RJH CONSULTANTS, INC.

GEOTECHNICAL AND
WATER RESOURCES ENGINEERING

FEASIBILITY EVALUATION

WALSENBURG CITY LAKE DAM REHABILITATION AND ENLARGEMENT

HUERFANO COUNTY, COLORADO
DAMID: 10327

Submitted to
City of Walsenburg
525 S. Albert Street
Walsenburg, Colorado 81089

Submitted by
RJH Consultants, Inc.
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FEASIBILITY STUDY APPROVAL
Pursuant to Colorado Revised Statutes 37-60-121 &122, and in accordance with policies adopted by the Board, the CWCB staff has determined this Feasibility Study meets all applicable requirements for approval.

Signed
Date 7/7/17

Project 14120
March 2017

COLORADO REGISTERED PROFESSIONAL ENGINEER
MICHAEL L. GRABER
19963
3-31-2017

Michael L. Graber, P.E.
Project Manager

COLORADO
Colorado Water Conservation Board
Department of Natural Resources

Water Project Loan Program

Application Type

☐ Prequalification *(Attach 3 years of financial statements)* ☑ Loan Approval *(Attach Loan Feasibility Study)*

Agency/Company Information

Company / Borrower Name: City of Walsenburg	
Authorized Agent &Title: Leslie Klusmire, City Administrator	
Address: 525 S. Albert Ave, Walsenburg,CO 81089	
Phone: (719) 738-1240 x243	Email: lklusmire@cityofwalsenburg.com
Organization Type: ☐ Ditch Co, ☐ District, ☑ Municipality ☐ other: ______	Incorporated? ☑ YES ☐ NO
County: Huerfano	Number of Shares/Taps: 1,700
Water District: n/a	Avg. Water Diverted/Yr 2,622 acre-feet
Number of Shareholders/Customers Served: 2,900	Current Assessment per Share $ n/a (Ditch Co)
Federal ID Number: 84-6000627	Average monthly water bill $ 62.31 (Municipality)

Contact Information

Project Representative: Leslie Klusmire	
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Engineer: RJH Consultants (Senior Project Manager - Michael Graber)	
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Project Information

Project Name: City Lake Dam Rehabilitation & Enlargement

Brief Description of Project: (Attach separate sheets if needed)

This project addresses dam safety deficiencies at the Walsenburg City Lake Dam and Reservoir (City Lake Dam) identified in the Colorado Office of the State Engineer's (SEO) Inspection Report dated July 9, 2014 and the SEO-ordered Dam Safety Compliance Plan dated September 9, 2014. The construction project will both rehabilitate City Lake Dam and increase the current reservoir storage capacity by approximately 120 ac-ft to address water demand of the customers served by the system. Other costs included below are contingencies and loan origination.

General Location: (Attach Map of Area)

Walsenburg Reservoir (also known as City Lake) is located west of Walsenburg and just north of the Cucharas River.

Estimated Engineering Costs: $963,578	Estimated Construction Costs: $5,138,891
Other Costs (Describe Above): $787,726	Estimated Total Project Costs: $6,869,195
Requested Loan Amount: $6,821,000	Requested Loan Term (10, 20, or 30 years): 30 Years

Project Start Date(s) Design: January 2017 Construction: November 2017

Signature

[signature] MAYOR 3/21/17
Signature / Title Date

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INTRODUCTION

This Feasibility Evaluation has been prepared in general accordance with the Colorado Water Conservation Board (CWCB) *Water Project Loan Program Guidelines*, Revised January 2006.

The City of Walsenburg (City) and other consultants working directly with the Owner have prepared work in the following outline sections and this work is presented in an italic font:

- *Section 1 - Introduction and Background*
- *Section 2 - Sponsor*
- *Section 3 - Water Rights and Water Demands*
- *Section 4 - Analysis of Alternatives*
- *Section 10 - Impacts*
- *Section 12 - Financial Plan*

RJH Consultants, Inc. (RJH) has prepared work in the following outline sections and this work is presented in a non-italic font:

- Section 1 - Introduction and Background
- Section 5 - Selected Alternative
- Section 6 - Subsurface Investigation and Site Stratigraphy
- Section 7 - Typical Embankment Section, Seepage and Stability Analysis
- Section 8 - Hydrologic Evaluation
- Section 9 - Hydraulic Structures
- Section 11 - Opinion of Probable Project Cost
- Section 13 - Implementation Schedule
- Section 14 - Limitations
- Section 15 - References

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SECTION 1 - INTRODUCTION

1.1 Purpose

*Walsenburg City Lake Dam and reservoir serves as the primary water supply for the **WTP** that is located approximately 500 feet east of the east dam embankment. Since September 9, 2014, City Lake Dam has been under a State Engineers Office (SEO) dam safety compliance plan due to identified dam safety deficiencies. The dam safety compliance plan was subsequently revised on February 21, 2017 and requires Walsenburg to complete a number of actions by specific deadline dates to improve the safety of the dam. The dam safety compliance plan imposes a 1-foot storage restriction beginning April 1, 2017, a 2-foot storage restriction on November 15, 2017, and a 3-foot storage restriction on May 1, 2019.*

*The City of Walsenburg will not be able to adequately supply the **WTP** and supply peak water demands within the City without a full City Lake Reservoir. The project to rehabilitate the dam and maintain full reservoir storage is extremely important to City for this reason.*

1.2 Objectives

The objectives of this report are to present the 30 percent complete design of the selected alternative, an opinion of probable costs, and a proposed implementation schedule for the Walsenburg City Lake Dam Rehabilitation and Enlargement for use in obtaining a Colorado Water Conservation Board (CWCB) loan. The selected alternative and cost opinion presented in this report were developed to enable an evaluation of the project technical and permitting requirements and associated costs. The selected alternative will be refined during final design based on additional investigations and analyses specific to the selected alternative. These specific analyses may result in modifications to the concepts presented in this report. Supporting calculations for the alternative included in final design will be developed and presented in future design reports.

1.3 Scope of Work

RJH Performed the following scope of work:

- Developed maps of the Project that include proposed and existing components, floodplains, and inundation zones.

- Developed preliminary level sizes and layouts for the reconstructed and raised dam embankment and foundation, outlet works, service spillway, emergency spillway, and reservoir bypass pipeline. Initial layouts, dam embankment alignments, and embankment sections were developed based on previously completed project work, judgment, general design criteria, and experience. Specific investigations and analyses were performed to support the relative size of facilities and dam embankment sections.
- Developed a modified dam embankment alignment for the new raised embankment section.
- Prepared a preliminary elevation-capacity curve.
- Developed typical dam embankment sections for and performed seepage and stability analyses of the maximum section to ensure the sections meet required seepage and stability design criteria.
- Performed wave run-up analysis to select the required freeboard height.
- Performed a hydrology analysis for use in developing the inflow design flood (IDF). Used this IDF and performed reservoir routing and peak discharge to support selection of the required spillway size.
- Described previous field investigations, methodology, site geology and geologic mapping, availability of construction materials, and subsurface investigations.
- Identified probable coordination required with federal, state, and local agencies.
- Estimated quantities of primary materials required for construction and prepared an overall opinion of probable project costs (OPPC) to construct the Project.
- Developed a Project schedule for design and construction of the dam and appurtenant facilities.
- Performed a preliminary permitting assessment to identify major technical permits and permitting issues for the Project. Identified anticipated required technical permits (i.e., local, state, federal).

1.4 Project Personnel

The following personnel from RJH are responsible for the work contained in this report:

Project Manager	Michael L. Graber, P.E.
Project/Lead Geotechnical Engineer	Edwin R. Friend, P.E., PMP
Hydraulic Engineer	Eric M. Hahn, P.E.

Geotechnical Engineer	Kevin T. Mininger, P.E.

SECTION 2 - SPONSOR

2.1 Sponsor

The City is an incorporated Statutory City that is the county seat and the most populous city of Huerfano County, Colorado. Walsenburg extends water and sewer service to City residents and adjacent County residents. Walsenburg codified and obtained significant water rights early in the development of the City and has continued to acquire additional water rights. These early and wise water acquisitions by City decision-makers has ensured Walsenburg sufficient water to serve significant future growth. Walsenburg is a participating member of the Huerfano County Water Conservancy District that is currently developing a long-term water storage and distribution plan for Huerfano County.

The City supplies potable water for a population of approximately 2,900 residents having approximately 1,700 taps for residential, business, municipal, and industrial uses. The WTP is located approximately 3 miles west of the City and a short distance downstream of City Lake Dam. Treated water is pumped into a storage tank adjacent to the plant and distributed to the water taps in the City by way of a network of underground pipes.

