RJH CONSULTANTS, INC.

GEOTECHNICAL AND
WATER RESOURCES ENGINEERING

60-PERCENT DESIGN REPORT

SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT

BOULDER COUNTY, COLORADO

Submitted to
City of Boulder
1777 Broadway
Boulder, Colorado 80302

Submitted by
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February 2024
Project 16134

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Abbreviation	Term
ac-ft	Acre-Feet
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials International
BCS	Base Construction Subtotal
BVCP	Boulder Valley Comprehensive Plan
CDOT	Colorado Department of Transportation
CDPHE	Colorado Department of Public Health and Environment
cfs	Cubic Feet per Second
City	City of Boulder
CLOMR	Conditional Letter of Map Revision
cm/s	Centimeter per Second
Collier	Collier Geophysics, LLC
CORVUS	CORVUS Environmental Consulting, LLC
CRS 9-1.5	Colorado Revised Statutes, Title 9, Article 1.5
CU	University of Colorado
CWA	Clean Water Act
CY	Cubic Yard
DCD2	Dry Creek Ditch No. 2
DCS	Direct Construction Subtotal
DHI	DHI Water and Environment, Inc.
El.	Elevation
ERO	ERO Resources Corporation
FEMA	Federal Emergency Management Agency
Flatirons	Flatirons, Inc.
fps	Feet per Second
Ft^3/d	Cubic Feet per Day
GPM	Gallons per Minute
GPS	Global Positioning System
HDR	HDR, Inc.
HFB	Horizontal Flow Barrier
H:V	Horizontal to Vertical
IDF	Inflow Design Flood
Lithos	Lithos Engineering
LOMR	Letter of Map Revision
Mg/kg	Milligrams per Kilogram
MHFD	Mile High Flood District

Abbreviation	Term
mph	Miles per Hour
MTMB	Microtunnel Boring Machine
Muller	Muller Engineering Company
NWP	Nationwide Permit
OPPC	Opinion of Probable Project Cost
OSBT	Open Space Board of Trustees
OSMP	Open Space and Mountain Parks
OS-O	Open Space - Other
PFM	Potential Failure Modes
PGA	Peak Ground Acceleration
PK-U/O	Park, Urban, and Other
PMJM	Preble's Meadow Jumping Mouse
PMF	Probable Maximum Flood
Project	South Boulder Creek Regional Detention
psi	Pounds per Square Inch
PUB	Public
PVC	Polyvinyl Chloride
QL	Quality Level
RCBC	Reinforced Concrete Box Culverts
RCP	Reinforced Concrete Pipes
Reclamation	U.S. Bureau of Reclamation
RESPEC	RESPEC Company, LLC
Report	Preliminary Design Report
RJH	RJH Consultants, Inc.
RMS	Root Mean Squared
ROW	Right-of-Way
Rules and Regulations	Rules and Regulations for Dam Safety and Dam Construction
SAM	Surveying and Mapping, LLC
SBC	South Boulder Creek
SEO	Colorado Office of the State Engineer
SHPO	State Historical Presentation Office
SPT	Standard Penetration Test
SUE	Subsurface Utility Engineering
T&E	Threatened and Endangered
TR	Technical Report
tsf	Tons per Square Foot
ULTO	Ute Ladies'-Tresses Orchid
US36	U.S. Highway 36

Abbreviation	Term
USACE	U.S. Army Corps of Engineers
USFWS	U.S. Fish and Wildlife Service
WSE	Water Surface Elevation
2D	Two-Dimensional

SECTION 1 - INTRODUCTION

1.1 Purpose and Objectives

RJH Consultants, Inc. (RJH) was retained by the City of Boulder (City) and Mile High Flood District (MHFD) to provide engineering services for the South Boulder Creek (SBC) Regional Detention Project (Project). The purpose of the Project is to improve floodplain resiliency in portions of the Frasier Meadows, Keewaydin Meadows, and East Boulder neighborhoods from floods originating along SBC up to a 100-year flood event. This 60-Percent Design Report (Report) presents the results and conclusions of the 60-percent design.

The 60-percent design is documented in this Report and the 60-percent design drawings (Appendix K, bound separately). This Report and the drawings are complimentary to each other and combined represent the 60-percent design of the Project.

The 60-percent design presented in this Report is based on hydrologic and hydraulic modeling, our current understanding of subsurface and groundwater conditions based on site investigations, engineering analyses to support development of Project components, engineering judgment, and our previous experience with similar projects. The information in this Report will be refined and modified during the final design phase.

1.2 Background

Over the past 80 years, SBC has significantly flooded six times. SBC has limited channel capacity upstream of U.S. Highway 36 (US36), and US36 overtops during large storm events. Overtopping stormwater flows north and west to a low point on the University of Colorado's (CU) Boulder South campus parcel near US36 and Table Mesa Drive. In sufficiently large flood events, stormwater overtops US36 and floods extensively through a portion of the City known as the West Valley that includes portions of the Frasier Meadows, Keewaydin Meadows, and East Boulder neighborhoods. SBC overtopped US36 in 1969 and 2013 and flooded the West Valley in 1938, 1950, 1969, and 2013.

The City and MHFD retained RJH to provide engineering services for design of a regional stormwater detention facility at US36. RJH, the City, and MHFD evaluated various concepts that could reasonably be implemented in the vicinity of the US36 regional detention facility site to reduce the risk for overtopping of US36 during a major flood event while also addressing other Project requirements. The methodology, results, and conclusions of the concept design work is presented in the *South Boulder Creek Regional Detention Concept*

Design Report (RJH, 2020). The City selected the Variant 1, Option 1 (100-Year) concept presented in the Concept Design Report (RJH, 2020) as the preferred alternative to advance to design.

1.3 Scope of Services

RJH performed the following services for the design phase of the Project to date:

1. Managed and coordinated the work performed by RJH and our subconsultants.
2. Supported and participated in meetings with key regulatory agencies and stakeholders, including the Colorado Department of Transportation (CDOT), Colorado Office of the State Engineer (SEO), City of Boulder Open Space and Mountain Parks (OSMP), U.S. Army Corps of Engineers (USACE), U.S. Fish and Wildlife Service (USFWS), and others.
3. Conducted geotechnical investigations that included drilling, excavating test pits, performing a geophysical survey, performing field and laboratory testing, and preparing a Geotechnical Data Reports.
4. Collected and evaluated groundwater data.
5. Developed a baseline groundwater model (i.e., existing conditions) and prepared a Baseline Groundwater Modeling Report.
6. Performed topographic surveying to update the Project base mapping and to support hydraulic modeling. Prepared a Survey Data report discussing surveying methods and accuracies.
7. Performed a subsurface utility engineering (SUE) survey and incorporated utility information into Project base mapping.
8. Identified and documented relevant operational, maintenance, technical, and regulatory criteria.
9. Performed hydrologic analyses to develop the Inflow Design Flood (IDF). Prepared a Hydrology Report and submitted to the SEO.
10. Developed a Corrected Effective Model in the MIKE FLOOD program.
11. Performed hydraulic modeling in the MIKE FLOOD program to support the site layout and sizing of Project facilities.
12. Performed geotechnical analyses to support design of the embankment.
13. Performed geotechnical, geostructural, and hydraulic analyses to support design of the spillway and appurtenant structures.

14. Performed hydraulic and geotechnical analyses to support design of the outlet works.
15. Performed geotechnical analyses to support design of the barrier wall and groundwater conveyance system.
16. Performed hydrologic and hydraulic analyses to support design of site drainage facilities.
17. Developed landscape architecture concepts for landscaping and aesthetic treatments along the spillway corridor.
18. Performed ecological evaluations and developed concepts for ecological mitigation and restoration.
19. Developed design drawings to 60-percent complete level.
20. Prepared an ASTM International (ASTM) E2516-11 Class 2 (ASTM, 2011) (i.e., budgetary level) opinion of probable project cost (OPPC) for the design.
21. Developed an SRH-2D hydraulic model of the US36 bridge.
22. Performed field mapping of existing Ute ladies' tresses orchid (ULTO) specimens.
23. Prepared and submitted a Request for Jurisdictional Determination to USACE.
24. Subcontracted a heavy-civil construction contractor to perform a constructability review.
25. Prepared this Report.

1.4 Project Personnel

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

Project Manager:	Robert Huzjak, P.E.
Project Engineer:	Eric Hahn, P.E.
Lead Geotechnical Engineer:	Adam Prochaska, Ph.D., P.E., P.G.(1)
Staff Engineers:	Jacquelyn Hagbery, P.G.(1), P.E. Adam Merook, P.E. Bryson Tillema, E.I. Andrew Atkins, E.I.
Technical Review:	Douglas Neighbors, P.E.

Note 1: Licensed in states other than Colorado.

The work described in this Report was completed by RJH as the prime consultant with assistance from the following subconsultants (collectively referred to as the RJH Team):

Hydraulic Modeling:	DHI Water and Environment, Inc. (DHI) Muller Engineering Company (Muller)
Environmental Permitting:	CORVUS Environmental Consulting, LLC (CORVUS) ERO Resources Corporation (ERO)
Ecological Restoration:	Westervelt Ecological Services (Westervelt) Headwaters Corporation (Headwaters)
Surveying:	Merrick and Company (Merrick)
Subsurface Utility Engineering:	Surveying and Mapping, LLC (SAM)
Tunnel Engineering:	Lithos Engineering (Lithos)
Cultural Resources:	PaleoWest
Landscape Architecture:	Architerra Group (Architerra)

The work described in this Report was overseen and coordinated by the City and MHFD. The City and MHFD team include the following personnel:

City Project Manager:	Brandon Coleman, P.E.
MHFD Advisor:	Kurt Bauer, P.E.
City Director of Public Works:	Joe Taddeucci, P.E.

We would like to recognize and thank OSMP staff for their support throughout the development of the work contained in this Report.

SECTION 2 - PREVIOUS STUDIES AND REPORTS

2.1 General

Numerous planning and engineering studies of SBC and surrounding areas have been performed over the last several decades for the City, MHFD, and others. The RJH Team collected and reviewed previous studies, including major drainageway master plans, flood mapping studies, and hydrology reports. Previous studies by others are documented in the *South Boulder Creek Regional Detention Concept Design Report* (RJH, 2020) and include the following:

- Comprehensive master plans developed in 2001 (Taggart, 2001) and 2015 (CH2M, 2015) to identify and evaluate flood mitigation concepts along SBC.
- Boulder Valley Comprehensive Plan (BVCP) update (City of Boulder and Boulder County, 2017) in July 2017, which changed the land use designations for approximately 80 acres of the CU Boulder South campus to facilitate construction of the regional stormwater detention facility at US36. The BVCP CU Boulder South Guiding Principles also provided direction to consider mitigating flood risk to the highest practicable standard while balancing environmental, social, and financial impacts.
- A Conditional Letter of Map Revision (CLOMR) prepared by Plenary Roads and Michael Baker Jr., Inc. to document changes in the SBC floodplain resulting from the US36 widening project.

A summary of previous studies relevant to design is provided below.

2.2 Flood Mapping Study

HDR, Inc. (HDR) completed a comprehensive flood mapping study that serves as the basis for the Federal Emergency Management Agency (FEMA) regulatory floodplain. The HDR study consisted of three reports:

- *South Boulder Creek Climatology/Hydrology Report* (HDR, 2007).
- *South Boulder Creek Hydraulic Modeling Report* (HDR, 2008).
- *South Boulder Creek Risk Assessment Report* (HDR, 2009).

The *South Boulder Creek Climatology/Hydrology Report* evaluated basin-specific design storms for both the general storm (i.e., long-duration) and thunderstorm (i.e., high-intensity, short-duration) precipitation events for return frequencies ranging from 2 to 500 years.

Various combinations of spatial orientations were evaluated to identify critical precipitation events. In general, storms containing the created main stem peak flows were determined to occur in the lower watershed (i.e., downstream of Gross Reservoir).

Rainfall-runoff analyses were performed using a MIKE 11 model, which is part of DHI's MIKE FLOOD proprietary software program. MIKE 11 is a dynamic, one-dimensional hydrologic model. The watershed was divided into 27 sub-basins, and hydrologic characteristics were developed for each sub-basin.

Hydraulic modeling was performed using a combination of MIKE 11 and MIKE 21 models. MIKE 11 was used to model the channel and hydraulic structures along the mainstem of SBC and major tributaries. MIKE 21 was used to model overbank and floodplain areas. The following blockages were used in the FEMA regulatory model at relevant structures:

- US36 bridge at SBC: 10-foot-wide obstructions at both bridge piers (approximately 20 percent blocked).
- Dry Creek Ditch No. 2 (DCD2) culvert at US36: 35 percent blocked.
- Viele Channel culvert at US36: 0 percent blocked.

Topographic information was developed from LiDAR data obtained by the City in 2003. A 4-meter grid was used to develop the FEMA regulatory model.

2.3 Concept Design Report

The RJH Team performed data collection, hydrologic and hydraulic modeling, and concept-level engineering analyses to develop concept-level alternatives to facilitate the City's selection of a preferred alternative to advance into design. The concept-level alternatives were identified based on Project objectives, constraints, site conditions; public and stakeholder input; and City staff input. The alternatives were developed for the 100-year flood event, 500-year flood event, and a flood event between the 100-year and 500-year floods. The alternatives are presented in the Concept Design Report (RJH, 2020).

Concept selection criteria were developed by the RJH Team, the City, and MHFD and generally included Project viability, technical, operational, environmental, and economic issues. The City selected the Variant 1, Option 1 concept as the preferred alternative to advance to design. This concept was designed for the 100-year flood event.

SECTION 3 - EXISTING CONDITIONS

3.1 General

The Project will be located in southeast Boulder, Colorado, and is generally located south of US36, west of SBC, and east of several residential communities. RJH has performed multiple site visits since 2017 to observe site conditions and perform data collection. The Project site is comprised primarily of undeveloped land and irrigated pasture. Existing land uses, site conditions, and constraints that impacted design of the Project are summarized in the following sections. A site vicinity map is presented on Figure 3.1, and a site plan is presented on Figure 3.2.

3.2 University of Colorado Boulder South Campus

The CU Boulder South campus is a 308-acre property located south of US36, east of several residential communities, and west of OSMP property. The CU Boulder South campus currently includes a tennis complex, a maintenance building with an asphalt parking lot, and a series of pedestrian trails. The pedestrian trails experience significant use from the public throughout the year. The tennis complex is used seasonally by the CU athletic department. Overhead electrical lines and multiple buried utilities exist on CU Boulder South campus, primarily near the maintenance building, tennis complex, and the northwestern portion of the property.

South Loop Drive is the primary means of vehicle access to the CU Boulder South campus. South Loop Drive is a 24-foot-wide, paved road that extends from Table Mesa Drive to the existing CU maintenance building and gravel parking lot. South Loop Drive is owned and maintained by CU.

Gravel mining operations were performed on the CU Boulder South campus property before it was acquired by CU in 1996. The gravel mining created a large excavation about 10 to 15 feet below the original ground surface. Gravel mining operations also created a series of below-grade ponds that fill with groundwater. Water levels in these ponds fluctuate with groundwater levels.

Two surface water ditches are located within the previously mined areas. The ditches collect groundwater and surface water and convey flow northward until discharging to ponds on the CU Boulder South campus. The ponds will ultimately overflow into Viele Channel.

