurry is pumped into fractures via a series of closely spaced boreholes. However, grouting of fractured rock is a highly uncertain technique to reduce seepage and is subject to potentially large grout takes and significant cost overruns. Because of this uncertainty and potential for cost overruns associated with grouting fractured rock masses with dissolution features, we recommend that the rock be excavated or buttressed at a slope of 3H:1V and lined with clay, consistent with the remainder of the reservoir. Our preliminary design requires limited rock cut by placing fill against the rock slope in this area.

13 PRELIMINARY RESERVOIR PLANS, SPECIFICATIONS AND COST ESTIMATE

We prepared preliminary plans for an approximately 1,208-acre-foot reservoir, dam, Bolts Ditch diversion and pipeline delivery system, and Eagle River pump station. These plans are provided under separate cover. We also prepared a table of contents for specifications anticipated to be required for the Project.

KMC (in-process) prepared a cost estimate following the guidelines from the American Society of Professional Estimators (ASPE) and Advancement of Cost Engineering (AACE) for a Class 3 (Budget Estimate), which is appropriate for preliminary design purposes and that can be expected to have an estimated accuracy range of -15% to +30%. The cost estimate is provided under separate cover.

14 ADDITIONAL STUDIES

This preliminary report presents a preliminary design to develop a dam, reservoir, and water delivery system for the BLR site. This preliminary report and accompanying drawings are not sufficient for final design and construction. Further analyses will be required for final design. If the Project moves forward, the designer will need to discuss analyses and assumptions with the SEO. The SEO will also need to review final design documents.

15 LIMITATIONS

This report was prepared for the exclusive use of the Eagle River Water & Sanitation District/Upper Eagle Regional Water Authority for use in the preliminary design of the BLR Project. Our evaluations and opinions are based on:

- The limitations of our approved scope, schedule, and budget described in our Contract Agreement.
- Our understanding of the Project and information as indicated in this report and the other study documents.

Within the limitations of scope, schedule and budget, the preliminary recommendations and opinions presented in this preliminary report were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

Our report should be made available to prospective contractors and/or the Contractor for information on factual data only and not as a warranty of subsurface conditions. Unanticipated soil conditions are commonly encountered and cannot be fully determined by a limited boring and testing program. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

Our opinions of probable costs (as presented under separate cover) are based on 2024 prices. We have no control over the cost of labor, materials, equipment, or work furnished by others; the contractor's actual or proposed methods or pricing; competitive bidding; or market conditions. We do not guarantee that proposals, bids, or actual construction cost will be similar to our estimate. Shannon & Wilson is not a construction cost estimator or contractor. Our opinion of probable cost should not be considered equivalent to the nature and extent of services a contractor would provide.

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