SECTION 3 - WATER RIGHTS AND WATER DEMANDS

3.1 Water Rights and Water Demands

The original storage right for City Lake was decreed in Civil Action No. 1414, which was adjudicated on 10/3/1921 with an appropriation date of 5/2/1904. The decreed storage capacity of the reservoir is 411.5 acre feet. The storage right for City Lake was decreed in Civil Action No. 1414, which was adjudicated on 10/3/1921 with an appropriation date of 5/2/1904. The decreed storage capacity is 411.5 acre-feet (ac-ft). The City has the right to store the water rights tabulated in Table 3.1 in City Lake pursuant to the decrees entered in Civil Action Nos. 3266 and 3848. These rights are diverted at the Walsenburg Pipeline. The calculated average annual inflow into City Lake from Walsenburg Pipeline diversions over the last 5 years (2012 – 2016) is 2,622 ac-ft/year (ranging from 2,375 ac-ft/year to 2,931 ac-ft/year). Water is released from City Lake to the WTP as well as the Coler System Reservoirs – Lake Miriam Reservoir (a/k/a Horseshoe Reservoir) and Lake Oehm Reservoir (a/k/a Martin Lake) – via the Lake Miriam Ditch (a/k/a Coler Inlet Ditch).

The City changed its 1/6th interest in the Gomez Ditch in Case No. 11CW56, which listed Walsenburg Reservoir (City Lake) as a new place of storage for the City's Gomez Ditch water rights. This source of supply will be available for filling the enlarged capacity in City Lake to the extent the depletion credits can be diverted at the Walsenburg Pipeline. The decreed average historical consumptive use for the City's Gomez Ditch water rights is 37.4 ac-ft/year for Priority No. 10 and 4.8 ac-ft/year for Priority No. 124.

The average daily potable water demand from the WTP varies but is approximately 2.5 million gallons per day (mgd). The remainder of the diverted water delivered by the Walsenburg Pipeline is stored in City Lake and once City Lake is filled to capacity, spills into the Coler ditch and is delivered to Horseshoe and Martin Lakes, where the water is used for recreation and other municipal uses.

TABLE 3.1
WATER RIGHTS DECREED IN CIVIL ACTION NOS. 3268 AND 3848 AND CASE NO. 11CW56

Case No.	Priority No.	Name of Ditch	City-Owned Amount (cfs)
CA 3266 and CA 3848	1	Francisco & Daigre Mill Ditch No. 1	0.2917
	2	Calf Pasture Ditch No. 2	0.5000
	3	Francisco & Daigre Mill Ditch No. 1 (1st Enl)	4.0833
	4	Guillen Ditch No. 4	1.0000
	4	Guillen Ditch No. 4	0.5000
	17	Calf Pasture Ditch No. 2	0.5000
11CW56	10	Gomez Ditch	0.5333
	124	Gomez Ditch	1.1667

SECTION 4 - ANALYSIS OF ALTERNATIVES

4.1 Alternatives

Complying with the SEO mandated dam safety compliance plan and maintaining full reservoir storage to supply the WTP and satisfy peak water demands within the City is the only reasonable long term alternative for suppling the City's potable water needs. The City owns other storage reservoirs but none can readily supply the WTP without the construction of extensive pumping and underground delivery systems. The long-term operation and maintenance on the pumping systems from other reservoirs would be cost-prohibitive compared to the all gravity feed water supply provided by City Lake Dam. Long-term loss of reservoir storage in City Lake Dam could also result in the loss of the valuable storage right, which would result in a tremendous financial loss to the City.

SECTION 5 - SELECTED ALTERNATIVE

5.1 Site Description and General Project Location

City Lake Dam is located in central Huerfano County, approximately 4 miles southwest of Walsenburg, Colorado, immediately south of US Highway 160 and approximately 1 mile north of the Cucharas River in Section 23, Range 67 West, Township 28 south of the Sixth Principal Meridian. The site topography is generally flat and gently slopes to the southeast toward the Cucharas River channel. A juniper-pinion forest extends along the west reservoir rim with the remainder of the site having prairie grasses with some wetlands vegetation. The general site location is shown on Drawing 1, 30 Percent Design Drawing Set, Appendix B.

City Lake Dam and Reservoir provides a stable water supply to the WTP located approximately 500 feet east of the eastern dam embankment. Water to the treatment plant is delivered by way of two parallel outlet conduits from the reservoir basin. Water is supplied to the reservoir from two underground conduits discharging from Wahatoya and Daigre Reservoirs located about 9 miles southwest of City Lake on the eastern city limits of La Veta, CO.

According to previously completed investigations and analyses performed by RJH and inspections performed the SEO found the City Lake Dam did not meet minimum required factors of safety for both seepage and stability. The emergency spillway was also found to have inadequate capacity to route the required IDF. Based on these findings, the SEO implemented a Dam Safety Compliance Plan (Appendix G) that required the City of Walsenburg (City) to take definitive actions having milestone completion dates to improve the safety of the dam. Based on the requirements contained in the SEO Dam Safety Compliance Plan, RJH developed alternatives to rehabilitate the existing dam at the current storage level and presented those alternatives in a Conceptual Design Report (Appendix D) dated April 2015. The Conceptual Design Report presented two embankment rehabilitation alternatives, three outlet and principle spillway rehabilitation alternatives, and two auxiliary spillway alternatives for rehabilitation. Subsequent to completing the Conceptual Design Report, the City obtained additional water rights and identified the need for additional reservoir storage in the amount of 120 ac-ft. Based on this need, RJH developed a 30 percent complete design to address the dam safety concerns and raise the existing dam embankment height 3 vertical feet to provide the needed additional storage. A general plan of the primary project components is shown on Drawing 6, 30 Percent Design Drawing Set, Appendix B and the basis for the 30 percent design is presented in the 30 Percent Design Report, Appendix A.

The existing dam embankment will be removed down to foundation bedrock, stockpiled, blended with imported borrow embankment material, and placed back generally along the existing dam alignment except on the north end adjacent to US Highway 160, where the alignment will shift toward the south to provide necessary clearance from overhead power lines and highway right-of-way. The new dam embankment will encompass approximately two-thirds of the reservoir, have an approximate length of 4,100 feet and vary in height from a few feet to approximately 23 feet at the maximum section. The dam embankment will create a reservoir surface area at the normal high water elevation of 6522.5 feet of approximately 50 acres having a total reservoir storage volume of 601 ac-ft. The new homogenous embankment will have 2:1 upstream and downstream slopes, a 14.5-foot crest width, rock riprap upstream erosion slope protection and incorporate an internal filter and drainage system to safely manage reservoir seepage.

A new outlet works and principle service spillway will be constructed on the east side of the dam embankment. This structure includes a low-level ungated 24-inch-diameter intake conduit from the inlet structure to a combination concrete gate tower and service spillway; two 18-inch-diameter outlet conduits with manually-operated slide gates, one controlling the flow to the WTP and the other providing a drain capability to the Coler Ditch, east of the reservoir. The service spillway will have an uncontrolled weir that will maintain the normal maximum reservoir water surface elevation of 6522.5 feet under normal inflow conditions. Water flowing over the service spillway will flow into the north outlet conduit and into the Coler Ditch where it will flow to Horseshoe Reservoir, located north of US Highway 160, within the boundary of Lathrop State Park. An 8-inch-diameter pumpback conduit from the Coler Ditch to the gate tower will also be constructed to allow excess flows in the Coler Ditch to be pumped into the reservoir.

A new 50-foot-wide open channel emergency spillway will be constructed at the southeast corner of the reservoir. The new emergency spillway will have a concrete wall control section with a flowline at Elevation (El.) 6523.0, which is 6 inches above the service spillway flowline, grouted riprap-lined channel on the embankment downstream slope, and a riprap-lined discharge channel. The spillway will discharge across a county road and into an arroyo that drains southeastward back to the Cucharas River. The flow path for the new emergency spillway generally matches the existing emergency spillway flow path. The extent of a maximum emergency spillway discharge inundation flow path is shown on Figure 5.1.

A bypass conduit from the reservoir water supply conduit to the existing north 18-inch-diameter conduit supplying raw water to the potable WTP will be constructed from the reservoir supply waterline discharge point on the west side of the reservoir across the drained reservoir basin. Water supply to the WTP will be maintained through this conduit during the construction process. While the reservoir is being drained and the bypass line constructed, the City will pump water from Martin Lake, located north of the project in Lathrop State Park, through an existing 8-inch-diameter conduit to the WTP. The pumped water supply from Martin Lake combined with the City's potable water storage will provide sufficient time to drain the lake and construct the bypass conduit without resulting in a disruption of potable water service to the City service area.

5.1.1 Elevation-Capacity Table

The reservoir water surface area, elevation-storage capacity for the reservoir, is provided in the 30 Percent Design Report, Appendix A. An approach channel will be constructed from the low point in the reservoir basin to the 24-inch-diameter outlet works intake conduit that will allow complete draining of the reservoir. The reservoir will have an approximate total storage volume of 601 ac-ft at the normal maximum water surface El. 6522.5 and a water surface area of 50 acres.

5.1.2 Right-of-Way and Land Purchase

The increased reservoir normal water surface elevation will extend onto private property on the northwest side of the reservoir. The City has discussed the enlarged reservoir water surface with the land owner and he is willing to provide an easement onto his property for the encroaching water surface. The extent of the water surface encroachment onto private property is shown on Drawing 6, 30 Percent Design Drawing Set, Appendix B.

The current emergency spillway has an access road to the west side of the reservoir where the supply pipeline delivers water to the reservoir. The new auxiliary emergency spillway will prevent vehicle access from the dam crest across the spillway and will not allow continuation of the existing access road to the west side of the reservoir. There are a number of undeveloped subdivision lots located on the south reservoir boundary and west of the spillway, one of which could be used to construct a new access road to the west side of the reservoir. A purchase of one of these lots is an anticipated requirement for a new access road. The subdivision lots on shown on Figure 5.2.