An earthen levee extends along the south and east boundaries of the CU Boulder South campus. The levee is approximately 7,500 feet long and varies in height, with a maximum height of about 14 feet. The levee was constructed in 1980 and consists primarily of clayey sand materials. The levee was raised in 1998 and certified by FEMA in 2000. The levee was raised again in 2009 based on updated hydraulic modeling and subsequently recertified by FEMA (Leonard Rice, 2009). A pedestrian trail extends along the crest of the levee. The dry-side slope is covered with grasses and other vegetation. The wet-side slope is covered by riprap slope protection. DCD2 extends along the wet-side (i.e., east) toe of the levee. A drainage channel extends along the dry-side (i.e., west) of the levee. This channel was constructed to collect surface water runoff from behind the levee and convey the runoff to an outfall at Viele Channel.

An existing earthen berm (i.e., west berm) is located along the west side of the CU Boulder property adjacent to E. Moorhead Circle. The berm was constructed concurrent with previous mining operations on the site. The berm ranges from 10 to 20 feet high and contains moderately dense tree growth on both sides of the berm. A pedestrian trail extends along the crest of the berm.

CU Boulder South campus contains wetlands near drainage ditches, irrigation ditches and laterals, and in unreclaimed mining ponds. ULTO habitat and populations occur near drainage ditches predominantly on the dry-side of the levee embankment near the east portion of the CU Boulder South campus and along irrigation ditches.

In September 2021, the City annexed CU Boulder South campus as part of negotiations to provide community benefits, including flood protection. As part of the annexation agreement, the parties agreed to the following land uses for the CU Boulder South campus:

- Open Space - Other (OS-O): This area generally corresponds with the regulatory 500-year floodplain on the east portion of the CU Boulder South campus (approximately 119 acres). This land will remain undeveloped and be used for floodplain functionality, riparian connectivity to the SBC riparian corridor, and open space. A large-scale ecological restoration of this area will be performed as part of the Project. The ecological restoration will include environmental mitigation needed to permit and construct the Project.
- Public (PUB): This area is located on the west portion of the CU Boulder South campus (approximately 129 acres). This land will be developed in the future as part of development of the CU Boulder South campus.
- Park, Urban, and Other (PK-U/O): This area is located on the north portion of the CU Boulder South campus (approximately 60 acres). This land will be used for Project flood mitigation facilities. CU may install facilities for active and passive recreational uses in the future as long as they do not impact the functionality of the flood mitigation facilities.

A plan of CU Boulder South campus land use designations is presented on Figure 3.3.

3.3 Open Space and Mountain Parks

OSMP property is located on both sides of US36, west of SBC and east of the CU Boulder South campus. The OSMP property is located within the South Boulder Creek State Natural Area and contains extensive wetlands and federally listed Threatened and Endangered (T&E) species habitat for the Preble's meadow jumping mouse (PMJM) and ULTO. The South Boulder Creek State Natural Area was designated by the state of Colorado in 2000 in recognition of the high-quality habitat and numerous rare and uncommon species and plant communities. To manage and support OSMP's agricultural program, seasonal cattle grazing is also used, and portions of the area are irrigated for hay production. Numerous irrigation ditches and small drainage channels extend through the OSMP property, including DCD2.

A gravel pedestrian trail extends north-south through OSMP property on both sides of US36 and experiences significant use from the public.

3.4 Colorado Department of Transportation

The CDOT Right-of-Way (ROW) extends parallel to and on both sides of US36. A small drainage ditch is located in the south ROW along the south shoulder of the road. The drainage ditch collects surface water runoff from east-bound lanes on US36. A concrete multi-use trail is also located in the south ROW. The multi-use trail experiences significant use from the public. Additionally, multiple buried utilities are located throughout the ROW.

A series of culverts extend beneath US36. These include dual 4-foot by 10-foot reinforced concrete box culverts (RCBC) that function as a wildlife crossing, a 4-foot by 6-foot RCBC to convey DCD2 flows, three 60-inch-diameter reinforced concrete pipes (RCP) to convey Viele Channel flows, and multiple smaller RCPs to convey local drainage and irrigation flows.

SBC flows under US36 through a multi-span bridge. The bridge was widened in 2014 as part of the US36 widening project. The bridge has three spans that total approximately 115 feet, with a row of concrete bridge piers on each creek bank about 47 feet apart. The concrete multi-use trail extends below the bridge to the west of SBC.

3.5 South Boulder Creek

SBC is a major drainageway that flows from its headwaters in the mountains through Eldorado Canyon and subsequently southeast of the City before discharging to Boulder Creek. The SBC watershed encompasses approximately 136 square miles. Flow in SBC is from a combination of groundwater, precipitation runoff, releases from Gross Reservoir, and snowmelt. Gross Reservoir is located on SBC upstream of Eldorado Canyon and is a water supply reservoir owned and operated by Denver Water. No reservoir volume is allocated for flood control in Gross Reservoir, but the reservoir provides significant temporary flood storage above the spillway crest. Approximately 90 square miles of the SBC watershed is located upstream of Gross Reservoir. A plan of the SBC watershed is presented on Figure 3.4.

SBC generally flows northward east of the Project facilities and consists of a relatively straight, alluvial stream channel. The right overbank is significantly higher than the channel and is not expected to be overtopped during extreme flood events. The left overbank is lower and is overtopped during both routine and extreme flood events.

During floods that overtop the left bank of SBC, the US36 embankment directs flood waters north and west to a low point located at the northwest corner of the CU Boulder South campus near US36 and Table Mesa Drive. Flood waters pond in this area then overtop US36 and extensively flood a portion of the City known as the West Valley. The West Valley generally follows the alignment of Foothills Parkway and consists of a mixture of residential and commercial structures. Flooding of the West Valley occurred in 1938, 1950, 1969, and 2013. The 2013 flood event on SBC was estimated to be between about a 75- to 100-year event (Wright Water Engineers, 2014).

3.6 Viele Channel

Viele Channel generally flows across the Project site from west to east. Viele Channel extends through the northwest portion of the CU Boulder South campus and through the west edge of OSMP property, north of US36. In this reach, Viele Channel consists of a trapezoidal channel with thick vegetation.

Viele Channel is a tributary to SBC and has a basin area of approximately 1 square mile upstream of the CU Boulder South campus. A majority of the Viele Channel watershed consists of residential land use. This channel collects groundwater and surface water runoff. Flow in Viele Channel is conveyed beneath the US36 east-bound on-ramp through three 72-inch diameter culverts and subsequently beneath US36 through three 60-inch diameter culverts.

3.7 Irrigation Ditches and Laterals

DCD2 is owned and maintained by the DCD2 Company. Flows in the ditch are diverted from SBC approximately 1.8 miles upstream of the Project site. DCD2 consists of an earthen ditch from the point of diversion through OSMP property to the Project site. Multiple turnout structures are located along this segment of the ditch that facilitates flood irrigation of OSMP property south of US36. DCD2 flows through a 6-foot by 4-foot reinforced concrete box culvert below US36.

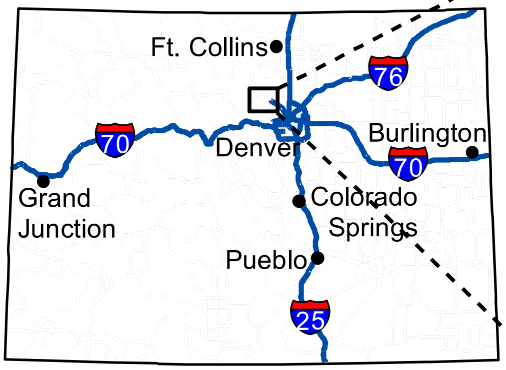
Numerous other irrigation ditches and smaller lateral irrigation channels (laterals) exist to distribute water throughout irrigated areas on OSMP. Based on information from OSMP and field observations by RJH, water is supplied to the OSMP fields using flood irrigation by placing check dams in irrigation ditches; the farmers control the location and timing of the flood irrigation and generally do not keep written records of this process.

3.8 Subsurface Conditions

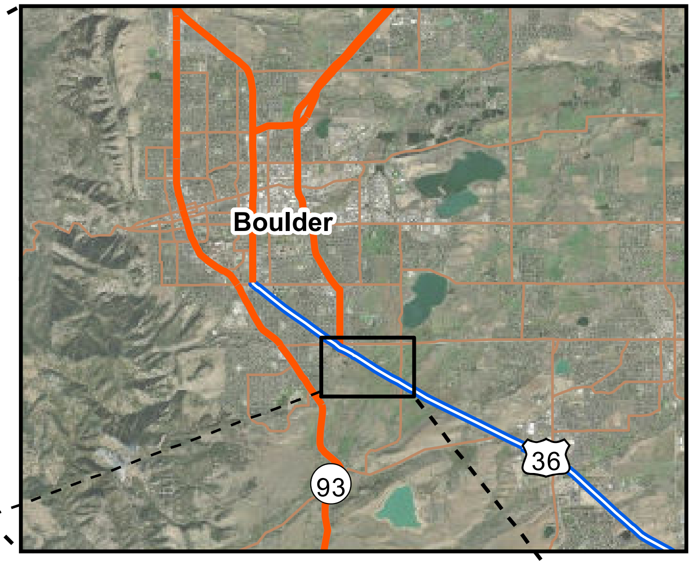
Based on site investigations performed by RJH (RJH, 2019; RJH, 2022b), our interpretation of the general subsurface profile at the Project site consists of fill or alluvium overlying bedrock of the Pierre Shale formation. In general, fill overlies bedrock throughout mined portions of the CU Boulder South campus, and alluvium overlies bedrock throughout the remainder of the Project site. Additional information regarding the geotechnical site conditions is presented in Section 8 and in the RJH Geotechnical Data Reports (RJH, 2019; RJH 2022b, RJH 2023).

The SBC alluvial valley begins upstream of the Project site as a relatively narrow mountain stream flowing from Eldorado Canyon. Near the Project site, the alluvial valley generally widens until it converges with the Boulder Creek alluvial valley downstream of Baseline Road. The SBC alluvial valley aquifer is an unconfined aquifer that extends throughout surficial soils (alluvium and fill) and is perched on the underlying low permeability bedrock. The alluvium generally decreases in thickness from upstream to downstream. The top of bedrock beneath the surficial soil appears to form a consistent broad surface that, in some locations, decreases in elevation to the west (away from SBC).

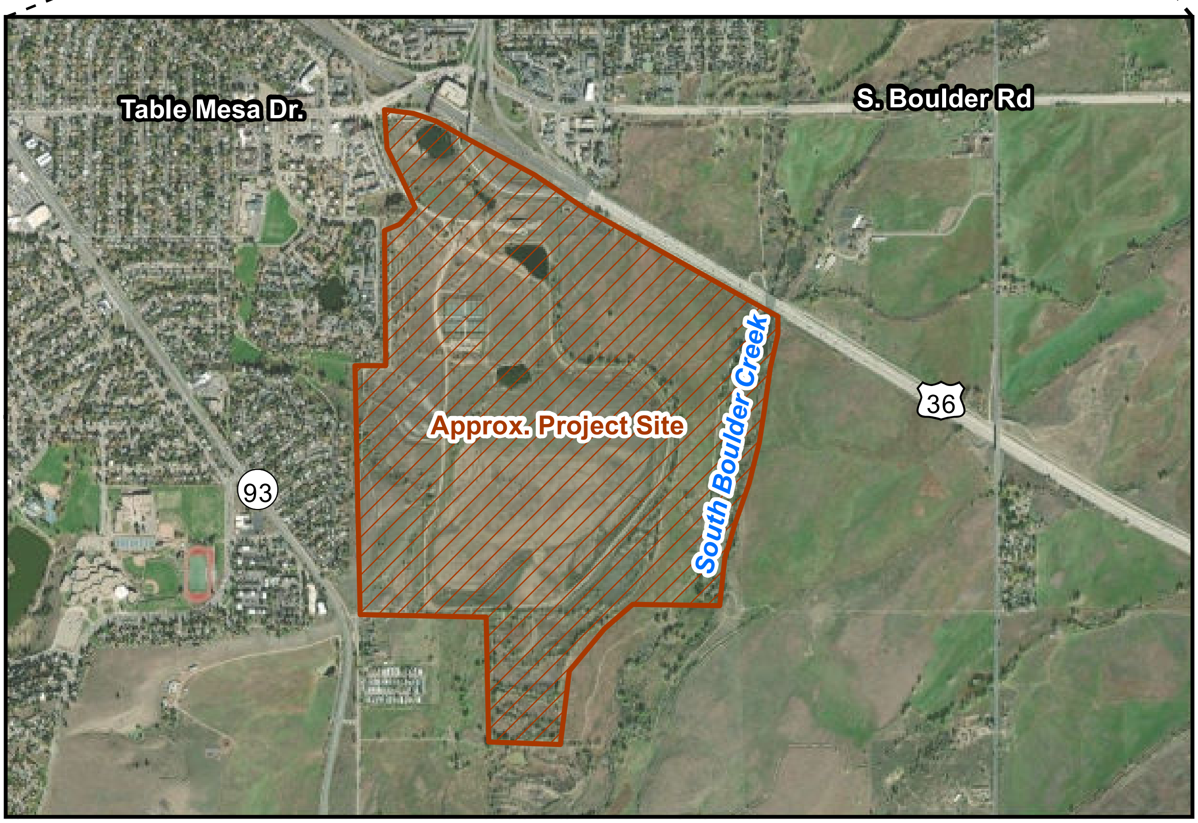
Based on site investigations and groundwater modeling performed by RJH, groundwater levels decline toward the north through the aquifer, which generally follows the slope of topography and the flow of SBC. Seasonal groundwater fluctuations are influenced by natural conditions through the hydrogeologic cycle (e.g., recharge, evapotranspiration, etc.) and irrigation applied to OSMP fields. Lowest groundwater levels are typically during the winter months of November through February. Highest groundwater levels typically occur from May through July.