An easement or purchase of the privately owned land adjoining the reservoir property on the northeast side, south of US Highway 160 and west of Spanish Peaks Drive will be required to stockpile removed embankment material and for processing and blending imported embankment material. The City has discussed an easement for this purpose with the land owner and he is agreeable with an easement for this purpose. The private property is shown on Drawing 6, 30 Percent Design Drawing Set, Appendix B.

REPRODUCED IN COLOR

RJH CONSULTANTS, INC.

WALSENBURG CITY LAKE DAM REHABILITATION AND ENLARGEMENT

PROJECT NUMBER 14120

CITY LAKE DAM SPILLWAY INUNDATION PLAN

March 2017

Figure 5.1

P:\14120 - Walsenburg City Lake Dam and Reservoir\Engineering\Task_4001_30%Design\Spillway\Figure 5.1_Spillway Inundation Area Map.mxd

PROPERTY TO BE ACQUIRED

EASEMENT TO BE ACQUIRED

N

0 50 100 200 300
SCALE IN FEET

Walsenburg

RJH CONSULTANTS, INC.

WALSENBURG CITY LAKE DAM REHABILITATION AND ENLARGEMENT

PROJECT NO. 14120

ADJACENT PROPERTY TO BE ACQUIRED

March 2017

Figure 5.2

P:\14120 – WALSENBURG CITY LAKE DAM AND RESERVOIR\CAD\TASK 4001\LOAN APPLICATION FIGURES\FIGURE 5.2–ADJ PROP TO ACQUIRE.DWG 3/23/2017 12:03 PM

SECTION 6 - SUBSURFACE INVESTIGATIONS AND SITE STRATIGRAPHY

6.1 RJH Investigation

RJH performed a geotechnical investigation of the existing dam embankment and foundation between September 29 and October 3, 2014 and a supplemental investigation around the reservoir perimeter and potential borrow area on February 23, 2016.

A total of seven borings were advanced during the initial subsurface investigation program. Six borings were advanced along the embankment crest and one boring was advanced on the lower downstream slope of the embankment, where additional fill may have been placed by the City. Drilling was performed by Elite Drilling, Inc. (Elite) of Denver, Colorado under subcontract to RJH. An RJH engineer observed drilling procedures, visually classified soil and rock samples, prepared a field log of each boring, recorded in-situ permeability test data, and observed and recorded relevant drilling information. A summary of the borings and the boring locations is provided in the Geotechnical Data Report and Addendum, Appendix C.

Subsequent to the initial subsurface investigation, a second field investigation was performed that included three test pits excavated around the reservoir perimeter and four test pits in a potential borrow area north of US Highway 160 on February 23, 2016. Test pits were excavated with a rubber-tired excavator that was provided by the City and operated by City personnel. Test pits were backfilled with excavated materials. A summary of the test pits is provided in the Geotechnical Data Report and Addendum, Appendix C.

6.1.1 Drilling/Sampling Methods

Borings were advanced using a truck-mounted CME 75 drill rig with an automatic hammer. Borings were advanced through soil and weathered bedrock using hollow-stem augers with an inside diameter (I.D.) of 4.25 inches and an outside diameter (O.D.) of approximately 8.25 inches. Within bedrock, borings were advanced using NQ-sized (2.980-inch O.D., 1.875 I.D.) continuous wireline coring.

Samples of surficial soils were collected using a 3.0-inch I.D. continuous tube sampler system (CME sampler) and a standard split-spoon sampler (2.0-inch O.D.). The split-spoon samplers were driven with an automatic-trip 140-pound hammer dropped 30 inches. Upon completion, the borings were backfilled with cement-bentonite grout or completed as monitoring wells.

6.1.2 Excavation Methods

The test pits were excavated with a rubber-tired backhoe with a 2-foot-wide bucket. Total depths ranged from 4.0 to 12 feet below the ground surface and were generally about 2 feet wide and 12 feet long. Each test pit was backfilled in approximate 1-foot-thick lifts with the excavated material and consolidated with the backhoe bucket upon completion. Each test pit location was recorded using a handheld GPS unit.

6.1.3 Logging Method

Soil samples were classified in the field in general accordance with ASTM D 2488 (visual-manual classification). Rock samples were identified in general accordance with the U.S. Bureau of Reclamation (Reclamation) *Engineering Geology Field Manual* (Reclamation, 2001). Classifications and field boring logs were reviewed by a senior geological engineer for quality control. Following laboratory testing, laboratory index test results were compared to field log classifications and, as appropriate, logs were modified according to ASTM D 2487 (Unified Soil Classification System (USCS)). Lithology between recovered samples is interpreted. As appropriate, RJH field staff utilized indirect observations (e.g., drill chatter, drilling resistance, etc.) to interpret unobserved lithology.

Collected soil and rock samples were packaged and transported in general accordance with ASTM D 4220. Recovered split-spoon samples were placed in sealed plastic bags. Continuous samples were wrapped in plastic, placed in 4-inch split polyvinyl chloride (PVC) tubing, and wrapped in duct tape. Samples selected for laboratory testing were placed in sealed plastic bags to help preserve the in-situ moisture content.

For the test pits, stratigraphy changes were measured from the middle of the trench (length wise). For each different type of material identified during excavation, the field engineer recorded the classification and depth interval, and placed a representative sample in a large bulk sample bag.

6.1.4 Laboratory and Field Testing

Geotechnical laboratory testing was performed on representative samples of soil and rock samples collected during the investigations by Advanced Terra Testing (ATT) of

Lakewood, Colorado. Laboratory testing included index testing, crumb dispersion testing, and unconfined compressive strength testing. A summary of the test results are included in Geotechnical Data Report and Addendum, Appendix C.

Field tests included standard penetration resistance tests in the embankment and foundation soils and hydraulic conductivity tests (Packer tests) in competent bedrock.

6.2 Site Geology

The Project is located within the Raton Section of the Great Plains Province of the Interior Plains Physiographic Region according to the U.S. Geological Survey (USGS) (USGS, 2000). The Raton Section is generally characterized by younger volcanic rocks (4 thousand to 26 million years in age) overlying older sedimentary rock (between Miocene and Cretaceous Period). Toward the north, where the Site is located, the landscape Raton Section is described as a nearly flat plateau cut on Cretaceous rock, and surmounted by young volcanic vents, cones, and lava fields (Trimble, 1980).

Bedrock at the Site is mapped within the Poison Canyon Formation of the Paleocene Age (53 to 65 million years in age). The Poison Canyon Formation is characterized by buff to red arkosic sandstone and conglomerate, and yellow shale. Thickness of the unit is between 2,000 and 2,500 feet. Based on a generalized section of the upper formations in the Walsenburg area, alluvium was deposited during the Quaternary Period (1.6 million years ago to the present) and is between 0 to 10 feet thick (Johnson and Stephens, 1951).

6.2.1 Embankment Fill

Embankment fill was encountered at the dam crest in each boring and ranged from 4.5 to about 23.0 feet in thickness. The different soil units appeared to be randomly distributed and could not be correlated laterally between borings.

Embankment fill predominantly consisted of coarse grained soils with varying amounts of fines. The coarse grained soils classified as clayey sand, clayey sand with gravel, silty sand, silty sand with gravel, well-graded sand with clay and gravel, and poorly-graded sand with clay. The coarse grained soils contained 0 to 40 percent gravel, 35 to 95 percent sand, and 5 to 50 percent non-plastic to medium plasticity fines. Three samples of coarse grained embankment fill had liquid limits that ranged from 25 to 35, and averaged 29, and plasticity indices that ranged from 12 to 23, and averaged 16. The maximum recovered particle size was 1.5 inch. SPT N-values in embankment fill ranged from 0 to 22, and averaged 8. Based on N-values, the coarse grained embankment fill was typically very loose to medium dense.

One zone of fine grained soil classifying as sandy lean clay was also encountered in boring B-105. The sandy lean clay contained no gravel, 15 to 25 percent sand, and 75 to 85 percent low plasticity fines. Fine grained embankment fill was very soft to soft.

Embankment fill was dry to moist above the water table and moist to wet below the water table. The moisture content of two samples obtained from above the groundwater table ranged from 5.7 to 12.8 percent, and averaged 9.3 percent. The moisture content of four samples obtained from below the groundwater table ranged from 20.5 to 26.4 percent, and averaged 22.4 percent. The field moisture content may be higher for coarse grained fill because some drainage from the samples likely occurred prior to packaging.

6.2.2 Alluvium

Aalluvium was generally similar to the embankment fill. Both coarse grained alluvium and fine grained alluvium were encountered; however, the soils were predominantly coarse grained with varying amounts of fines. The different soil units appeared to be randomly distributed and could not be correlated laterally between borings. Individual soil layers ranged from 0.9 to 5.3 feet thick. Alluvium was encountered below the embankment fill in each boring and ranged from less than 2 feet to about 7.5 feet thick.

The coarse grained alluvium consisted of clayey sand, silty sand, and poorly graded sand with clay. The coarse grained alluvium contained 0 to 5 percent gravel, 70 to 95 percent sand, and 5 to 25 percent non-plastic to medium plasticity fines. SPT N-values in alluvium ranged from 1 to 39, and averaged 9. Based on N-values, the coarse grained alluvium was very loose to dense.