PROJECT VICINITY MAP
(NOT TO SCALE)



PROJECT LOCATION MAP
(NOT TO SCALE)



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SITE LOCATION MAP
(NOT TO SCALE)

RJH CONSULTANTS, INC.	SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT	SITE VICINITY MAP	
	PROJECT NO. 16134	February 2023	**Figure 3.1**

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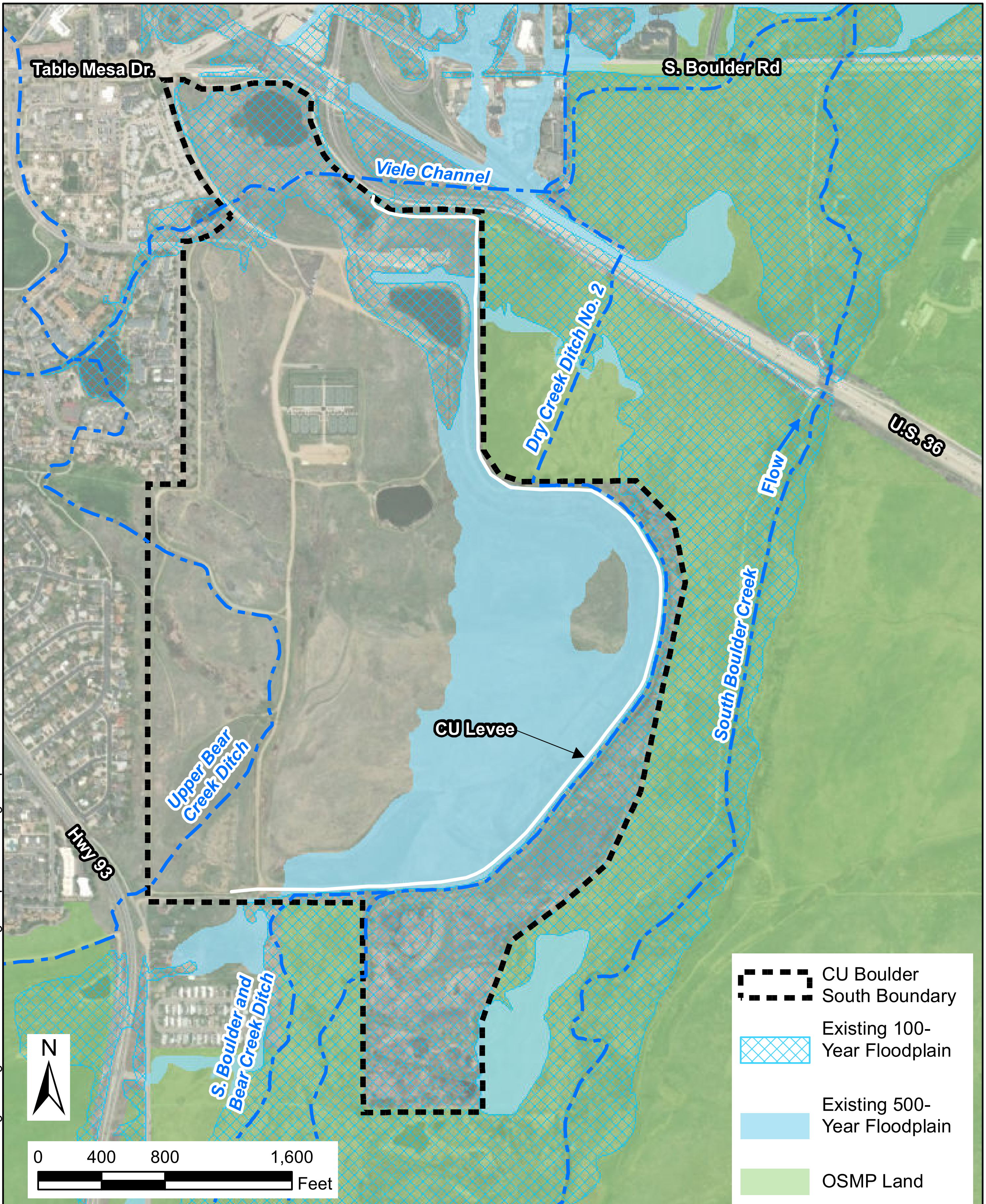

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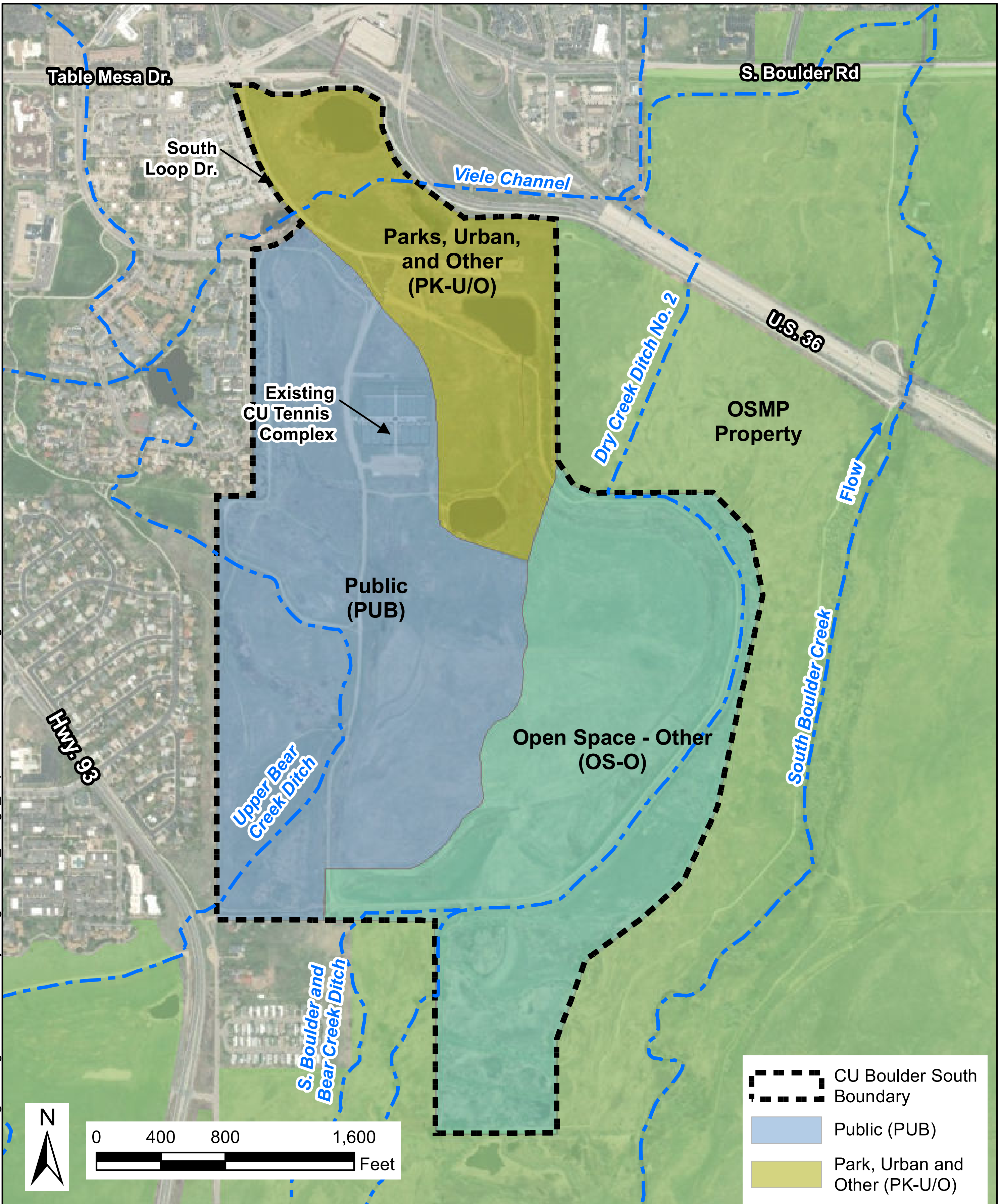
REPRODUCE IN COLOR	RJH CONSULTANTS, INC.	SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT	SITE PLAN
		PROJECT NO. 16134	February 2023 **Figure 3.2**



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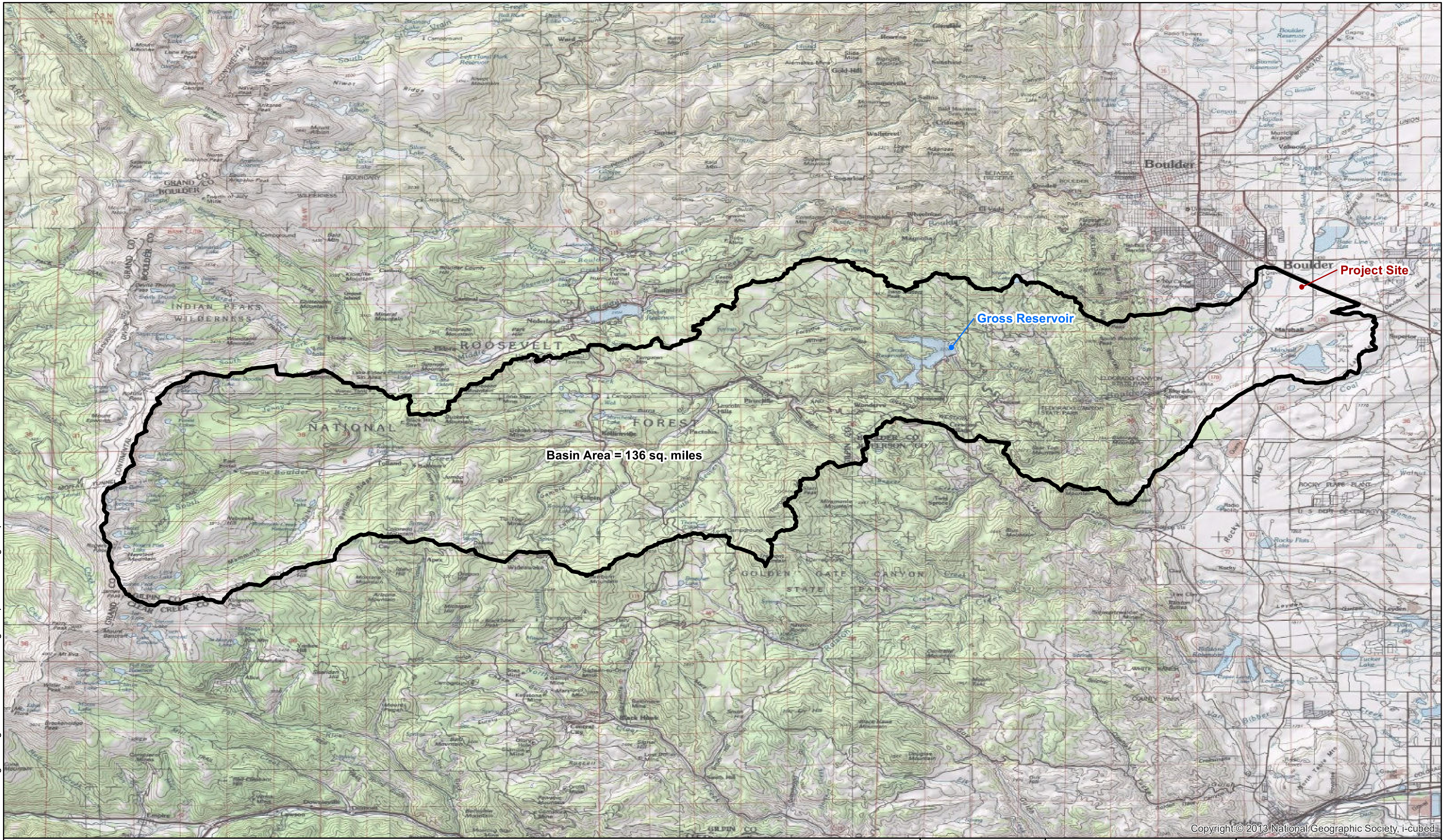
REPRODUCE IN COLOR	RJH CONSULTANTS, INC.	SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT	CU BOULDER SOUTH CAMPUS AND LAND USE DESIGNATIONS
		PROJECT NO. 16134	February 2023 **Figure 3.3**



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Miles

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RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT

PLAN OF SOUTH BOULDER CREEK WATERSHED

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Figure 3.4

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SECTION 4 - PROJECT DESCRIPTION

4.1 General

The primary Project components include an embankment dam along the north and west portion of the CU Boulder South campus, a structural spillway wall on OSMP property along the CDOT ROW, an outlet tunnel below US36, detention excavation on the PK-U/O land use area, grading and site access modifications, and ecological restoration on the OS-O land use area. A description of primary Project components is provided below and shown on Figure 4.1.

4.2 Embankment Dam

The embankment dam will consist of a zoned earthfill embankment with internal filters and drains and a barrier wall in the foundation. The embankment dam will extend along the north and west portions of the CU Boulder South campus. The embankment dam will connect to natural high ground consisting of bedrock at the west (left) end and to the spillway at the east (right) end. Key components of the embankment dam will include:

- Earthfill: The earthfill will consist of a central core and upstream and downstream shells. The central core will have sufficiently low permeability to reduce seepage during transient reservoir loading. The upstream and downstream shells will consist of on-site clayey sand, sand, and gravel random fill.
- Internal filter and drains: Internal filter and drain zones will be included within the embankment to safely manage seepage through the embankment fill. The filter and drain zones will consist of specially graded sand and gravel.
- Toe drain: A toe drain at the base of the downstream slope will collect, convey, and distribute seepage. The volume of seepage will be measured and monitored and then conveyed to the exfiltration system, where it will be distributed back to groundwater.
- Barrier wall: A barrier wall will be included below the embankment dam to manage foundation seepage when the reservoir is storing water and will consist of a soil-bentonite barrier wall below the centerline of the embankment dam alignment. The barrier wall will connect to the embankment dam fill at the ground surface and extend into the underlying Pierre Shale bedrock to provide a continuous low-permeable seepage barrier along the dam alignment.

Additional information regarding the embankment design is presented in Section 10.

4.3 Spillway

The spillway will consist of an above-ground concrete wall with below-ground secant piles to provide seepage control and structural support. The embankment dam and spillway wall will collectively comprise the high-hazard, jurisdictional dam. The spillway will be located to the south of the CDOT ROW on property owned by OSMP. The spillway will connect to the earthfill embankment at the west (left) end and the US36 embankment at the east (right) end. Key components of the spillway will include:

- Spillway wall: The spillway wall will consist of a vertical, reinforced concrete wall. The spillway wall will retain flows during flood events up to and including the 100-year event and will convey flows from more extreme flood events over the top of the wall.
- Spillway foundation: The foundation will be comprised of a row of secant piles extending into the bedrock. The secant pile wall will provide both structural support for the spillway wall and seepage control.
- Spillway apron: The spillway apron will provide energy dissipation for flows overtopping the spillway wall and will consist of a reinforced concrete slab with an end sill.
- Landscaping and Aesthetics: Earthfill is planned to be placed along the downstream face of the spillway wall to assist with concealing portions of the wall and limit opportunities for vandalism. Aesthetic treatment of exposed faces is planned for both the upstream and downstream spillway wall and is expected to include concrete stamping and possibly concrete coloring. This concept was identified during this phase of design but this work is still in the concept development phase.

Additional information regarding the spillway design is presented in Section 11.

4.4 Groundwater Conveyance System

The groundwater conveyance system will allow normal groundwater to pass through the spillway foundation. Conveyance of normal groundwater flows is critical to maintaining the existing hydrogeologic system and prevent upstream groundwater mounding and downstream groundwater decline, which could impact wetlands and critical habitats. The groundwater conveyance system will consist of facilities for collecting, conveying, and distributing groundwater. Key components of the groundwater conveyance system will consist of:

- Collection: Groundwater will be collected upstream of the spillway in a collection trench using slotted pipes and permeable backfill.

- Conveyance: Groundwater will be conveyed from the upstream side of the spillway to the downstream side in a connector pipe. Gates will be installed that can be manually adjusted to control the volume of flow from the collection pipes and through the connector pipes.
- Distribution: Groundwater will be distributed downstream of the spillway using a distribution trench consisting of slotted pipes and permeable backfill.

Additional information regarding the groundwater conveyance system design is presented in Section 12.