The fine grained alluvium consisted of sandy lean clay and sandy fat clay. The fine grained alluvium contained no gravel, 20 to 45 percent sand, and 55 to 80 percent low to high plasticity fines. One sample of fine grained alluvium had a liquid limit of 49 and plasticity index of 34. Fine grained alluvium was very soft to medium stiff.

The alluvium was dry to moist above the groundwater table and moist to wet below the groundwater table. The moisture content of one sample obtained from below the groundwater table was 24.7 percent.

6.2.3 Poison Canyon Formation

Bedrock encountered at the Site was the Poison Canyon Formation. Bedrock generally consisted of sandstone, siltstone, and claystone in layers from less than 0.5 foot up to about 10 feet thick. Bedrock was encountered below alluvium in all borings. Bedrock was encountered in the borings between about Elevation (El.) 6497.0 and El. 6510.6.

Sandstone consisted of mostly fine to medium grained sand with less than 15 percent non-plastic fines. Silty and clayey sandstone consisted of mostly fine grained sand and 15 to 25 percent non-plastic to low plasticity fines.

Siltstone consisted of mostly non-plastic to low plasticity fines with less than 15 percent fine grained sand. Sandy siltstone consisted of mostly non-plastic to low plasticity fines with 15 to 50 percent fine grained sand.

Claystone and silty claystone consisted of mostly low to medium plasticity fines with less than 15 percent fine grained sand. Sandy claystone consisted mostly of low to medium plasticity fines and 15 to 30 percent fine grained sand. Two samples of claystone had liquid limits that ranged from 30 to 40, and averaged 35, and plasticity indices that ranged from 14 to 18, and averaged 16.

Shale consisted of low to medium plasticity fines with no observed sand.

Bedrock ranged from fresh to intensely weathered, and slightly fractured to very intensely fractured, and was generally intensely weathered and intensely fractured. Fractures in bedrock were generally open to tight, moderately rough, and clean to moderately thin iron staining with occasional calcite infilling. Slickensides were generally observed throughout the fine grained bedrock.

The hardness of bedrock ranged from H4 to H7. The sandstone was typically H5 to H6 and occasionally H4. Claystone and siltstone were typically H7.

Core recovery ranged from 40 to 100 percent, and averaged about 88 percent. Rock quality designation (RQD) ranged from 0 to 68, and averaged 33.

Three SPT tests in bedrock had N-values ranging from 26 to 56 and averaged 46. Five of the eight SPT tests performed in bedrock resulted in refusal.

The moisture contents of two bedrock samples tested ranged from 10.7 to 15.4 percent. The unconfined compressive strength of two samples of sandstone was 2,070 and 8,840 pounds per square inch (psi), and averaged 5,455 psi. Two samples of interbedded sandy claystone and claystone had unconfined compressive strengths of 30 and 70 psi, and averaged 50 psi.

Three Packer tests in bedrock resulted in no flow conditions and were assigned nominal permeabilities of 0.1 Lugeon (1.0×10^{-7} cm/sec). Seven tests had laminar, washout, or void filling flow conditions and calculated permeabilities ranged from 0.1 to 55 Lugeons (1.0×10^{-7} to 6.3×10^{-4} cm/sec). The geometric mean of the seven permeability tests where flow was measured was 3.4 Lugeons (1.9×10^{-5} cm/s).

6.2.4 Groundwater

RJH encountered groundwater in all borings. Groundwater was encountered as shallow as 5.4 feet in B-102(P) and as deep as 14.0 feet in B-103, ranging between El. 6499.0 and El. 6518.0.

6.3 Availability of Construction Materials

Based on subsurface investigation, testing, and analysis, the material in the existing embankment after processing appears to be suitable for reuse and reconstructing and raising the dam embankment. The moisture content of the existing embankment is significantly above optimum and will require blending and processing to reach optimum moisture content prior to placing in the new reconstructed embankment. The existing dam embankment will be removed down to bedrock and stockpiled. Imported borrow material will then be blended with the stockpiled embankment and processed so that moisture content is at optimum and the fine grained material is more uniformly graded for placement and compaction back into the reconstructed embankment fill. Sand for filter drains, riprap bedding, riprap slope protection, and roadway gravel surfacing will all require import to the Site. These materials are all commercially available. The roadway gravel surfacing is available in the Walsenburg area, the filter sand and riprap bedding is available in the Pueblo area, and the closest known suitable riprap source is the Tezak Rock Quarry, located north of Penrose approximately 100 miles distance from the Site.

SECTION 7 - TYPICAL EMBANKMENT SECTION, SEEPAGE AND STABILITY ANALYSIS

7.1 General

Initial seepage and stability analyses for the existing embankment and proposed rehabilitation alternatives for maintaining current embankment height and reservoir storage were performed for the April 2015 Conceptual Design Report contained in Appendix D. These analyses were updated for the raised embankment height and modified embankment sections for the 30 Percent Design Report, Appendix A. The reconstructed and raised City Lake embankment dam will enclose approximately two-thirds of the normal maximum water surface and will vary in height from a few feet on both ends of the embankment to approximately 23 feet at the maximum section. The upstream embankment slope will have graded rock riprap erosion protection from the dam crest, El. 6328.0, down to El. 6518.0. The downstream slope will be seeded with native grass.

Reservoir seepage through the embankment and foundation will be managed, collected, and safely conveyed to multiple daylight discharge points with an internal drainage system that includes an embankment chimney drain, blanket drain, and toe drain. The embankment will be approximately 4,100 feet long, creating a reservoir surface area of approximately 49 acres, having a total storage capacity of 601 ac-ft at a normal maximum pool at El. 6522.5.

A plan of the new reconstructed embankment is shown on Drawing 6 and typical sections on Drawing 10, 30 Percent Design Drawing Set, Appendix B.

7.2 Typical Embankment Sections

Three typical embankment sections were developed based on the varying embankment heights. All sections have 2:1 upstream and downstream slopes, 14.5 feet crest widths, and will have 10-foot-wide key trenches excavated 3 feet into bedrock. Section 1 represents embankment heights of generally 5 feet or less having only 2 feet of water at normal maximum pool against the embankment. Sections 2 and 3 incorporate internal drainage and filter systems to include a chimney drain, blanket drain, and toe drain. Seepage and slope stability analyses were performed on Section 3 that represents the maximum section. Typical embankment sections are located on Drawing 10, 30 Percent Design Report, Appendix A.

7.3 Slope Stability

Two-dimensional stability analyses were performed to document the slope stability factor of safety for the embankment geometry. Critical failure surfaces and minimum safety factors were computed using Spencer's Method, which considers moment and force equilibrium. We used the computer program Slope/W© 2007 to perform the iterative task of locating the critical failure surfaces and calculating the minimum safety factors.

Slope stability analyses were performed for the maximum section of the embankment dam for steady state seepage and rapid drawdown conditions. The rapid drawdown load condition represents a condition where the reservoir is rapidly lowered from maximum normal pool. The preliminary rapid drawdown analyses are based on a conservative assumption that drawdown is instantaneous and that drainage (pore pressure dissipation) does not occur in the fine grained embankment materials.

Material properties for seepage and stability analyses utilized material properties developed in the Conceptual Design Report (RJH, 2015). Material properties were developed from in-situ tests, laboratory tests, calibration to observed field conditions, and engineering judgement. Material properties used for the seepage and slope stability modeling are presented in Table 7.1.

TABLE 7.1
MATERIAL PROPERTIES

Material Property	Alluvium	New Embankment Fill	Bedrock	Filter
Dry Unit weight (pcf)	110	110	-	115
moist unit weight (pcf)	126.5	126.5	127	136
Effective Friction Angle	33	33	27	37
Effective Cohesion (psf)	0	0	0	0
Residual Strength Φ_r	-	-	19	-
Undrained Strength (psf)	500	1000	2,160	0
Vertical Permeability cm/s (ft/s)	1E-5 (3.3×10^{-7})	1E-5 (3.3×10^{-7})	4E-6 (1.3×10^{-7})	1E-1 (3.3×10^{-4})
Horizontal to Vertical Permeability Ratio	4	4	1	1
Horizontal Permeability cm/s (ft/s)	4E-5 (1.3×10^{-6})	4E-5 (1.3×10^{-6})	4E-6 (1.3×10^{-7})	1E-1 (3.3×10^{-4})

7.4 Embankment Design - Changes from Conceptual Design

The embankment section used for 30 percent design seepage and stability analyses was similar to the one used during conceptual design except:

- The embankment crest was raised 3 feet from El. 6525.0 to El. 6528.0.
- The crest width was increased from 14.0 feet to 14.5 feet to meet SEO requirements for the taller embankment.
- The upstream and downstream toe elevations were lowered to be representative of the ground surface at the maximum section.
- The reservoir elevation was raised from El. 6519.5 to El. 6522.5.

Table 7.2 includes a summary of the analyzed geometry.

TABLE 7.2
ANALYZED EMBANKMENT GEOMETRY

Element	Geometry for Conceptual Design Analyses	Geometry for 30 Percent Design Analyses
Crest Elevation	6525	6528
Crest Width	14.0	14.5
Embankment Slope Angles	2H:1V	2H:1V
Upstream Toe Elevation	6507.0	6505.0
Downstream Toe Elevation	6509.5	6506.0
Reservoir Elevation	6520.0[(1)]	6522.5

Notes:
1. The existing reservoir level is El. 6519.5, but was modeled slightly higher during conceptual design.

The ground surface upstream and downstream of the embankment was lowered an equal amount as the toe to maintain the same slope used during conceptual design analyses. The top of rock below the embankment was also adjusted so the thickness of alluvium under the embankment was representative of the thickest alluvium identified in the two closest borings to the maximum section. Additional embankment stability and seepage analyses detail is provided in the 30 Percent Design Report, Appendix A.

Two-dimensional stability analyses were performed to evaluate the static stability of the embankment. One typical section was used at the maximum embankment height. Embankment slope stability was evaluated for the loading conditions and bedrock strengths summarized in Table 7.3 and results were compared to results from conceptual design.

TABLE 7.3
LOADING CONDITIONS AND BEDROCK STRENGTHS FOR STABILITY MODELING

Slope Analyzed	Loading Condition	Bedrock Strength
Downstream	Steady State	Peak
Downstream	Steady State	Residual
Upstream	Rapid Drawdown	Peak
Upstream	Rapid Drawdown	Residual
Downstream	End of Construction	Peak
Upstream	End of Construction	Peak

During conceptual design, the upstream slope stability was evaluated under steady state conditions with the reservoir empty. Analysis results of this loading condition do not impact design and are not discussed further because this analysis is not required by the SEO and, based on RJH's understanding of expected reservoir operations, the reservoir is not expected to remain empty long enough for steady state conditions to develop.

Increasing the embankment height 3 feet reduced the factor of safety for all evaluated loading conditions, but the factors of safety remained above the minimum factors of safety specified by the SEO or selected by RJH. A summary of the stability analyses results is provided in Table 7.4.

TABLE 7.4
RESULTS OF STABILITY ANALYSES

Design Phase	Loading Conditions	Slope Analyzed	Bedrock Strength	Calculated Factor of Safety	Minimum Factor of Safety[1]	Comments
30 Percent Design	Steady State	DS	Peak	1.61	1.5	Failure through bedrock
30 Percent Design	Steady State	DS	Residual	1.34	1.1	Failure through bedrock
30 Percent Design	Rapid Drawdown	US	Peak	1.43	1.2	Failure through bedrock
30 Percent Design	Rapid Drawdown	US	Residual	1.03	1.01	Failure through bedrock
30 Percent Design	End of Construction	DS	Peak	1.59	1.3	Failure not through bedrock
30 Percent Design	End of Construction	US	Peak	1.63	1.3	Failure not through bedrock

Note:
1. The minimum recommended factors of safety are based on the Colorado *Rules and Regulations for Dam Safety and Dam Construction* (SEO, 2007) and our experience.

RJH also performed a parametric sensitivity analyses of the strength of the fill and alluvium. Based on these analyses, the steady state embankment and alluvium strength could be as low as 31 degrees and not adversely impact the embankment stability factors of safety.

Computer model outputs showing embankment sections, materials properties, and critical failure surfaces are presented in the 30 Percent Design Report, Appendix A.

7.5 Seepage Analysis

Two-dimensional seepage analyses were performed to estimate seepage quantity through and below the embankment, seepage forces, exit gradients, and to establish existing groundwater conditions.

The permeability of the various materials were based on field and laboratory test results, calibrations to observed field conditions, and experience. Anisotropy of the various embankment and foundation materials were developed based on engineering judgment. RJH developed hydraulic conductivity functions based on Soil-Water Characteristic Curves presented in the SEEP/W manuals and the design values for saturated permeability.

A combination chimney, blanket and toe drain will be constructed downstream of the dam embankment centerline. The chimney drain width of 3.0 feet was selected based on constructability. The total seepage collected by the toe drain is expected to be between about 10 and 15 gallons per minute (gpm). Based on results from the seepage analysis at the maximum embankment section, the capacity of the drain is estimated to be about 25 times the amount required to carry flow into the drain. Seepage into the chimney drain will be conveyed through the blanket drain to the toe drain and a slotted 8-inch-diameter drain pipe. The toe drain pipe will discharge to established and constructed wetlands areas downstream of the dam.

Using the results from the seepage analyses, we estimated total losses through the dam embankment and foundation by taking the results from the two-dimensional seepage analysis times the approximate length of the dam, which is conservative. Annual losses could be up to about 15 gpm or 24 ac-ft per year (ac-ft/yr) if the reservoir is maintained at normal pool for the entire year. These amounts are for seepage through the dam embankment and foundation only and do not include evaporation losses or seepage through the bottom of the reservoir basin. Seepage analyses details are located in the 30 Percent Design Report, Appendix A.

7.6 Slope Protection

The interior slope of the dam will need to be protected from erosion caused by wave action. Typically upstream slopes of dam embankments requires slope protection to maintain embankment integrity and significantly reduce wave erosion. For this Project, graded rock riprap was selected because it is a proven and effective slope protection method and is commercially available within a reasonable haul distance.

Under normal reservoir operations, the reservoir will be kept full to the service spillway El. 6225.5 to supply a constant pressure through the reservoir outlet conduit to the WTP. For this reason, RJH proposed extending the riprap slope protection from the dam crest El. 6528.0 down 10 vertical feet to El. 6518.0. If the reservoir is to be operated below El. 6520.5, the riprap slope protection will need to extend to a lower elevation. The SEO has agreed in principle with this 30 percent design slope protection concept. RJH performed wave run-up analysis and riprap design to identify the required gradation of riprap and riprap bedding materials using (Reclamation 2012 and USACE, 2008). Based on our analyses, the D50 of the riprap is 12 inches The selected gradation conforms to a standard Colorado Department of Transportation (CDOT, 2011) gradation and is provided in Table 7.5. The selected design thickness will be 2.0 feet.

TABLE 7.5
RIPRAP GRADATION

D_{50} Stone Size (inches)	Stone Size (inches)	% Smaller By Weight
12	21	70-100
	18	50-70
	12	35-50
	4	2-10

Based on our analyses the riprap bedding needs to be 12 inches thick. Wave run-up, riprap, and riprap bedding analyses are provided in 30 Percent Design Report, Appendix A.

SECTION 8 - HYDROLOGIC EVALUATION

8.1 100-Year Floodplain

City Lake Dam is an off-stream reservoir with a small drainage basin, approximately 0.45 square mile, with all normal inflows to the reservoir resulting from diversions from the Cucharas River. The dam and reservoir are located within a Federal Emergency Management Agency (FEMA) Zone A regulatory 100-year floodplain which is entirely within the Project property boundary. Reconstructing and raising the dam embankment will not change the current designated extent of the Zone A floodplain but will raise the water surface elevation by 3 feet. A Zone A designation includes areas with a 1 percent annual chance of flooding and a 26 percent chance of flooding over the life of a 30-year mortgage. Because detailed analyses are not performed for such areas; no depths or base flood elevations are shown within these zones. The FEMA Flood Insurance Rate Map (FIRM) is shown on Figure 8.1.

8.2 Dam Breach Flow and Dam Hazard Classification

Prior to 2015 the dam and reservoir had been classified as a Small, Significant-Hazard dam in accordance with requirements set forth in the State Engineer *Rules and Regulations for Dam Safety and Dam Construction* (SEO, 2007). A breach inundation mapping report dated January 2015 prepared by RJH concluded the dam met hazard classification criteria for a small High Hazard Dam. A high hazard dam is one for which human loss of life is expected to result from failure of the dam. Subsequent to the completion of the dam breach mapping report, the SEO reviewed the report, concurred with the modelling results and conclusions, and changed the hazard classification to High Hazard in a memorandum dated September 22, 2015. The HEC-HMS simulated breach hydrograph model resulted in a peak breach outflow of 5,657 cubic feet per second (cfs) with a total breach volume of 410 ac-ft. Due to the approximate 3,500-foot embankment length, the development of the embankment breach was selected at three different locations, on the north embankment, on the east embankment at the maximum section, and on the southeast embankment. Each modeled breach location resulted in differing breach flow paths and inundation zones. The raised embankment height and increased reservoir storage will increase the extent of the dam breach flow inundation zones. Revised breach mapping should be completed once the rehabilitation construction has been completed. The RJH Breach Inundation Mapping Report is contained in Appendix F.