4.5 Outlet Works

The lower portion of the reservoir pool will not freely drain back to SBC. An outlet works will be required to meet SEO dam safety requirements and to allow the entire reservoir to be drained to meet water rights requirements. The outlet works will extend from the detention excavation to Viele Channel north of US36. Key components of the outlet works will include:

- Intake Structure: The intake structure will consist of a reinforced concrete sloping riser structure located at the upstream toe of the detention excavation. The front and top of the structure will be open and include steel trashracks. The trashrack at the top of the structure will be a pentagonal prism shape.
- Conduit: The conduit will consist of an 875-foot long, 60-inch diameter steel pipe. The pipe will be installed using a combination of open excavation and tunneling techniques. The portion of pipe upstream of US36 (615 linear feet) will be installed in an open excavation and encased in flowfill (i.e., low strength concrete) for corrosion protection. The portion of the conduit underneath US36 (260 linear feet) will be installed by tunneling. The tunneled portion of the pipe will consist of a 60-inch diameter carrier steel pipe within a 96-inch diameter casing pipe.
- Outlet Structure: The outlet structure will consist of reinforced concrete apron with end sill at the downstream end of the outlet works conduit. The outlet structure will discharge to Viele Channel. Riprap will be installed in Viele Channel in the vicinity of the outlet structure for erosion protection.

Additional information regarding the outlet works design is presented in Section 13.

4.6 Site Drainage

Several natural drainages and irrigation ditches flow through the Project site. The Project facilities will impact DCD2, US36 wildlife crossing, and site drainage below US36.

Modifications to existing facilities will be required to maintain site drainage and historic irrigation operations at the site. Key components of the site drainage and irrigation facilities modifications will include:

- DCD2: The spillway alignment intersects DCD2. The reinforced box culvert below US36 will be extended to the face of the spillway wall. The culvert extension will consist of a 4.5-foot (span) by 4-foot (height) reinforced concrete box culvert. The culvert extension is 18 inches narrower than the existing DCD2 culvert below US36. The narrower extension will restrict flows during the 100-year flood event to below existing conditions 100-year flows through the DCD2 culvert.
- Wildlife Crossing: The spillway alignment is located approximately 53 feet upstream of the face of the wildlife crossing. The wildlife crossing will be extended to provide the same interior dimensions to the face of the spillway wall to facilitate continued wildlife access.
- US36 Culverts: The OSMP property south of US36 drains through a series of culverts below US36. These culverts include the US36 wildlife crossing and DCD2 crossing discussed above and multiple smaller culverts. The spillway alignment is located between 25 to 40 feet upstream of the face of these culverts. The drainage culverts will be extended to the upstream face of the spillway wall. The concepts for these culverts will be impacted by the landscape berms, but the general concept will be to extend the culverts and install riser-type inlets near the existing inlet of each culvert to facilitate drainage of the area between the US36 embankment and the spillway wall.

Additional information regarding site drainage and irrigation facilities is presented in Section 14.

4.7 Site Grading and Access

Site grading and site access modifications will be required to support the Project facilities discussed in the preceding sections and to meet Project design criteria. Key components of the site grading and access modifications will include:

- Detention Excavation: Between approximately 73 to 105 acre-feet (ac-ft) of detention storage is needed to meet hydraulic and floodplain design criteria. The detention storage will be achieved by excavation on the northern portion of the CU Boulder property. A barrier wall will be installed along the perimeter of the detention excavation to isolate this area from groundwater so that the required capacity for flood mitigation can be maintained.

- South Loop Drive Modifications: The embankment dam will extend across South Loop Drive. An earthen roadway ramp will be constructed to provide access for South Loop Drive over the earthen dam following construction. The design elevation for grading the earthen roadway ramp south of the embankment dam will be set at the 500-year water surface elevation, and the south end of the ramp will terminate at existing ground on the CU Boulder PUB land use area. The existing Viele Channel culvert that extends below South Loop Drive and will be removed and replaced with a longer culvert to accommodate the earthen roadway ramp.
- Multi-Use Trail Modifications: The alignment of the spillway connection to US36 extends across the existing multi-use trail. An earthfill ramp will be placed along both sides of the spillway wall at this location to accommodate the multi-use trail.
- Levee Removal: The existing levee will be partially removed to connect existing ground on both sides of the levee.
- OS-O Inflow Rundown: An inflow rundown will be constructed at the south end of the OS-O land use area to appropriately convey inflows from the SBC floodplain. The inflow rundown will consist of about a 600-foot-wide sloping area at a 5-percent longitudinal slope constructed by a combination of fill and excavation. The inflow rundown will be lined with buried riprap.
- Miscellaneous Site Grading: Miscellaneous site grading will be required to promote site drainage to SBC, Viele Channel, US36 culverts, and the detention excavation.
- Access Roads: Access will be required to inspect and maintain Project facilities. Gravel access roads will be included in the future stages of design where needed.

Additional information regarding the site grading and access design is presented in Section 15.

4.8 Ecological Mitigation and Restoration

Construction of Project facilities will impact jurisdictional and non-jurisdictional wetlands and T&E species habitat for the PMJM and ULTO. The Project is also anticipated to impact mesic tallgrass prairie, uplands, and northern leopard frog habitat. Impacts to regulated environmental resources will be mitigated by constructing new areas of wetlands and habitats for PMJM and ULTO. The mitigation will be constructed on-site in the OS-O portion of the CU Boulder South campus and will be performed in conjunction with a larger ecological restoration and conservation of this area. The new wetlands and habitat will be graded and revegetated to facilitate suitable habitat.

SECTION 5 - DATA COLLECTION

5.1 General

Various types of data collection have been and will be required to advance the design. The RJH Team performed topographic surveying, a subsurface utility engineering investigation, a geotechnical investigation program, and environmental surveys. A description of data collection performed is provided below.

5.2 Topographic Survey

Merrick performed topographic surveying in the fall of 2022 to develop a base map for the Project site. Topographic surveying was performed using a combination of aerial survey equipment and conventional (i.e., field) survey equipment. Bathymetric field surveying was performed at cross sections every 50 feet for approximately 22,000 linear feet along South Boulder Creek, Viele Channel, and other tributaries within the survey limits. Merrick ordered title commitments for parcels within the survey area. Property boundaries were drafted from vesting deeds based on section corners and field surveying of monumentation for each property. Easements and rights-of-way identified in the title commitments were drafted to show evidence of possible utilities, public and private rights-of-way, and other encumbrances that may impact the Project. The limits of the 2022 survey are presented on Figure 5.1.

5.3 Subsurface Utility Engineering Investigation

Colorado Revised Statutes, Title 9, Article 1.5 (CRS 9-1.5) (Colorado State Legislature, 2018) requires SUE for any project with subsurface excavations. A SUE investigation was performed by SAM. SUE is typically performed in two phases to achieve the required level of quality. The quality level (QL) is described in the American Society of Civil Engineers (ASCE) 38-02, *Standard Guideline for the Collection and Depiction of Existing Subsurface Utility Data* (ASCE, 2002), and is summarized as follows.

- QL-D: Information comes solely from existing utility records and as-built drawings.
- QL-C: Involves surveying visible utility facilities, such as manholes, valve boxes, posts, etc., and correlating this information with existing utility records (QL-D).
- QL-B: Involves the use of surface geophysical techniques to determine the existence and horizontal position of underground utilities. This information may be sufficient to accomplish preliminary engineering goals.

- QL-A: Involves the use of nondestructive digging equipment at discrete, critical points to determine the precise horizontal and vertical position of underground utilities, as well as the type, size, condition, material, and other characteristics. This activity is called "locating" and is appropriate for developing bid documents.

A QL-A SUE survey for areas within the detention facility footprint was performed in 2023. Additional SUE surveying with the ecological restoration area will be performed in early 2024. A plan of the limits of the SUE survey is presented on Figure 5.2.

5.4 Geotechnical Investigations

RJH performed geotechnical investigations to obtain subsurface data needed to advance the Project design. The investigation was performed in three phases between 2018 and 2023. Objectives of the geotechnical investigation included:

- Advancing the generalized understanding of geologic, geotechnical, and hydrogeological conditions at and around the site.
- Evaluating foundation conditions along the alignment of the spillway, outlet works tunnel, soil-bentonite barrier wall, and embankment.
- Evaluating available on-site borrow materials.
- Evaluating subsurface conditions in proposed environmental reclamation areas.

The geotechnical investigation included the following components:

- Performing geological mapping.
- Drilling 70 borings at the Project site and in the SBC alluvial valley upstream and downstream of the Project site.
- Installing monitoring wells with data logging piezometers in 54 of the borings and installing data-logging piezometers in five monitoring wells owned by OSMP to provide long-term monitoring of groundwater levels.
- Installing datalogging piezometers in three stilling wells to monitor surface water levels.
- Performing a geophysical survey near the spillway alignment to identify the top of bedrock, confirm the presence of any paleochannels, and provide data to support boring locations. The geophysical investigation was performed by Collier Geophysics, LLC (Collier).
- Excavating 19 test pits to evaluate onsite borrow materials.

- Performing hydraulic conductivity tests in surficial soil and bedrock and water pressure (Packer) tests in bedrock.
- Performing laboratory testing on collected subsurface materials.

A summary of data collected, and laboratory test results is presented in the *Phase I Geotechnical Report - South Boulder Creek Regional Detention* (RJH, 2019), *Phase II Geotechnical Report - South Boulder Creek Regional Detention* (RJH, 2022b), and *Phase III Geotechnical Report – South Boulder Creek Regional Detention* (RJH, 2023). Additional geotechnical investigations may be performed in subsequent stages of Project development as appropriate to advance the design.

Additional information regarding the geotechnical site conditions is presented in Section 8.

5.5 Environmental Investigations

5.5.1 General

Construction of Project facilities will impact wetlands and T&E species habitat for the PMJM and ULTO. These impacts will require obtaining environmental permits which are presented in Section 7 and Section 16. Environmental investigations performed to support environmental permitting included:

- Wetland survey
- ULTO surveys
- Cultural resource evaluation

A description of the environmental investigations is provided below. A composite environmental resources map was developed for the Project area, which is presented on Figure 5.3. The data presented on Figure 5.3 are based on field surveys performed by CORVUS in 2023, ULTO data from the City and Colorado Natural Heritage Program, wetland data from the City, and PMJM habitat data from the City and USFWS.

5.5.2 Wetland Survey

CORVUS performed an environmental survey in July 2023 that included identifying channels, ditches, open water, and wetlands. The wetland determination followed methods described in the USACE *Wetlands Delineation Manual* (USACE, 1987) and, where applicable, in accordance with the methods identified in the Regional Supplement to the

USACE Wetland Delineation Manual: Great Plains Region (Supplement) (USACE, 2010). The survey limits generally included the US36 corridor and CU Boulder South Campus.

5.5.3 ULTO Surveys

CORVUS performed field surveys to assess existing populations or individuals of ULTO in and near the Project site. Three years of ULTO surveys are generally required for Formal Consultation with the USFWS. USFWS requires surveys be performed during the flowering period, which is typically between July 20 to August 31. ULTO surveys were completed during the following ULTO flowering seasons:

- Project Area except Southern Ponds Area:
    - 2020 ULTO Survey: August 10 and 18, 2020
    - 2021 ULTO Survey: August 9 and 11, 2021
    - 2022 ULTO Survey: August 9 and 10, 2022
- Southern Ponds Area:
    - 2023 ULTO Survey: August 7, 2023

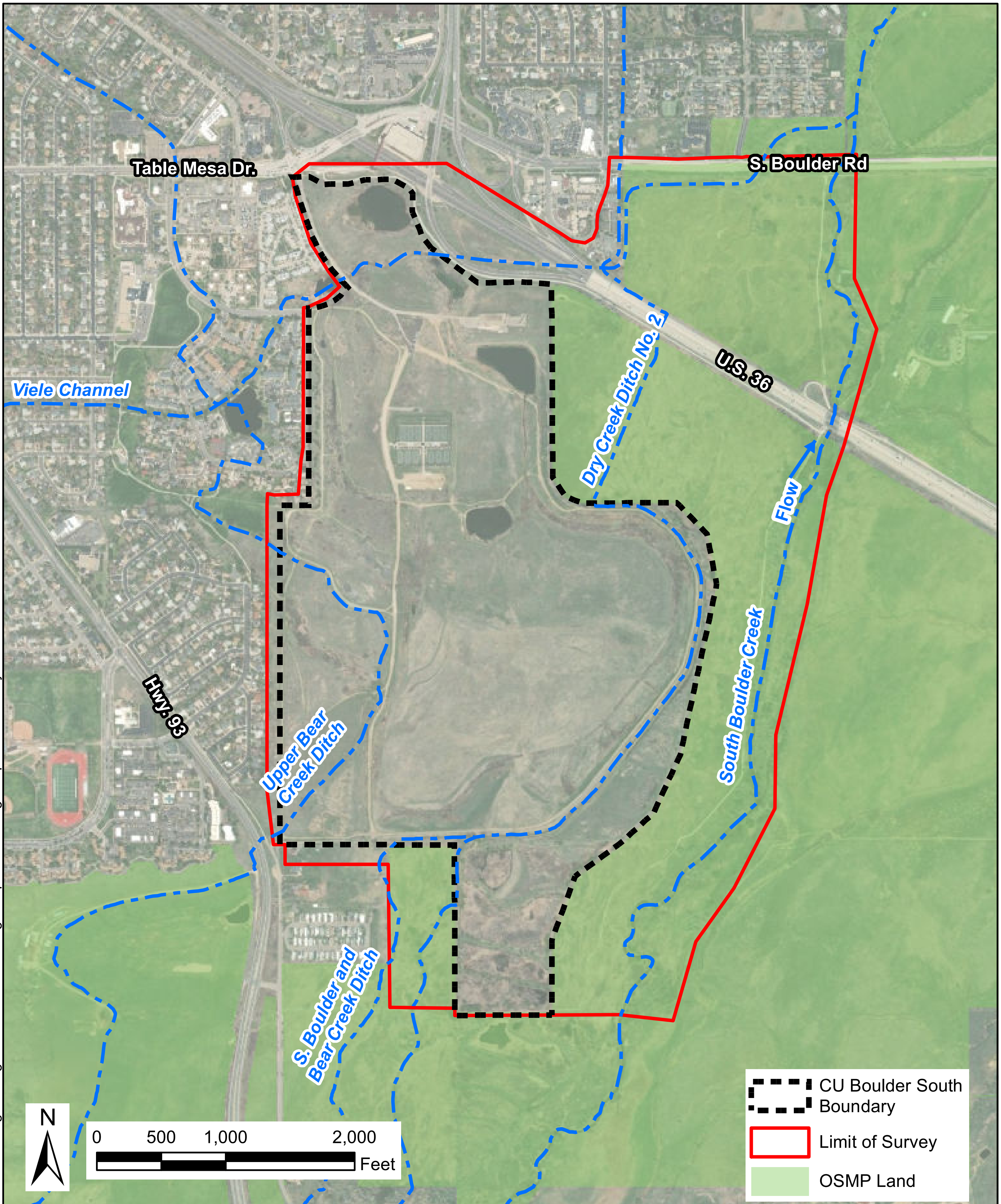
ULTO habitat was identified based on the presence of common associated species identified in the USFWS *Interim Survey Requirements for Spiranthes diluvialis* (USFWS, 1992). GPS coordinates were collected for each plant occurrence. If multiple plants occurred within the same square-foot, a note of the number of individuals was made. For populations of about 500 or more individual ULTO plants, the plants were counted, and the boundary of the population was mapped.