8.3 Wave Run-Up Analysis

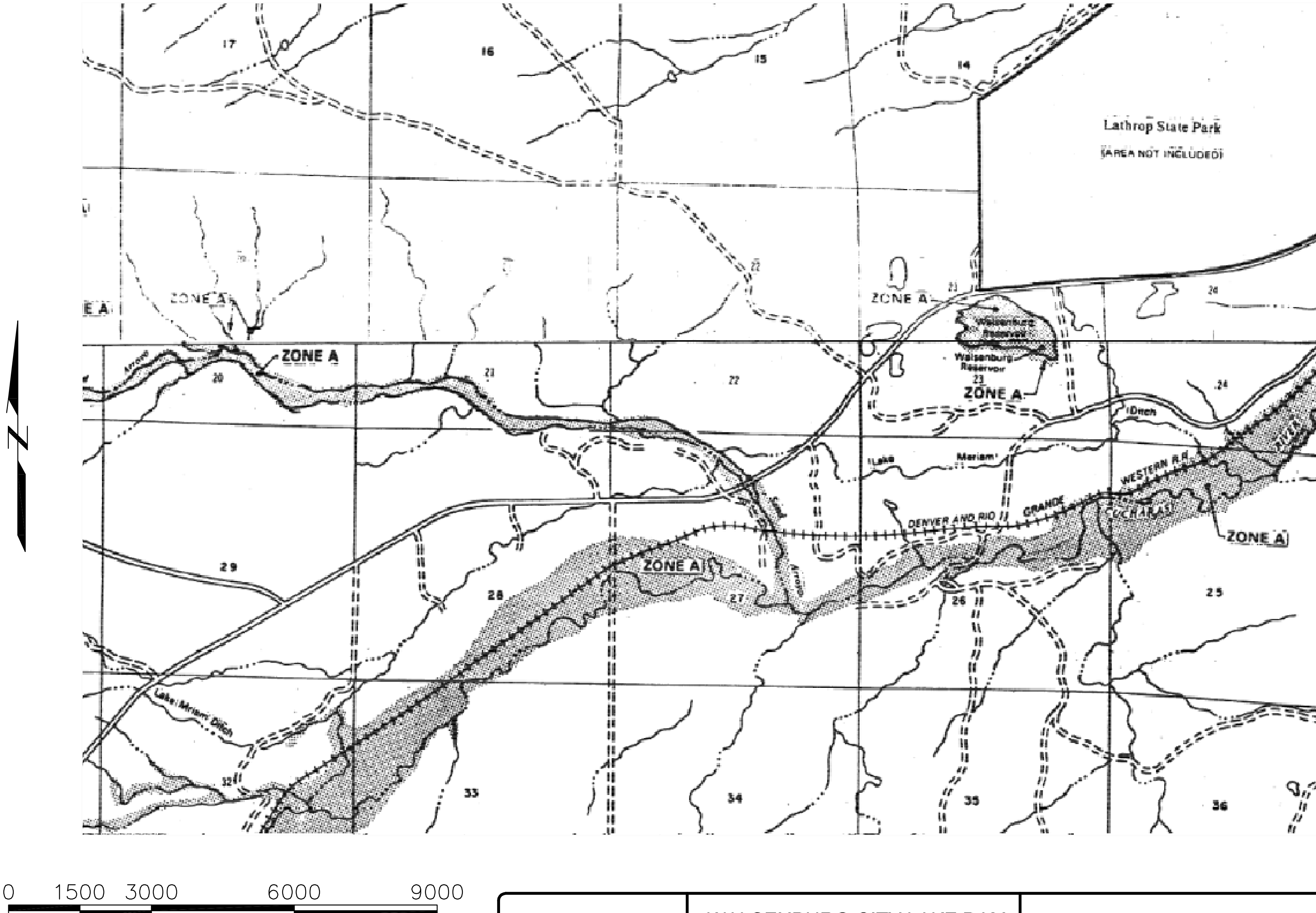
A wave run-up analysis was performed to evaluate the amount of minimum freeboard required by SEO guidelines. Guidance from the U.S. Army Corps of Engineers (USACE) and Reclamation was used for the analysis.

The total wave run-up is estimated to be about 2.9 feet, which is less than the minimum freeboard required by SEO of 5.0 feet. Based on the past poor performance of the existing embankment under sustained high wind loading conditions, a waiver for a reduced freeboard requirement will not be requested. Riprap sizing for the computed wave height is to be Type M with a D_{50} = 12 inches, minimum 24-inch thickness with 12 inches of free draining bedding gravel. These analyses are contained in the 30 Percent Design Report, Appendix A.

8.4 Inflow Design Flood and Reservoir Routing

The SEO requires the spillway to of sufficient size to pass the IDF while providing a minimum freeboard that is equal to the IDF maximum water surface level plus 1 foot of residual freeboard, and the total freeboard cannot be less than 5 feet. The invert of the emergency spillway would be set at El. 6523.0. This is 6 inches above the maximum normal pool and would allow the emergency spillway to be engaged less frequently, and would also provide a total freeboard of 5 feet between the spillway invert and dam crest at El. 6528.0 to meet SEO requirements.

RJH performed reservoir and spillway routing analyses using the USACE HEC-HMS computer program to size the auxiliary spillway based on the IDF presented in the SEO-approved *Hydrology Report* (RJH, 2015). A 50-foot-wide spillway would be required to pass the IDF event with 4 feet of routing head and would result in a peak outflow of 1,043 cfs. Reservoir routing calculations are provided in the 30 Percent Design Report, Appendix A.



RJH CONSULTANTS, INC.

WALSENBURG CITY LAKE DAM REHABILITATION AND ENLARGMENT

FEMA FLOOD INSURANCE RATE MAP

PROJECT NO. 14120

March 2017

Figure 8.1

SECTION 9 - HYDRAULIC STRUCTURES

9.1 General

Hydraulic facilities for the Project will include an outlet works, service spillway, auxiliary spillway, and pumpback system. The purposes of these facilities are:

- Outlet Works: a) Supply water to the WTP, b) provide reservoir releases during emergency conditions, and c) allow the reservoir to be drained for maintenance activities.
- Principal Spillway: a) Control maximum normal pool, b) convey flows from routine storm events to reduce the frequency that the emergency spillway is engaged, and c) discharge excess diverted flows from upstream reservoirs through the Coler Ditch to Horseshoe Reservoir.
- Auxiliary Spillway: Safely pass the IDF through the reservoir.
- Pump-back System: Convey flows from Coler Ditch into the reservoir.

9.2 Outlet Works and Service Spillway

The outlet works and service spillway will be combined into a single structure to reduce construction costs and long-term maintenance. The SEO requires that outlet works have sufficient hydraulic capacity to release the top 5 feet of the reservoir capacity in 5 days and be capable of releasing the entire reservoir in a reasonable period of time. Also, guard gates and trash racks must be installed on the upstream end of outlet works conduits. The SEO does not provide specific hydraulic design criteria for principal (i.e., non-emergency) spillways. The selected outlet works concept includes the following components:

- Excavated approach channel to lowest portion of reservoir.
- Reinforced concrete inlet structure with trash rack (no gate) at the upstream toe of the dam. The inlet structure will include a bulkhead that can be installed by divers if the inlet pipe needs to be dewatered.
- Twenty-four-inch-diameter, steel inlet pipe between the inlet structure and control tower. The inlet pipe will be encased in reinforced concrete.
- Reinforced concrete, dual-chamber control tower at the crest of the dam.
- Bulkhead at the downstream end of inlet pipe. The bulkhead will be mounted inside the control tower and will include a winch-system to allow the bulkhead to

be easily installed/removed. This will allow dewatering of the control tower without needing divers to install the bulkhead at the inlet structure.

- Five-foot wide service spillway weir in the control tower with invert at El. 6522.5. Service spillway flows will discharge to the outlet works pipe and then to Coler Ditch. The principal spillway will pass about the 25-year flood event with a hydraulic head of 6 inches before the auxiliary spillway is engaged.
- Two 18-inch-diameter, manually actuated slide gates will be used to control flow to the outlet works pipe and supply pipe to the WTP.
- Two 18-inch-diameter steel pipelines. The north pipeline (i.e., outlet works pipe) will discharge into Coler Ditch and be used for reservoir evacuation and principal spillway releases. The south pipeline (i.e., supply pipeline) will supply flows to the WTP. Both pipes will be encased in reinforced concrete through the dam embankment.
- Energy dissipation structure at the downstream end of the outlet works pipe that consists of a reinforced concrete, Reclamation-type, baffled outlet structure. The energy dissipation structure will be located on the west bank of Coler Ditch north of the WTP. A riprap apron will be installed along the bottom and banks of the Coler Ditch immediately downstream of the energy dissipation structure to protect against erosion and scour.

The computed total capacity of the outlet works (not considering flows through the supply pipeline) is about 26.6 cfs at maximum normal pool. The corresponding flow velocity is about 15.1 feet per second (fps), which is considered acceptable for steel pipes. The top 5 feet of reservoir storage could be drained in about 4.3 days and drained down to El. 6510.0 in approximately 10 days. Hydraulic calculations for the outlet works are provided in the 30 Percent Design Report, Appendix A. Structure details are provided in the 30 Percent Design Drawing set, Appendix B.

9.3 Emergency Spillway

The SEO requires that the spillway to be sized to pass the IDF while providing a minimum freeboard that is equal to the IDF maximum water surface level plus 1 foot of residual freeboard, and the total freeboard cannot be less than 5 feet. The invert of the emergency spillway would be set at El. 6523.0. This is 6 inches above the maximum normal pool and would allow the emergency spillway to be engaged less frequently, and would also provide a total freeboard of 5 feet between the spillway invert and dam crest at El. 6528.0 to meet SEO requirements. Refer to Drawings 19 and 20, 30 Percent Design Drawing set, Appendix B, for additional emergency spillway details.

RJH performed reservoir and spillway routing analyses using the USACE HEC-HMS computer program to size the auxiliary spillway based on the IDF presented in the SEO-approved *Hydrology Report* (RJH, 2015). A 50-foot-wide spillway would be required to pass the IDF event with 4 feet of routing head and would result in a peak outflow of 1,043 cfs. Reservoir routing calculations are provided in the 30 Percent Design Report, Appendix A.

The spillway concept includes the following components:

- 50-foot-wide reinforced concrete control structure at the centerline of the dam.
- 50-foot-wide, 270-foot-long, riprap-lined discharge channel with cutoff wall at the downstream end. The initial 20 feet of the riprap lining would be grouted in the vicinity of the embankment. The discharge channel will require earthen berms along each side of the channel to contain the flow. The downstream cutoff wall would reduce the potential for headcutting erosion to undermine the discharge channel.

RJH performed hydraulic analyses to evaluate peak flow depths and velocities in the discharge channel. These analyses resulted in a peak flow depth of 2.9 feet and a peak velocity of 7.5 fps for a flow of 1,043 cfs. This velocity is well below the maximum allowable velocity for riprap (i.e., about 12 fps) but is above the maximum allowable velocity for grass-lined channels (i.e., about 5 to 6 fps). Based on these results, a riprap-lined channel should be acceptable and is not expected to require significant maintenance. The flow containment berms will need to be 3 feet high. Discharge channel calculations are provided in n the 30 Percent Design Report, Appendix A.

SECTION 10 - IMPACTS

10.1 Impacts

Impacts resulting from reconstructing the existing dam and raising the embankment 3 vertical feet will be minimal. The reconstructed embankment will basically following the alignment of the existing embankment except where the embankment borders US Highway 160 where it will be shifted slightly southward to provide needed clearance from an existing overhead powerline and the highway.

Some existing wetlands adjacent to reservoir will be impacted. The USACE has agreed the existing wetlands can be relocated and re-established in conjunction with dam construction.

The reservoir will have to be drained during the construction and to accommodate the loss of water supply to the WTP, a temporary reservoir water supply pipeline will be constructed to supply water to the WTP. Dam construction is currently scheduled to occur during the winter of 2017-2018 when peak water demands are low and the reservoir is out of service.

SECTION 11 - OPINION OF PROBABLE PROJECT COST

This opinion of probable costs are based on the 30 percent design-level design concepts presented in this report. Quantities were estimated for the major construction items. The lump sum item prices are based on qualitative estimates of the work required and the corresponding cost. Estimated unit prices and costs for the primary work items were derived from the following sources:

- Published and non-published bid price data for similar work from similar projects.
- R.S. Means Heavy Construction Cost Data for 2016.
- Manufacturer's budgetary price quotes.
- Our previous experience and engineering judgment.

It is our opinion that this cost opinion represents a Class 3 level estimate as defined by the Association for the Advancement of Cost Estimating (AACE). This level is appropriate for a study where the design engineering is between 10 and 40 percent complete. The reliability of this level of estimate according to the AACE should be considered to be between about minus 10 to 20 percent and plus 20 to 30 percent, when all costs are compared to 2017 dollars.

The sum of the listed items for each category is defined for this study as the "Base Construction Subtotal" (BCS). The sum of the BCS, mobilization, bonds, and insurance is defined as the "Direct Construction Cost" (DCC). For this study an allowance of 2.0 percent of the BCS was included to account for the construction contractor's costs for mobilization, bonds, and insurance.

The Opinion of Probable Project Costs (OPPC) is the sum of the DCC, construction contingencies, and engineering and administration costs. For this Project, the OPPC includes an allowance of 37 percent for contingencies, engineering, and administration as follows:

- 15 percent of the DCC to account for construction contingencies. This allowance will decrease as project development progresses toward more detailed levels of design.
- 10 percent of the DCC for design engineering and permitting.
- 12 percent of the DCC for construction engineering.

A summary of quantities and our OPPC is presented in Table 11.1. Costs are presented in 2017 dollars.

This opinion of probable construction costs (OPCC) is based on professional opinion of the costs to construct the Project as described in this report. Actual costs would be affected by a number of factors beyond current control, such as supply and demand for the types of construction required at the time of bidding and in the Project vicinity, changes in material supplier costs, changes in labor rates, the competitiveness of contractors and suppliers, changes in applicable regulatory requirements, the ability of the City to obtain needed easements for embankment stockpiling and processing and changes in design standards and concepts. Therefore, conditions and factors that arise as Project development proceeds through construction may result in construction costs that differ from the estimates documented in this report.

Much of the cost for this Project is earthwork and the cost for earthwork is highly sensitive to fuel costs. If fuel costs change significantly in the next few years the cost of the Project could be directly impacted. RJH has not attempted to predict changes in future fuel prices to develop this OPPC.

TABLE 11.1
OPINION OF PROBABLE CONSTRUCTION COSTS

Item No.	Item	Unit	Quantity	Unit Price ($)	Total Cost ($)
General Site					
1	Stripping and Stockpiling Topsoil	CY	8,100	2.60	21,100
2	Clearing and Grubbing	Acre	12	4,200	50,400
3	Erosion Protection and Sediment Control	LS	1	12,000	12,000
4	Stream Diversion	LS	1	60,000	60,000
5	Reservoir Dewatering	LS	1	26,000	26,000
6	Demolition	LS	1	55,000	55,000
7	Abandon/Grout 8" Drain Pipe	LS	1	8,000	8,000
8	Supply Flow Bypass to WTP	LS	1	115,000	115,000
9	Access Road	LS	1	80,000	80,000
10	Boat Ramp	LS	1	15,000	15,000
11	Relocate Wetlands	LS	1	25,000	25,000
12	Traffic Control	Day	32	1,000.00	32,000
13	Property Easement/Purchase	LS	1	50,000	50,000
14	Site Reclamation	Acre	11	2,500	27,500
Embankment					
15	Dewatering for Embankment	LS	1	225,000	225,000
16	Excavation to Stockpile	CY	120,200	3.40	408,700
17	Remove Existing Slope Protection	CY	6,500	5.80	37,700

Item No.	Item	Unit	Quantity	Unit Price ($)	Total Cost ($)
18	Foundation Preparation	SY	7,500	10.80	81,000
19	Earthfill from Stockpile	CY	122,200	4.25	519,400
20	Import to Stockpile	CY	32,000	13.00	416,000
21	Filter Sand	CY	12,300	60.00	738,000
22	Filter Gravel	CY	1,200	97.00	116,400
23	PVC Slotted Drain Pipe	LF	3,250	47.50	154,400
24	PVC Solid Drain Pipe	LF	65	30.25	2,000
25	Riprap	CY	6,100	73.50	448,400
26	Riprap Bedding	CY	3,300	59.00	194,700
27	Class 6 Base Course on Crest and Access Roads	CY	2,200	40.00	88,000
Outlet Works					
28	Dewatering for Outlet Works	LS	1	35,000	35,000
29	Intake Structure	LS	1	5,500	5,500
30	24" Concrete Encased Conduit	LF	40	370.00	14,800
31	Gate Tower	LS	1	60,700	60,700
32	18" Gate and Operator	Each	2	7,700	15,400
33	18" and 8" Concrete Encased Conduit	LF	80	725.00	58,000
34	18" and 8" Buried Conduit (incl pumpback line)	LF	351	325.00	114,100
35	Outlet Structure	LS	1	8,500	8,500
36	Diversion Structure	LS	1	8,500	8,500
37	48" Buried Culvert	LS	1	5,900	5,900
38	48" Gate and Operator	Each	1	17,600	17,600
39	Riprap	CY	33	73.50	2,400
40	Riprap Bedding	CY	1	59.00	100
41	Regrade Ditch	LS	1	2,500	2,500
Auxiliary Spillway					
42	Earthfill from Stockpile	CY	510	4.25	2,200
43	Concrete Control Structure	LS	1	44,000	44,000
44	Concrete Cutoff Wall	LS	1	16,500	16,500
45	Grouted Riprap	CY	220	195.00	42,900
46	Riprap	CY	780	73.50	57,300
47	Riprap Bedding	CY	470	59.00	27,700
48	Geotextile	SY	330	3.00	1,000
Base Construction Subtotal (BCS)					**4,547,000**
Mob/Demob (5% of BCS)					227,350
Bonds/Insurance (2% of BCS)					90,940
Unscheduled Items (2.5% of BCS)					113,675
Direct Construction Subtotal (DCS)					**4,979,000**
Construction Contingencies (15% of DCS)					747,000
Design Engineering (10% of DCS)					498,000
Construction Engineering and Testing (12% of DCS)					597,000
Opinion of Probable Construction Cost (OPCC, 2017)					**6,821,000**

SECTION 12 - FINANCIAL PLAN

12.1 Loan Amount

The requested loan amount is $6,821,000. The loan amount is based on the 30 percent project design documents and Opinion of Probable Construction Cost prepared by RJH. The requested loan term is 30 years.

12.1.1 Financing Sources

The expected financing source is a WPLP Loan through the CWCB for the dam repair project. Funding for the initial dam assessment has been provided in part by a grant from DOLA.