ULTO plants in the Project site can be separated into two populations based on hydrology and plant community. Population 1 is located on the dry-side of the levee embankment on CU Boulder South campus, and population 2 is located on the wet-side of the levee embankment on CU Boulder South campus and OSMP within the SBC floodplain. In population 1, ULTO habitat generally occurred in a narrow band of wetlands that is bordered by uplands on both sides. No individual ULTO were observed within the wetlands along the west side of CU Boulder South campus; wetlands in this area generally lacked commonly associated species and were overall drier or transitioning to uplands. In population 2, most individual ULTO were observed south of US36, and this ULTO population connects with populations outside the Project site that are monitored by OSMP. Also, individual ULTO were observed along the banks of the southern ponds.

5.5.4 Cultural Resources

PaleoWest completed a Class III cultural resources survey for the Project area. The goals of the cultural resource survey were to identify cultural resources within the identified area of potential effects (APE), make recommendations on their eligibility for the National Register of Historic Places (NRHP), and propose management planning efforts for eligible sites. The cultural resource survey was performed for compliance with Section 106 of the National Historic Preservation Act (NHPA), which requires agencies of the U.S. Government to consider the effect of federal undertakings on "historic properties." As defined by implementing regulations, historic properties are "any prehistoric or historic district, site, building, structure, or object included in, or eligible for inclusion in the National Register of Historic Places" (36 CFR 800.16).

PaleoWest performed the cultural resource inventory on February 13 and 14, 2023. PaleoWest conducted a field inventory using transects spaced no more than 15–20 meters (m) apart. During the inventory, PaleoWest documented three unrecorded segments of one previously record site and five isolated finds. PaleoWest also revisited the location of three previously recorded sites, but determined they were no longer in existence within the APE. PaleoWest identified one of the sites as eligible for inclusion in the National Registry of Historic Places but identified that the Project would have no adverse effect on historic properties.



REPRODUCE IN COLOR	RJH CONSULTANTS, INC.	SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT	PLAN OF SURVEY LIMITS
		PROJECT NO. 16134	February 2023 **Figure 5.1**

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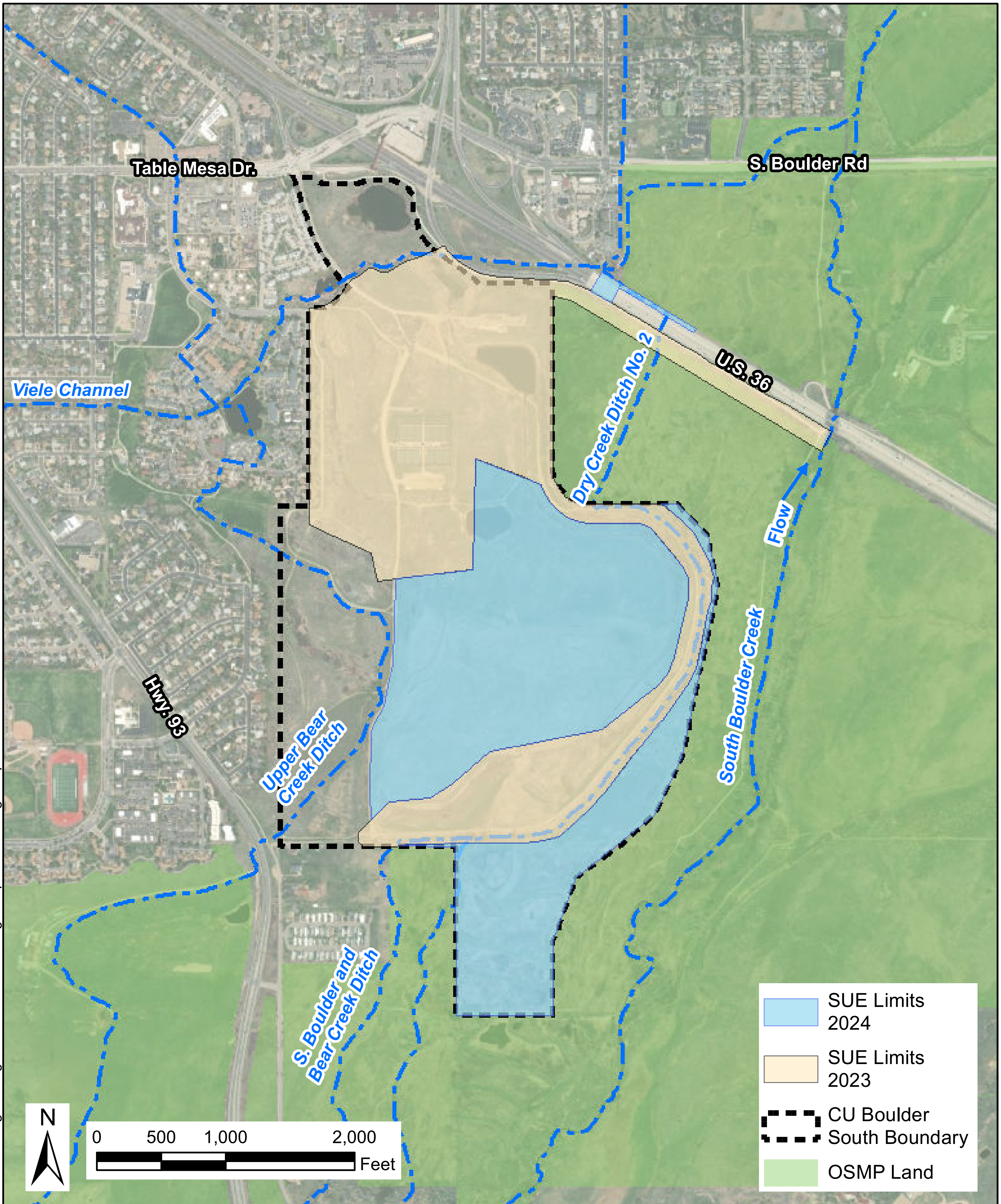

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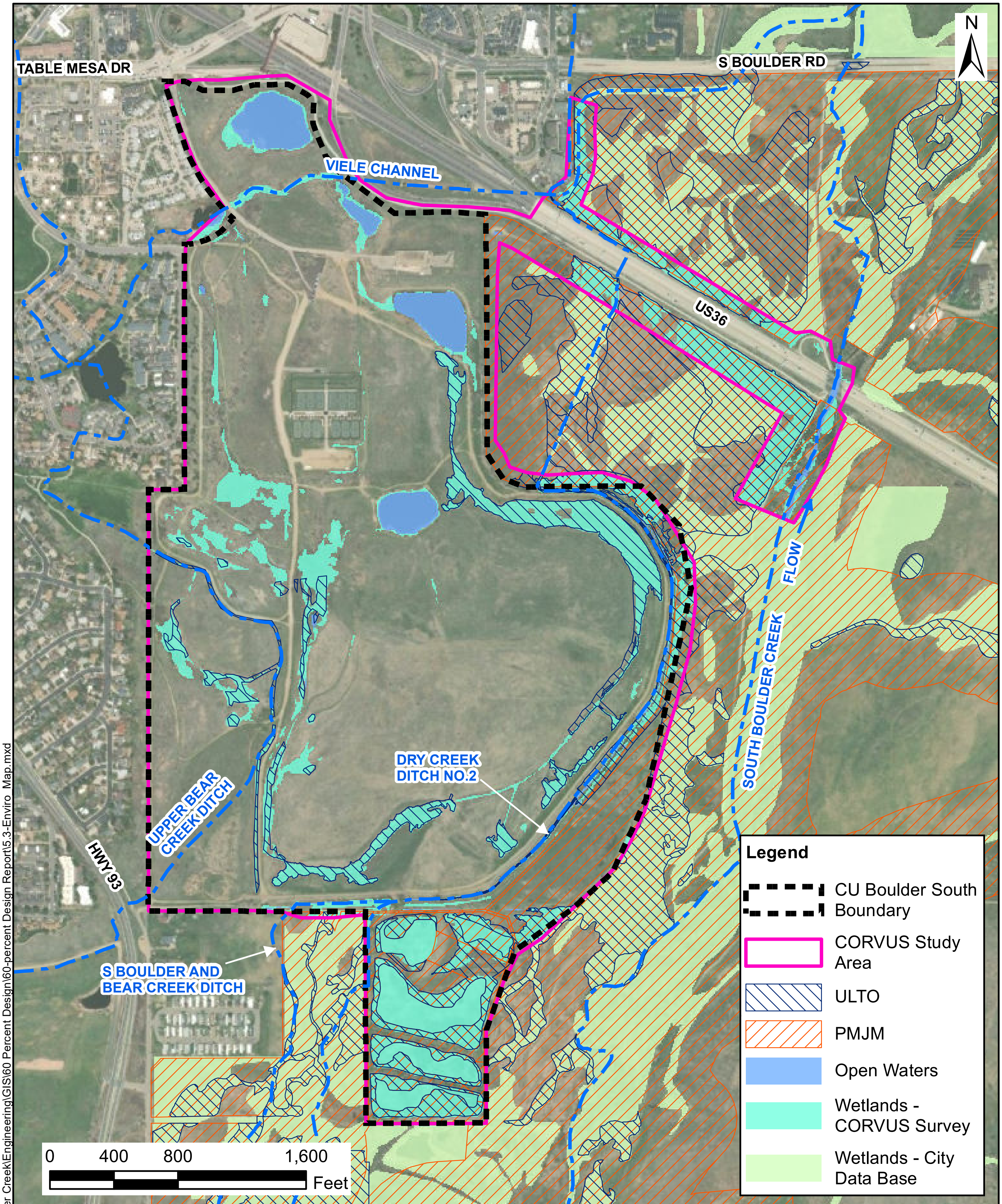
REPRODUCE IN COLOR	RJH CONSULTANTS, INC.	SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT	PLAN OF SUBSURFACE UTILITY ENGINEERING SURVEY LIMITS
		PROJECT NO. 16134	February 2023 **Figure 5.2**



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REPRODUCE IN COLOR

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT

COMPOSITE PLAN OF ENVIRONMENTAL RESOURCES

PROJECT NO. 16134

February 2023

Figure 5.3

SECTION 6 - SUPPLEMENTAL REPORTS

6.1 General

RJH collected and evaluated geotechnical and groundwater data and performed hydrologic and groundwater modeling prior to and concurrent with development of the design presented in this Report. The methodology, results, and conclusions of this data collection and modeling is presented in the supplemental reports summarized below.

6.2 Hydrology Report

RJH performed hydrologic analyses based on SEO guidelines to develop the IDF, which is the regulatory flood event used for spillway sizing. The IDF for an extreme hydrologic hazard dam is the Probable Maximum Flood (PMF). The IDF is documented in the *South Boulder Creek Regional Detention Project Hydrology Report* (Hydrology Report) (RJH, 2022a). The controlling IDF event is a 6-hour Local Storm located on the portion of the watershed downstream of Gross Reservoir. The IDF has a peak flow of 83,282 cubic feet per second (cfs) and an inflow volume of 23,792 ac-ft. The Hydrology Report was reviewed and accepted by the SEO in 2022.

6.3 Geotechnical Data Reports

As previously discussed in Section 5, RJH performed a geotechnical investigation program to obtain subsurface data needed to advance the Project design. A summary of data collected, and laboratory test results is presented in the *Phase I Geotechnical Report - South Boulder Creek Regional Detention* (RJH, 2019), *Phase II Geotechnical Report - South Boulder Creek Regional Detention* (RJH, 2022b), and *Phase III Geotechnical Report – South Boulder Creek Regional Detention* (RJH, in progress). Additional information regarding the geotechnical site conditions is presented in Section 8.

6.4 Baseline Groundwater Model Report

Construction of Project facilities could impact natural groundwater conditions in the vicinity of the Project. RJH developed a baseline groundwater model (Baseline Model) to support the evaluation of Project impacts and to support the design of the facilities to mitigate the impacts. Groundwater modeling was performed using the MODFLOW-USG software program. The modeled area included SBC and the adjacent alluvial valley from about Highway 93 at the upstream end to Baseline Road at the downstream end.

The Baseline Model was developed using Project-specific data, publicly available data, and information provided by OSMP. This model simulated conditions from November 2018 through October 2019 and was calibrated to data collected by RJH from 32 monitoring wells throughout the Project site. The Baseline Model had an unweighted scaled root mean squared (RMS) error of 1.1 percent following calibration, which is within the industry-acceptable limit of less than 5 percent.

The groundwater conditions simulated by the Baseline Model are consistent with RJH's conceptualization of the hydrogeologic system of the Project site and vicinity, and key takeaways from the Baseline Model include:

- Groundwater levels decline toward the north through the aquifer, which generally follows the slope of topography and flow of SBC.
- Total flows through the model vary from about 350,000 cubic feet per day (ft^3/d) during the summer to 115,000 ft^3/d during the winter.
    - Predominant components of the hydrogeologic system are inflow from recharge (both irrigation and natural precipitation), outflow from evapotranspiration, and interactions with surface water in SBC. These predominant components account for flows that range from 65 to 82 percent of the total flow through the modeled area throughout the year.
    - Groundwater flow is a relatively minor contributor to the overall flows through the hydrogeologic system within the modeled area. Groundwater flow rates of approximately 6,000 ft^3 are predicted to occur beneath US36, which is predominantly occurring through alluvium in the western portion of the Project site. The total groundwater flow is relatively stable seasonally and ranges from approximately 2 percent of the hydrogeologic system in the summer to 5 percent in the winter.
- The alluvial aquifer does not appear to be strongly gaining water from or strongly losing water to SBC.
- Seasonal groundwater fluctuations are influenced by natural conditions through the hydrogeologic cycle and irrigation applied to OSMP fields.
- The model was most sensitive to irrigation recharge rates and the alluvium specific yield.

In RJH's opinion, the Baseline Model provides a reasonable approximation of the existing groundwater system in the Project vicinity and is suitable for evaluating impacts that Project components could have on the hydrogeologic system and for supporting design of Project features that mitigate impacts to the existing groundwater system. The baseline groundwater modeling is presented in the *Baseline Groundwater Model Report - South Boulder Creek Regional Detention* (RJH, 2021).

SECTION 7 - BASIS OF DESIGN

7.1 General

The Project will be advanced based on regulatory criteria, City and MHFD criteria and preferences, and stakeholder criteria and preferences. RJH identified a list of anticipated regulatory agencies and key project stakeholders, and based on our current understanding of their criteria and preferences, we developed design criteria that have been used to develop this 60-percent design. The design criteria will be refined as the Project progresses based on continuing discussions with the regulatory agencies and Project stakeholders.

7.2 Regulatory Agencies

We anticipate that approval of the Project will be required from the following regulatory agencies:

- SEO: The embankment dam, spillway, and appurtenances will collectively comprise a jurisdictional dam that will be regulated by the SEO. The dam is expected to be classified as a high hazard potential dam and an extreme hydrologic hazard dam. The dam will also retain stormwater runoff, and the runoff will need to be released within a specified time to meet SEO water right's requirements.
- CDOT: A portion of the spillway and a portion of the outlet works will be located within the CDOT ROW along US36, and obtaining a CDOT access permit will be required. CDOT will consult with the Federal Highway Administration (FHWA).
- USACE: A Clean Water Act Section 404 permit will be needed to construct the project because of anticipated impacts to wetlands. The USACE will be the lead regulatory agency for this permit. Other agencies that may consult with the USACE regarding the 404-permit application are expected to include the USFWS, Environmental Protection Agency, Colorado Department of Public Health and Environment (CDPHE), and the State Historical Presentation Office (SHPO).
- FEMA: The Project will impact the regulatory floodplain along SBC and will require obtaining a Conditional Letter of Map Revision from the FEMA prior to construction.
- City: A City Wetland Permit will be required to construct the Project because of anticipated impacts to wetlands. This permit will be obtained from the City Planning Department. Other City permits are anticipated to be required to construct the Project, but these will be obtained by the contractor.

7.3 Project Stakeholders

The following stakeholders have provided and/or will provide input regarding design preferences and criteria:

- OSMP: A portion of the spillway and outlet works, and groundwater monitoring wells (instrumentation) will be constructed on OSMP property. Open Space Board of Trustees (OSBT) will need to issue a land disposal prior to construction. OSBT and OSMP staff reviewed and provided comments on the 30-percent design and will be provided an opportunity to review 60-, and 90-percent design submittals. Also, through the design process, OSMP staff have provided and will continue to provide input for development of the environmental and ecological restoration.
- CU Boulder: The dam embankment, detention excavation, levee removal, ecological restoration, and portions of the spillway and outlet works will be constructed on CU Boulder property. CU Boulder and the City authorized an annexation agreement in 2021, and relevant terms of that agreement have been incorporated into the design. CU staff will be provided an opportunity to review 60-, and 90-percent design submittals and provide comments.
- CDOT: Construction of the portion of the spillway that will connect to the US36 embankment will need to be coordinated with CDOT. The spillway will protect US36 from overtopping from flood events up to and including the 100-year event. CDOT staff reviewed the 30-percent design submittal and led a Field Inspection Review (FIR) workshop following their review of the 30-percent design submittal. Also, CDOT has participated and will continue to participate in monthly meetings with the City and RJH throughout the design process. CDOT staff will be provided with an opportunity to review the 60- and 90-percent design submittals. CDOT reviewed the landscape architecture concepts along the spillway and provided comments.
- MHFD: MHFD will be a funding partner for construction and has been actively involved in development of the concept and designs for the Project. MHFD staff reviewed and provided comments on the 30-percent design and will review 60-, and 90-percent design submittals and provide input. Also, through the design process, OSMP staff have provided and will continue to provide input for development of the environmental and ecological restoration.
- DCD2 Company: DCD2 is owned and maintained by the DCD2 Company. The spillway will intersect DCD2 approximately 44 feet upstream of US36, and DCD2 facilities at the spillway will need to be modified to accommodate the Project.

7.4 State Regulatory Status

7.4.1 Jurisdictional Size

The SEO has established criteria to identify the jurisdictional size of a dam. Jurisdictional dams in Colorado are regulated and subjected to the authority of the SEO. In accordance with Rule 4.6.1 of the SEO Rules and Regulations, a jurisdictional size dam must meet one of the following criteria:

- Reservoir with a capacity that exceeds 100 ac-ft.
- Reservoir surface area that exceeds 20 acres at the maximum normal pool.
- Jurisdictional height that exceeds 10 feet.

The dam for this Project meets all those criteria and will be regulated by the SEO as a jurisdictional dam. Additional information regarding RJH's evaluation of the jurisdictional size is presented in Appendix A.

7.4.2 Hazard Classification

The SEO has established criteria to determine the hazard classification of a dam. The hazard classification establishes all of the SEO's design criteria for a dam except for spillway sizing. The hazard classification is identified based on potential consequences associated with a failure of the dam with the water surface elevation up to the spillway crest (i.e., sunny-day failure). A high hazard dam is a dam for which loss of human life is expected to result from a dam failure. RJH performed a simulated sunny-day dam breach evaluation in general accordance with the SEO's Guidelines for Dam Breach Analyses (SEO, 2020a). Based on this evaluation, loss of life is expected to result from a dam failure, and we have developed the design based on a high hazard classification. Additional information regarding RJH's evaluation of the hazard classification is presented in Appendix A.

7.4.3 Hydrologic Hazard Classification

The SEO has established criteria to determine the hydrologic hazard classification of a dam. The hydrologic hazard classification establishes design criteria for spillway sizing. The hydrologic hazard classification is identified based on potential consequences associated with an overtopping failure of the dam during the IDF. For 60-percent design, we assumed the dam will have an extreme hydrologic hazard classification based on the proximity of the dam to US36, the Tantra neighborhood, and the Manhattan Circle office complex. Even if the

IDF was reduced, the length of the spillway will not change because the length is fixed based on the criteria of reducing impacts to OSMP property. However, a reduction in the IDF may result in a minor decrease to the height of the dam embankment. The design has been developed based on an extreme hydrologic hazard classification.

7.5 Design Criteria

7.5.1 Regulatory Criteria

7.5.1.1 Water Rights

Water rights requirements for a legally-protected stormwater detention facility were identified from Colorado Revised Statute 37-92-602 (Colorado State Legislature, 2015). The detention facility must:

- Continuously release or infiltrate at least 97 percent of all the runoff from a rainfall event that is less than or equal to a 5-year storm within 72 hours after the end of the event.
- Continuously release or infiltrate at least 99 percent of the runoff within 120 hours after the end of events greater than a 5-year storm.
- Operate passively and not subject the stormwater runoff to any active treatment process.

7.5.1.2 Dam Safety

Dam safety requirements were identified based on requirements from the SEO Rules and Regulations for Dam Safety and Dam Construction (Rules and Regulations) (SEO, 2020a).

<u>Embankment</u>:

- The minimum embankment freeboard should meet both normal and residual freeboard requirements:
  - Normal freeboard should be 3 feet or the wave setup and runup generated by a sustained 100 miles per hour wind, whichever is greater. Normal freeboard is the vertical distance between the top of the spillway and crest of the embankment dam.
  - Residual freeboard should be 1 foot or the wave setup and runup generated by a 10 percent annual exceedance probability wind, whichever is greater. Residual freeboard is the vertical distance between the routed IDF elevation and the crest of the embankment dam.

- The crest width must be equal to the jurisdictional height of the dam in feet divided by 5 plus 10 feet.
- The crest should have a camber sufficient to maintain the design freeboard based on the anticipated magnitude of crest settlement. Camber should be no less than 0.5 foot or the predicted deformation (settlement) of the dam, whichever is greater.
- Roads located on the dam crest should have appropriate surfacing material to resist rutting and provide adequate traction in wet conditions.
- Embankment dams must be designed to have stable slopes during construction and under all conditions of reservoir operation with factors of safety based on EM-1110-2-1902 (USACE, 2003b). Table 7.1 presents the required minimum safety factors for various load conditions.

TABLE 7.1
REQUIRED MINIMUM SAFETY FACTORS

Load Condition (Analyzed Slope)	Minimum
Steady State Seepage - Empty Reservoir (Upstream and Downstream)	1.5
Steady State Seepage - Full Reservoir (Upstream and Downstream)	1.4
End of Construction (Upstream and Downstream)	1.3
Rapid Drawdown (Upstream)	1.1-1.3

- The SEO Rules and Regulations and documents referenced therein do not discuss embankment stability requirements for a transient loading condition, which will be more appropriate for a dry flood control dam. Transient loading criteria for embankment stability will be discussed with the SEO in the future stages of design.
- Steady state seepage loading conditions for both a full reservoir and an empty reservoir were evaluated to be conservative. Embankment stability under transient loading conditions will be evaluated in the final design to evaluate how the embankment will respond to short-term hydraulic loads associated with temporary flood retention.
- The SEO Rules and Regulations and documents referenced therein do not specify a recurrence interval to be used for seismic loading. A 5,000-year return frequency was used as the design seismic load.
- Upstream slope protection for wave action is required on the entire upstream slope unless lesser coverage can be justified based on engineering analysis and reservoir operational criteria. The upstream slope protection should consist of riprap or a hardened lining (e.g., soil cement), but geosynthetics may be accepted by the SEO on a case-by-case basis. The reservoir will typically be dry, so continual wave erosion is

not a significant concern. Therefore, justification will be developed for lesser coverage during the next phase of design.

- A minimum corridor of 50 feet should be provided beyond the downstream toe of the dam for maintenance. For this Project, the 50-foot offset will be from the CU Boulder South campus property boundary or the top of bank of Viele Channel, whichever is more restrictive. This 50-foot offset will extend into the CDOT ROW in places, and a maintenance agreement with CDOT will be required.

Spillway:

- The spillway should be capable of conveying the IDF, which is based on the PMF for an extreme hydrologic hazard dam. The IDF is documented in the South Boulder Creek Regional Detention Project Hydrology Report (RJH, 2022a).
- The starting water surface elevation when routing the IDF should be the spillway crest unless a lower water surface elevation can be justified. We considered the reservoir is empty at the beginning of the IDF because the reservoir will typically be dry.
- The spillway wall will retain the maximum normal pool and be considered part of the dam. The spillway wall will be designed to meet structural requirements for concrete dams based on Gravity Dam Design EM-1110-2-2200 (USACE, 2003a)
- A minimum 5-foot crest width is required for a concrete dam. We will coordinate with the SEO to obtain a variance for this criterion in the next stage of design because a smaller width will be structurally adequate for this Project.
- Ice loading will not be considered because the reservoir will drain in less than 120 hours, and development of an ice cap is extremely unlikely.
- Spillway discharges for flows up to the IDF should not cause excessive erosion of the abutments and foundation of the spillway.

Outlet Works:

- The outlet works should be capable of releasing the top 5 feet of reservoir storage in five days (SEO, 2020a).
- Intake structures for outlet works should have a trashrack.
- The SEO Guidelines for Project Review (SEO, 2020b) provides recommendations for trashrack velocity and requirements for structural design. The maximum velocity for trashracks accessible for cleaning is 5 feet per second (fps), assuming 50 percent of the open area is clogged with debris.

- The required structural loading condition for structural design is 20 feet of differential hydraulic head.
- The outlet works should have an energy dissipator to prevent undesirable erosion or damage of nearby structures. The energy dissipator should be based on the IDF reservoir water surface elevation.

Instrumentation:

The SEO Rules and Regulations require that high hazard dams have the following instrumentation:

- Station markers every 100 feet on the crest of the dam.
- Survey monuments along the dam and top of the spillway.
- Piezometers to monitor the phreatic surface within the dam.
- Seepage measuring devices.
- Staff gage in close proximity to the outlet works with the zero mark of the gage corresponding to the invert elevation of the outlet works.

7.5.1.3 Federal 404 Permit

Requirements and criteria for the Clean Water Act (CWA) 404 permit have not been identified yet. Additional discussions with USACE and USFWS will be required. Construction of Project facilities will impact jurisdictional waters of the United States.

7.5.1.4 City Wetland Permit

Requirements and criteria for the City wetland permit have not been identified yet. Additional discussions with the City Planning department will be required. We anticipate discussions will occur early in the next stage of design.

7.5.1.5 CDOT Access Permit

CDOT requirements for the Project were identified based on a letter from CDOT to the City dated September 9, 2019, and multiple meetings and discussions between City and CDOT staff. CDOT requirements are:

- The Project cannot impede or reduce CDOT's ability to control, operate, and maintain US36.

- The spillway substructure may be located within the existing CDOT ROW. The spillway superstructure should generally be located outside of the existing CDOT ROW with the following exception:
  - A portion of the spillway superstructure can extend through the existing CDOT ROW to connect to the US36 embankment, provided it will not increase the risk of flood damage to the US36 embankment and will not result in the US36 embankment being classified as a levee by FEMA.
- Impacts to the existing US36 bridge at SBC are not acceptable. This prohibits a) physical modifications to the bridge, b) increases in hydraulic conditions through the bridge, and c) increases in scour potential through the bridge.

7.5.1.6 FEMA Floodplain Permitting

The Project was advanced based on the regulatory floodplain principle of generally not increasing downstream flood extents or depths. A CLOMR will be prepared and submitted early in the next stage of design. We anticipate additional floodplain regulatory requirements will be identified as part of the CLOMR review by FEMA.

7.5.2 City (Owner) Criteria

City requirements for the Project were based on requirements identified during design and on-going discussions with the City.

General

- Project facilities will be visible from US36, CU Boulder South campus, OSMP trails, and nearby residences. Project facilities should be aesthetically pleasing and integrate into the surrounding infrastructure and landscape.
- The multi-use trail located downstream of the spillway in the CDOT ROW must be restored following construction of the Project. A temporary detour of the multi-use trail should be provided during construction.
- The Project will be funded by the City and MHFD. Reducing costs to the extent reasonably practicable without negatively impacting Project operations, safety, or design criteria is desirable.
- Construction will require a detour of the multi-use trail, possibly impact the US36 east-bound shoulder, and create visual and noise disruptions to nearby residences and OSMP

users. Reducing the duration of construction to the extent reasonably practicable without negatively impacting Project operations, design criteria, or cost is desirable.

- The Project site will be closed to the public during construction for public safety.
- OSMP is a major stakeholder, and there will be both direct and indirect impacts to OSMP property. Impacts to OSMP property should be reduced to the greatest extent reasonably possible.

Hydraulic and Hydrologic

- Prevent overtopping of US36 from the 100-year flood event. Both the short-duration, high intensity, and long-duration 100-year events should be considered. Hydrology for the 100-year event will be obtained from the *South Boulder Creek Climatology/Hydrology Report* (HDR, 2007).
- The Project cannot negatively impact existing floodplains at any upstream or downstream location for the 100-year flood event.
- Methodology for performing hydraulic modeling and floodplain evaluations will generally be consistent with the methodology used to develop the FEMA regulatory hydraulic model.
- Viele Channel and other local off-site drainages flow through the site. Project facilities should allow off-site flows to be conveyed through or around the site without causing additional upstream or downstream flood impacts along these drainages for flood events up to and including the 100-year event.
- The facility should be designed to function with sediment and debris loads that are typical with extreme flood events.
- SBC flood flows discharging into the OS-O land use area should be conveyed in a controlled manner that does not cause excessive erosion of the old gravel pit slopes.

Hydrogeologic

- Convey groundwater through Project facilities in a manner that substantially replicates existing flow patterns to prevent upstream groundwater mounding, downstream lowering, and potential adverse impacts to existing vegetation and associated habitat.

Environmental

- Mitigate wetland and critical habitat impacts by conserving and restoring areas, and constructing new wetlands and critical habitat on the OS-O land use area of the CU Boulder South campus.

Ecological Restoration:

- Improve existing wetland, native grassland, and T&E habitats.
- Increase ecological connectivity between the restoration area and SBC.
- Ensure the long-term sustainability of wetlands and uplands considering Site hydrology.
- Minimize impacts to existing wetlands and buffer zones.
- Avoid impacts to existing irrigation ditches.
- Reduce impacts and protect T&E species habitats and communities.

7.5.3 CU Boulder Requirements

CU Boulder requirements for the Project were identified based on the annexation agreement between the City and CU Boulder and include:

- A minimum of 129 acres of developable area must be provided for future CU development. It may be acceptable to modify the configuration of developable area.
- 60 acres in the PK-U/O land use area has been designated for flood mitigation. If the City does not use the entire 60 acres for flood mitigation, the remaining area will be dedicated as open space.
- Removal of a portion or the entirety of the levee is acceptable.
- Access along South Loop Drive will be maintained at a width of 80 feet and paved to a width of approximately 24 feet. Future enlargement or enhancement of South Loop Drive will be the responsibility of CU.
- Modifications to South Loop Drive should include placing a roadway berm with the crest at the 500-year water surface elevation to prevent inundation of the PUB land use area during the 500-year flood event.
- The 100-year or 500-year floodplain limits within the PUB land use area cannot be modified without prior approval from CU.