12.1.2 Revenue and Expenditure Projections

A 2017 proposed budget has been provided for the Water Enterprise Fund. However, increases in revenue over the next several years can be estimated in the 3 to 6 percent range consistent with overall growth in water usage related to increases in residential and commercial usages. Expenses are expected to remain relatively flat with nominal increase in expenses related only to CPI in the 1.5 percent range.

12.1.3 Loan Repayment Sources

Loan repayment sources will be from the ongoing Water & Sewer enterprise operations and may include revenues from ongoing Water & Sewer enterprise revenues, transfers from current capital improvement funds, potential increases in water usage rates and/or an increase to current water service fees.

12.1.4 Financial Impacts

Total debt for the city will increase substantially. However, current water and sewer revenue are able to provide a debt coverage ratio at higher than 1.25 percent including the debt to be incurred related to the City Dam project. The City, by ordinance, does have the ability to increase water rates and/or services fees for the Dam loan. Preliminary estimates indicate an increase in water rates of $1.25 to $1.75 per 1,000 gallons of usage may be needed to fund loan repayment. However, overall water usage increases year over year will mitigate the need to impose large increases to water rates.

Increases to water rates require Council approval. The dam project will also increase lake capacity allowing for additional storage to mitigate impact by drought or other water shortages and prevent water restrictions that will impact the ability to collect revenue at the same usage rates. The additional storage capacity may allow the City other revenue generating opportunities by selling storage capacity to other entities.

12.1.5 TABOR

The City is pursuing a CWCB loan through its water enterprise. The water enterprise is an enterprise pursuant to Colorado Const. Art. X Sec. 20(2)(d). The enterprise receives less than 10 percent of its annual revenue from Colorado State and local grants combined. Under Colorado Const. Art. X Sec. 20, the City, acting by and through its water enterprise, may incur multi-year debt without a vote of the electorate. An opinion letter by the City attorney is provided in Appendix H addressing this issue.

12.1.6 Collateral

Collateral will be negotiated by Walsenburg's bond council at the time of loan approval. The dam, reservoir, and storage rights are available to be used as loan collateral.

12.1.7 Creditworthiness

The City's creditworthiness is based on its ability, by ordinance, to generate revenue by imposing usage fees to customers using the water system. Water usage rates can be increased by resolution upon approval by the City Council to meet the City's ongoing obligations. The most recent audited financial statements and the current water and sewer rate schedules are provided in Appendix H.

SECTION 13 - IMPLEMENTATION SCHEDULE

A proposed Project implementation schedule is presented in Table 13.1. A desired construction start date of November 1, 2017 is scheduled. Many preliminary steps are required to meet this desired start date and any significant delay could result in the construction start date being postponed for approximately 1 year. It would be operationally difficult for the City to meet peak water supply demands during the summer months if the reservoir is drained for construction. If the Cucharas River has sufficient base flow during the summer to meet the City's water demand, the two existing upstream reservoirs, Wahatoya and Daigre, have the capacity to supply water to the treatment plant during the summer months via the bypass pipeline. If the Cucharas River flow is short of meeting the City's needed water demand, both upstream reservoirs could be drained leaving only the 8 inch pump back pipeline from Martin Lake to supply water to the treatment plant. The pump back pipeline does not have sufficient capacity to meet the City's water demand during the summer months and watering restrictions would likely have to be implemented.

TABLE 13.1
PROJECT IMPLEMENTATION SCHEDULE

Item	Schedule Date
Loan Application and Feasibility Study to CWCB	April 2017
Final Project Engineering Design Started	April 2017
Permitting Started	April 2017
Feasibility Study Reviewed and Approved by CWCB	May 2017
Funding Approved by CWCB Board	May 2017
Project Design Completed	July 2017
Designs, Plans, and Specifications Submitted to SEO	July 2017
SEO Approves Project	August 2017
Bidding and Procurement	September 2017
All Permitting Obtained	October 2017
Project Construction Started	November 2017
Project Construction Completed	May 2018
Project Closeout and Construction Completion Documents To the SEO	July 2018

13.1 Permitting and Institutional Feasibility

Permitting from and coordination with a number of governmental agencies will be required to construct the Project. Following is a listing of the agencies and the anticipated permits that will be required.

13.1.1 State Engineers Office

The dam and reservoir must be designed and constructed in accordance with the SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO, 2007). Review and approval of Project designs, plans, specifications, and construction by the SEO will be required.

13.1.2 United States Army Corps of Engineers

The Pueblo Colorado Permitting Office of the USACE, Albuquerque District, has reviewed the planned dam site modifications and performed a site inspection of the Project. The USACE has determined the Project will require a Section 404 Permit of the Federal Clean Water Act. They are currently evaluating whether the Project can be considered maintenance under the nationwide category of Section 404 of the Clean Water Act or whether an individual Section 404 Permit may be required. The USACE expressed their opinion that since the process of obtaining the required Section 404 permit had been started with adequate lead time in advance of the planned construction start date of November 1, 2017, there should be sufficient time to obtain either permit. Their evaluation of the needed permit is ongoing at the time this report was prepared.

13.1.3 Colorado Division of Mine Safety

The planned location for obtaining borrow embankment material for reconstructing the dam from Lathrop State Park, owned by Colorado Parks and Wildlife (CPW), will require a permit from the Colorado Division of Reclamation and Mine Safety. Following are the general requirements for obtaining the permit.

- The City of Walsenburg or CPW will have to apply for a Government 111 Reclamation Permit. The cost of the application is $898 and an annual cost of $504/year to maintain the permit until the reclamation of the borrow area is completed including vegetation reestablishment.
- The reclamation plan must also comply with applicable sections of the Minerals Rules and Regulations.
- The permit is generally approved within 15 days of receipt of the application assuming the application package is complete.
- A reclamation bond is required until the reclamation effort is complete and accepted by the Minerals Division.

- No water may be diverted or stored in the borrow excavations prior to acceptance of the final reclamation effort by the Minerals Division without an approved SEO augmentation plan.
- The reclamation plan should be simple and should include reapplying stockpiled top soil to the cut slopes, to be 3:1 or flatter, and establishing a native grass mix recommended for the area by the Natural Resources Conservation Service (NRCS).

13.1.4 Colorado Department of Transportation

If the construction contractor elects to haul embankment material from the borrow site north of US Highway 160 directly to the Site by way of a constructed haul along and crossing US Highway 160 where there is currently no access control, a special use permit will be required. Discussions with Valerie Sword, Permit Manager, Colorado Department of Transportation Region 2 Office (CDOT) located in Pueblo indicated a special use permit could be obtained for a temporary borrow area haul road crossing on US Highway 160. The temporary crossing would be located directly north of the Site where there is no current highway access from the north side of the highway. The special use permit application is four pages in length and must comply with CDOT Utility/Relocation/Special Use Permit Standard Provisions, effective date March 1, 2006. The selected construction contractor will have to complete the application and obtain approval from CDOT prior to constructing and using the temporary crossing. If properly addressed in the application, it will be possible for off-road haul trucks to use the temporary highway crossing. As an alternative to obtaining the haul road special use permit, the contractor could elect to use existing county and state roads for hauling the borrow material subject to legal load limits and over-the-road truck requirements.

13.1.4 Floodplain

A FEMA Letter of Map Amendment (LOMA) will be required to raise the reservoir water surface elevation by 3 three vertical feet. The normal high water surface of the reservoir is designated as FEMA Flood Insurance Rate Map (FIRM) Zone A. A Zone A designation includes areas with a 1 percent annual chance of flooding and a 26 percent chance of flooding over the life of a 30-year mortgage. Because detailed analyses are not performed for such areas; no depths or base flood elevations are shown within these zones.

13.1.5 Huerfano County

A county Land Use 1041 permit will be required to construct the Project. The Huerfano County Planning Department will review the Project application and the County Commissioners will approve the permits based on recommendation of the Planning Department staff. The Huerfano County Planning Department staff will route the Special Use Permit application to all local and state agencies that might have comments on the Project.

SECTION 14 - LIMITATIONS

The information presented in this report is suitable for planning and funding purposes only. The information in this report is based on data obtained from review of existing documents and data, previously completed studies and analyses, and recently completed studies and analyses for the Project. Additional data will be needed for final design to confirm the concepts and details presented in this report. Also, the nature and extent of variations between specific subsurface data may not become evident until future phases of exploration and construction. Timely and comprehensive observation and evaluation of actual subsurface conditions, supported by appropriate field and laboratory testing, will be critical during final design and construction phases. Variations in the subsurface profile described herein should be anticipated.

RJH has endeavored to conduct our professional services for this Project in a manner consistent with a level of care and skill ordinarily exercised by members of the engineering profession currently practicing in Colorado under similar conditions as this project. RJH makes no other warranty, expressed or implied.

Opinions of Probable Project Costs presented in this report are based on our professional opinion of the cost to construct the Project as described in this report. The estimated costs are based on the sources of information described herein, and our knowledge of current construction cost conditions in the locality of the Project. Actual Project construction costs are affected by a number of factors beyond our control. Therefore, conditions and factors that arise as Project development proceeds through design and construction may result in construction costs that differ from the estimates documented in this report.

This report has been prepared for use by the City of Walsenburg and for exclusive application to the City Lake Dam Rehabilitation and Enlargement.

SECTION 15 - REFERENCES

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