- The detention area on the PK-U/O land use area must adequately drain during non-flooding periods. Ponding should not occur during non-flood periods.
- Aesthetics for Project facilities facing CU developable area should be coordinated with CU.
- Fill on the PUB land use area should be constructed in accordance with the CU Design and Construction Standards.

7.5.4 Dry Creek Ditch No. 2 Company Requirements

RJH developed the following design criteria for modifications to DCD2 based on engineering judgment and experience with similar projects. Additional discussions with DCD2 will be required during the next stage of design.

- Do not modify the alignment, condition, or capacity of the existing DCD2 channel.
- Extend the US36 culvert to convey DCD2 flows below US36 following construction of the Project. The culvert extension shall:
    - Convey the deeded flow rate of 69 cfs under open channel conditions (i.e., without pressuring the culvert) and without raising the water surface elevation upstream of US36 compared to existing conditions.
    - Convey the CDOT design flow rate of 123 cfs under open channel conditions and limit the water surface elevation raise upstream of US36 to within 6 inches of existing conditions.
    - Limit 100-year peak flows through the culvert following construction of the Project to the existing conditions 100-year peak flows through the culvert.

SECTION 8 - GEOTECHNICAL SITE CONDITIONS

8.1 General

Surficial soils at the Project site consist of fill and alluvium. Fill is located along the US36 embankment within CDOT ROW, in previously mined portions of the CU Boulder South campus, and in the levee that generally separates CU Boulder South campus and OSMP property. Fill is generally finer-grained soil than the alluvium; however, the fill composition is variable and ranges from clayey soil to cobbles and boulders. We classified soil within the SBC alluvial valley as undifferentiated Quaternary age (less than 2.6 million years old) alluvium, which generally consists of sand, gravels, cobbles, and boulders. Bedrock throughout the Project site is the Late Cretaceous age (66 to 100.5 million years old) Pierre Shale Formation, which is generally clayey shale with some sandstone.

8.2 Fill

8.2.1 General

Three primary areas of fill were identified: US36 embankment, CU Boulder South campus, and levee. Fill consisted of a variety of soil types and was commonly a clayey sand with some gravel.

8.2.2 US36 Embankment Fill

US36 embankment fill was encountered at the ground surface in three borings and ranged from 1 to 6 feet in thickness. The fill consisted of clayey sand with gravel, gravelly lean clay with sand, sandy lean clay with gravel, and gravelly fat clay. Uncorrected Standard Penetration Test (SPT) N-values ranged from 11 to 35 and averaged 22. The N-values were generally higher south of the multi-use trail. The fill was typically dry to moist and soft to very stiff. Liquid limits ranged from 41 to 53 and averaged 46, and plasticity indices ranged from 26 to 29 and averaged 27. The maximum particle size recovered was 1.0 inch. Pocket penetrometer results ranged from 1.0 to 4.0 tons per square foot (tsf), and the vertical hydraulic conductivity is about 1×10^{-7} centimeter per second (cm/s).

8.2.3 CU Boulder South Campus Fill

Fill on CU Boulder South campus was encountered in areas previously mined and in the berm along the west end of the Project site (west berm). Fill was generally encountered at the ground surface or below top soil and ranged from 1.2 to 26.0 feet in thickness and was underlain by alluvium or bedrock. The fill composition mostly ranged from clayey sand with gravel to lean clay. Twenty-six sampler locations encountered refusal (50 blows for less than 6 inches). At 73 other sample locations, uncorrected SPT N-values ranged from 1 to 72 and averaged 24. The fill ranged from dry to wet and very soft to very stiff or loose to very dense. One sample wase nonplastic. Twenty-five samples had liquid limits that ranged from 23 to 80 and plasticity indices that ranged from 3 to 54, with averages of 36 and 16, respectively. The maximum particle size was recovered in the test pits and was 22 inches. Pocket penetrometer results ranged from 0.25 to 3.0 tsf. Horizontal hydraulic conductivity measured from 11 rising head tests ranged from 3.4×10^{-6} to 3.6×10^{-4} cm/s, and the geometric mean was 5.4×10^{-5} cm/s. Vertical hydraulic conductivity was 2.6×10^{-5} cm/s for an intact sample of fill. Three samples that were remolded to approximately 95 percent of the standard Proctor maximum dry unit weight at about 0 to 2 percent above the optimum moisture content had vertical hydraulic conductivity of about 2.0×10^{-6} to 3.3×10^{-6} cm/s. For samples of intact fill, RJH interpreted that the drained strength failure envelope can be represented by a drained friction angle of about 33 to 37 degrees and no cohesion. The undrained strength failure envelope was interpreted to be an undrained friction angle of 15 to 17 degrees and undrained cohesion of about 31 pounds per square foot. In two borings, black gravel-sized particles were recovered and consisted of oil, grease, and Silica Gel treated-Hexane Extractable (SGT-HEM) material. The oil and grease concentration in two samples was 46,900 and 41,800 milligrams per kilogram (mg/kg), and the SGT-HEM material concentration was 12,200 mg/kg.

There are no records or test data that document the placement of the fill once the mining operations were completed. The fill appears to be non-engineered, and material properties are variable.

8.2.4 Levee Fill

Levee fill was encountered at the ground surface in four borings and five test pits, and ranged from about 4.5 to 12.6 feet in thickness. The fill mostly consisted of clayey sand with gravel and sandy lean clay and included some processed Pierre Shale. Alluvium or Pierre Shale was interpreted below the levee fill, and Pierre Shale was encountered below alluvium. A cluster of insulated electrical wires that did not appear to be continuous was encountered in one boring from about 1.0 to 1.5 feet below the ground surface. Six sampler locations encountered refusal (50 blows for less than 6 inches). At eight other sample locations, uncorrected SPT N-values ranged from 12 to 33 and averaged about 22. The fill ranged from dry to wet, sands were generally medium dense to very dense, and clays were very soft to very stiff. Liquid limits ranged from 29 to 41, and plasticity indices ranged from 4 to 21, with averages of 34 and 14, respectively. The maximum particle size recovered in a test pit was 25 inches and were mostly less than 1.5 inches.

The levee was designed by Leonard Rice Engineers, Inc. in 1979 and constructed in 1980; it was raised in 1998 and again in 2009. CTL/Thompson, Inc. performed a geotechnical investigation and evaluation of the levee from 1997 to 1999 and concluded that the levee met FEMA geotechnical requirements for certification in 1998 and 2009; CTL/Thompson, Inc. also provided testing and observation of the installation and compaction of engineered fill when the levee was raised in 1998 and 2009 (Leonard Rice, 2009). The levee was certified by FEMA in 2000 and recertified after the raise in 2009.

8.3 Alluvium

The natural alluvial valley is bounded on the east and west sides by elevated surfaces of Pierre Shale. We interpret that alluvium historically extended throughout much of the CU Boulder South campus. However, much of the alluvium on CU Boulder South campus has been removed and replaced with fill; therefore, the current alluvial aquifer is constricted around the east side of CU Boulder South campus.

Alluvium was encountered below portions of fill or topsoil west of the levee and at the ground surface in areas east of the levee. Alluvium ranged in thickness from 1.0 to 22.0 feet and was underlain by Pierre Shale bedrock. Alluvium predominantly consisted of a variety of coarse-grained material; however, some fine-grained alluvial layers were encountered near the south end of the Site. In several of the borings, cobbles and/or boulders were encountered at or near the ground surface or while drilling. The amount of cobbles and boulders identified in test pits represent up to about 30 to 60 percent of the volume. Samples collected in the test pits better represent the coarser material, which was generally a gravel with silt or clay, sand, cobbles, and boulders. The alluvium appears to be a deposit of heterogeneous particles with minor amounts of silt or clay. Coarser or finer layers, either vertically or laterally, were not identified. The shear wave velocity of the alluvium ranged from as low as 800 to 1,500 feet per second (fps).

About 56 percent of the SPT samples encountered refusal (50 blows for less than 6 inches). Uncorrected SPT N-values ranged from 2 to 73 and averaged 32. The alluvium ranged from dry to moist above the groundwater table and moist to wet below the groundwater table. The density ranged from very loose to very dense. Eight samples were nonplastic. Eleven samples had liquid limits that ranged from 20 to 62 and plasticity indices that ranged from 2 to 38, with averages of 29 and 10, respectively. The maximum particle size was recovered in the test pits and was 25 inches. Horizontal hydraulic conductivity measured in 41 rising head and constant head tests ranged from 5.6×10^{-5} to 3.1×10^{-2} cm/s, and the geometric mean was 4.8×10^{-4} cm/s.

8.4 Pierre Shale

Bedrock of the Pierre Shale formation was encountered below alluvium and fill at depths that ranged from 0.4 to 32.7 feet below the ground surface. Pierre Shale bedrock encountered near the connection of the dam embankment, and spillway alignments ranged from about 18 to 21 feet below the ground surface, which is deeper than along the alignments of other Project components. Depth to bedrock was shallowest near the embankment left abutment and within the environmental reclamation area, where bedrock was commonly less than about 5 feet below the ground surface).

Pierre Shale is generally a low-permeability clayey shale composed mostly of low to medium plasticity fines and is mostly soft to very soft. Infrequent beds of very hard sandstone were also encountered in borings. Bedrock is generally horizontally bedded and is predominantly unfractured. Generally, throughout the site, Pierre Shale is fresh to slightly weathered. The interpreted top of weathered bedrock had a shear wave velocity of approximately 1,100 to 1,500 fps, and the shear wave velocity increased to 4,000 fps within the depths explored.

About 33 percent of the SPT samples encountered refusal (50 blows for less than 6 inches). Uncorrected SPT N-values ranged from 14 to 72 and averaged 38. Recovered samples of Pierre Shale were mostly dry to wet. Liquid limits ranged from 29 to 46 and averaged 37. Plasticity indices ranged from 9 to 28 and averaged 19. Packer test results ranged from 0.1 to 82 Lugeons (1.0×10^{-7} to 8.1×10^{-4} cm/s), and the geometric mean was 0.1 Lugeons (1.9×10^{-7} cm/s). The unconfined compressive strength of 18 rock core samples ranged from 71 to 2,152 pounds per square inch (psi) and averaged 575 psi. The unconfined compressive strength of tested samples is generally higher along US36, which averaged 774 psi, and is generally lower toward the west side of the Project site (i.e., CU Boulder South campus), which averaged 317 psi.

8.5 Groundwater

During the geotechnical investigation, groundwater was encountered at depths of about 2.0 to 28.0 feet below the ground surface; however, most test pits were terminated before encountering groundwater. Groundwater was observed in alluvial or fill material, and bedrock; and the phreatic surface exists within fill and alluvium. The elevation of groundwater generally declined to the north, which generally follows the slope of topography and flow of SBC.

Monitoring wells in the fill have varied responses to seasonal groundwater fluctuations and precipitation, likely because of local heterogeneities within the fill. Groundwater levels measured in the monitoring wells in fill vary up to about 8 feet seasonally. Groundwater levels from the monitoring wells in alluvium vary from about 4 to 8 feet seasonally and generally respond to precipitation trends and irrigation activity on OSMP fields south and north of US36.

SECTION 9 - HYDRAULIC MODELING

9.1 General

The existing regulatory floodplain model (i.e., Effective Model) along SBC consists of a combination one-and two-dimensional hydraulic model that was developed using the MIKE FLOOD software program. The Effective Model for SBC through the City is from the Flood Mapping Study as documented in the *South Boulder Creek Climatology/Hydrology Report* (HDR 2007). This model was adopted by FEMA as the Effective Model in 2008. Digital copies of the Effective Model were obtained by DHI from the MHFD in October 2017.

A CLOMR was prepared by Plenary Roads and Michael Baker Jr., Inc. to document changes in the SBC floodplain resulting from the US36 widening project. Typically, a CLOMR is performed using the same modeling approach and software as the effective regulatory study. However, modeling for this CLOMR was performed using a one-dimensional Hydrologic Engineering Center River Analysis System (HEC-RAS) model instead of the Effective MIKE FLOOD model. The change in modeling approach and software was discussed and approved by the City, Boulder County, MHFD, and FEMA. The CLOMR model included widening US36, widening the US36 bridge over SBC, and adding a dual barrel wildlife crossing culvert below US36. The HEC-RAS model was subsequently updated following construction, and a Letter of Map Revision (LOMR) was issued by FEMA in 2017.

Project facilities will alter the SBC floodplain both at the Project site and downstream of the Project site. Floodplain mapping changes are anticipated to include removing large portions of the West Valley from the regulatory floodplain and minor floodplain changes along the main stem of SBC. Prior to construction of the Project, a CLOMR will need to be obtained from FEMA, documenting changes to the floodplain mapping. Development of the CLOMR will require development of the following hydraulic models:

- Duplicate Effective Model: This model is a copy of the Effective Model that is rerun on the requester's equipment to ensure it has been correctly transferred.
- Corrected Effective Model: This model corrects any errors in the Duplicate Model, updates the model to the latest version of the software, and incorporates more detailed or updated topography and LOMRs.
- Proposed Conditions Model: This model is modified to reflect the post-project conditions.

For this 60-percent design submittal, the RJH Team developed a preliminary Corrected Effective Model and a Preliminary Proposed Conditions Model, which are described below.

9.2 Corrected Effective Model

DHI developed a Corrected Effective Model that was used as the baseline for comparisons with the Proposed Conditions model. The Effective Model obtained from the MHFD was in the Version 2009 SP1 of the MIKE FLOOD software modeling package. DHI upgraded the models from Version 2009 SP1 to Version 2017 SP1 to incorporate recent software updates.

Both the 100-year and 500-year design flood events in the Effective Model are generated by a short-duration, high-intensity thunderstorm (i.e., the 100-year Thunderstorm and 500-year Thunderstorm). Initial model simulations for the 100-year General Storm performed by the RJH Team during Concept Design resulted in lower flood inundation extents and depths than the 100-year Thunderstorm. Based on this evaluation, it was concluded that the Thunderstorm is the governing design storm for flood extents and depth relative to these two events. Therefore, the General Storm was not used in development of the Preliminary Corrected Effective Model.

The Effective Model was modified to develop the Corrected Effective Model by:

- Updating bathymetric and channel topography using LiDAR data from the post-flood 2013 survey.
- Incorporating US36 embankment and bridge modification geometry and the dual wildlife crossing culverts from CDOT’s US36 expansion project.
- Updating topographic data at the Project site based on 2017 and 2021 survey data.
- Updating SBC channel topography based on in-stream channel construction survey as-built drawings.
- Correcting incomplete and/or incorrect culvert information from the Effective Model.
- Identifying and resolving culvert issues in the modeling approach used in the Effective Model regarding 1D-2D bypass at hydraulic structures.
- Updating the Manning’s n roughness coefficient in Viele Channel to reflect current conditions.

A comparison of Effective Model and Corrected Effective Model results indicate similar overall characteristics for the 100-year and 500-year events despite significant changes to bathymetry and cross sections and updates to hydraulic controls throughout the domain. However, there are significant differences (i.e., greater than one foot in depth) in specific locations. There are also areas along the fringe of the floodplain that are removed from the floodplain for both the 100-year and 500-year events. These differences are likely the result of the higher resolution topography. A plan of differences in flood depths between the Effective Model and Corrected Effective Model for the 100-year event is presented on Figure 9.1. Areas shown in green are areas that were part of the 100-year floodplain in the Effective Model but are removed for the Preliminary Corrected Effective Model.

Additional information for the Corrected Effective Model is presented in the *Draft South Boulder Creek- MIKE FLOOD Corrected Effective Model Development Report* (DHI, 2023).

9.3 Proposed Conditions Model

DHI developed a Proposed Conditions Model by modifying the Corrected Effective Model. Revisions generally included modifying the topographic terrain to reflect the 60-percent design including the dam embankment, detention excavation, spillway, levee removal, and outlet works below US36. Additional information regarding the proposed conditions model is presented in Appendix B.1.

A plan of differences in 100-year flood depths between the Corrected Effective Model and the Proposed Conditions Model is presented on Figure 9.1. Based on the modeling results, the 60-percent design is effective in meeting hydraulic design criteria for the Project. A summary of key modeling results for the Proposed Conditions Model for the 100-year event is as follows:

- US36 will not be overtopped.
- Significant portions of the West Valley neighborhood will not be inundated during a 100-year event on SBC.
- Flood elevations and extents along the main stem of SBC will be comparable or less compared to existing conditions.
- The 100-year peak flow rate through the bridge for the proposed conditions will be about 60 cfs less than the existing conditions.

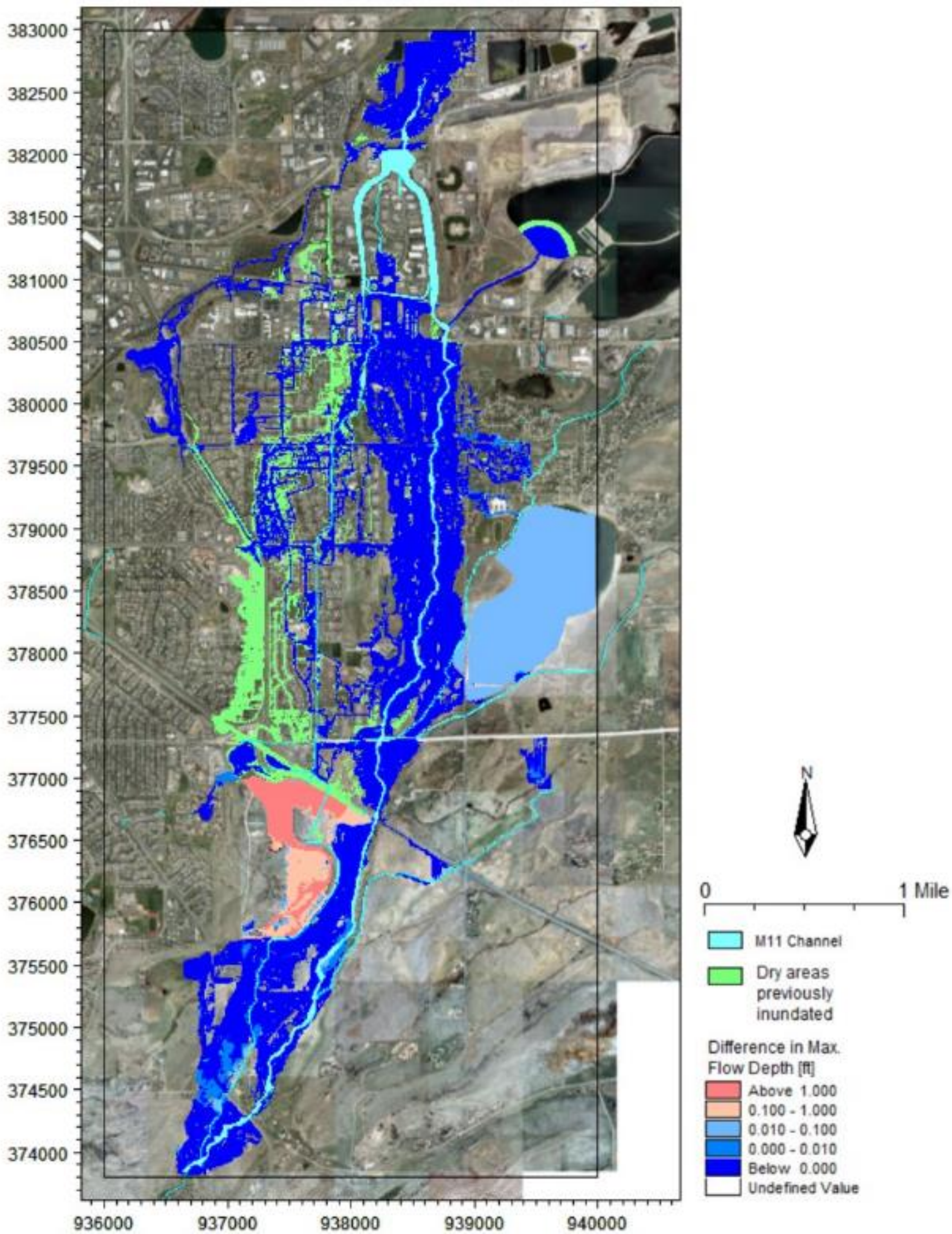


Figure 9.1: Difference in 100-year Flood Depth between Corrected Effective Model and Proposed Conditions Model

9.4 SRH-2D Model

A Sedimentation and River Hydraulics – Two-Dimension (SRH-2D) model was developed by Muller to assess hydraulic conditions through and near the US36 bridge and to confirm that the MIKE FLOOD model is appropriately modeling hydraulic conditions at the US36 bridge. Development of the SRH-2D model was requested by CDOT because it is their preferred model for evaluating bridge hydraulics. The SRH-2D model was developed using topography, Manning's n roughness coefficients, and other pertinent hydraulic parameters from the MIKE FLOOD model. This will allow for a direct comparison between models.

The SRH-2D model extends from about 1,000 feet upstream, 900 feet downstream, and 1,000 feet west of the US36 bridge at SBC. These modeling extents contain the US36 bridge and wildlife crossing. These model extents were selected because boundary conditions are sufficiently far from the US36 bridge to not impact hydraulics through the bridge. Modeling was performed for existing and proposed conditions for the 100-year and 500-year flood events.

Based on the modeling results, the 100-year event proposed conditions peak flow through the US36 bridge will be about 120 cfs less than the existing conditions peak flow through the bridge. Also, 100-year velocities and shear stresses through the bridge will be comparable or slightly less in the proposed conditions compared to the existing conditions. However, there are localized areas immediately downstream of the bridge that show an increase in velocity (5.7 ft/sec existing conditions to 8.9 ft/sec proposed conditions) and shear stress (0.8 pounds per square foot (psf) existing conditions to 1.7 psf proposed conditions) during the 100-year event. Localized increases in velocity and shear stress immediately downstream of the bridge appear to be the result of different flow interactions and flow patterns at the bridge in the proposed conditions. The impact of the velocity and shear stress increases on scour potential in a 100-year event has not been evaluated.

Based on the modeling results, the 500-year event proposed conditions peak flow through the US36 bridge would be about 200 cfs higher than the existing conditions peak flow through the bridge. Also, the 500-year results show an increase in velocity (16.8 ft/sec existing conditions to 17.0 ft/sec proposed conditions) and shear stress (8.8 psf existing conditions to 9.1 psf proposed conditions) through the bridge during the 500-year event. The impact of the velocity and shear stress increases on scour potential in a 500-year event has not been evaluated.

Additional information for the SRH-2D modeling evaluation is provided in Appendix B.

SECTION 10 - DAM EMBANKMENT

10.1 General

The dam embankment will consist of a zoned earthfill embankment with internal filters and drains with a barrier wall through the foundation soils extending into the underlying Pierre Shale bedrock. The dam embankment will extend along the north and west portion of the CU Boulder South campus. The dam embankment will be approximately 3,000 feet long and will connect to natural high ground, which is Pierre Shale bedrock, at the southwest (left) end and to the spillway at the east (right) end.

The dam embankment will consist of a central core (i.e., Zone 1), and upstream and downstream shells (i.e., Zone 2). The central core (Zone 1) will be 10 feet wide and will have sufficiently low permeability to safely detain the IDF event. The upstream and downstream shells (Zone 2) will consist of fine-grained to coarse-grained materials. The dam will have a crest width of 17 feet, which was selected based on SEO criteria and to provide a sufficient travel corridor for maintenance vehicles. The upstream and downstream slopes will be at 4 horizontal to 1 vertical (H:V) to reduce long-term maintenance and provide improved aesthetics. The embankment crest will be at elevation (El.) 5372.4. This provides 1 foot of freeboard above the routed IDF water surface elevation (WSE), which is greater than the wave runup.

Internal filter (i.e., Zone 3) and drainage zones (i.e., Zone 4) will be included within the embankment to safely manage seepage through the embankment fill. The seepage management collection system will include a 4-foot-wide Zone 3 chimney filter adjacent to the downstream edge of the Zone 1 core and horizontal finger drains that will convey seepage from the chimney to the downstream toe drain. The finger drains will be 3 feet thick and 10 feet wide and consist of 1 foot of Zone 4 material surrounded by 1 foot of Zone 3 material. The filter and drainage zones will consist of specially graded sand and gravel.

The embankment will include a toe drain system to collect and manage seepage that is collected by the embankment filter/drainage zones and to control groundwater levels downstream of the dam. The toe drain system will collect and convey flows using 8-inch diameter slotted polyvinyl chloride (PVC) pipes with periodic manhole cleanouts along the alignment. The embankment filter/drain is not anticipated to regularly convey water because the detention facility will usually be empty. However, based on proposed conditions groundwater modeling (see Section 12), the embankment toe drain could potentially collect groundwater along the west edge of the Project site (between the embankment and Tantra

Drive) during seasonally high groundwater periods. In this area, the embankment toe drain pipe will be installed near or slightly above the seasonally-high groundwater level to facilitate construction and prevent the routine collection of groundwater.

We designed the toe drain pipe to redistribute collected water into the subsurface. We anticipate that some of the collected water will re-infiltrate along the length of the slotted toe drain pipe as it flows through locations where the pipe is above the natural groundwater table. Also, exfiltration areas will be placed at the ends of the toe drain pipes to reintroduce collected water to the groundwater system. Weir boxes will be provided within vaults periodically along the toe drain alignment for flow monitoring.

A barrier wall will be used below the embankment dam to manage seepage through the foundation soils when the reservoir is storing water and will consist of a 3-foot-wide soil-bentonite barrier wall below the Zone 1 core of the embankment that will extend into the underlying Pierre Shale bedrock.

Foundation soil consists of fill that was previously placed to reclaim the CU Boulder South campus after mining operations and alluvium near the right abutment. The left abutment will connect to Pierre Shale bedrock. The existing berm (west berm) on the west side of the CU Boulder South campus will be removed for construction of the embankment. This will involve excavating the existing earthen berm and reusing the material for earthfill.

10.2 Analyses

10.2.1 General

RJH performed geotechnical analyses to support design of the dam embankment. These analyses included an evaluation of wave runup and freeboard, material properties, slope stability, seepage, and seismic deformation which are described below.

10.2.2 Wave Runup, Spillway Routing, and Freeboard

Required freeboard was identified using guidance from the United States Bureau of Reclamation (Reclamation) Design Standards No. 13 (Reclamation, 2021) in accordance with the SEO Rules and Regulations (SEO, 2020a) and is based on wave runup. Freeboard was evaluated for the following conditions:

- Condition 1: IDF Pool (El. 5371.4) plus runup and setup from a wind velocity exceeded 10 percent of the time, which is 15 mile-per-hour (mph).

- Condition 2: 100-Year Flood Pool (El. 5363.8) plus runup and setup from a 100-mph wind velocity.
- Earthfill embankment at a 4H:1V slope covered with native grasses.

The wave runup analyses calculated a wave runup plus setup of a) 1.0 feet for the wind velocity exceeded 10-percent of the time and b) 2.6 feet for a 100-mph wind velocity. A summary of freeboard and dam crest elevation requirements is presented in Table 10.1. Based on this evaluation, RJH selected a dam crest of El. 5372.4. The wave runup and freeboard analysis is presented in Appendix C.

TABLE 10.1
FREEBOARD AND DAM CREST ELEVATIONS

Reservoir Pool	Reservoir Pool Elevation (ft)	Minimum Freeboard (ft)	Wave Runup plus Setup (ft)	Minimum Dam Crest Elevation (ft)
Condition 1	5371.4	1.0	1.0	5372.4
Condition 2	5363.8	--	2.6	5367.8

10.2.3 Material Properties

The dam embankment core (Zone 1) will be comprised of onsite fine grained borrow material, and the embankment shell (Zone 2) will consist of material that could range from fine grained to coarse grained material sourced from onsite excavations or imported as necessary. Filter (Zone 3) and drain material (Zone 4) were considered to be imported, but possibly could be processed from on-site alluvium. Zones 3 and 4 are expected to be imported from a commercial source and were combined into a homogeneous filter zone for analyses. Foundation materials beneath the embankment are fill (mostly clayey sand), alluvium (generally sand and gravel), and Pierre Shale bedrock. The foundation soil was modeled as one unit and was based on a range of material properties that are considered to be conservative for both the fill and alluvium for the types of analyses being performed. The barrier wall will consist of soil-bentonite.

A summary of the material properties used for seepage and stability modeling is presented in Appendix C.4.

10.2.4 Seepage and Slope Stability Analyses

RJH performed two-dimensional seepage and slope stability analyses using the computer programs SEEP/W and SLOPE/W, which are part of the GeoStudio 2021 software package. Analyses were performed for a typical section of the embankment selected near the maximum embankment section and where Viele Channel is closest to the downstream toe. We considered the crest of the detention excavation upstream of the embankment to be 20 feet from the upstream toe of the embankment and at a 4H:1V slope to the bottom of the detention excavation.

Loading conditions and required safety factors are from USACE EM 1110-2-1902 (USACE, 2003b) in accordance with the SEO Rules and Regulations (SEO, 2020a). Analyses were performed for the following key loading conditions:

- Steady state conditions with an empty reservoir (seepage of groundwater into an empty detention excavation). This is the normal operating condition for the facility.
- Empty reservoir at the end of construction.
- Steady state conditions from a full reservoir (estimated 100-year flood water surface El. 5364).
- Rapid drawdown from a full reservoir to the bottom of the detention excavation.

Flood events will produce transient loads on the embankment and steady state conditions are not anticipated to occur under short-term reservoir loads. It is conservative to evaluate steady state conditions from a full reservoir. It is also conservative to evaluate rapid drawdown from a full reservoir because the analysis method assumes that steady state seepage conditions develop from the full reservoir prior to drawdown.

Seepage analyses were performed for the following foundation conditions and downstream hydraulic conditions:

- High-permeable foundation material properties to represent alluvial soil and typical groundwater conditions in Viele Channel for the empty reservoir condition and an empty Viele Channel for the full reservoir condition.
- Low-permeable foundation material properties to represent fill soil and typical groundwater conditions in Viele Channel for the empty reservoir condition and an empty Viele Channel for the full reservoir condition.
- High-permeable foundation material properties to represent alluvial soil and bank-full water conditions in Viele Channel.

- Low-permeable foundation material properties to represent fill soil and bank-full water conditions in Viele Channel.

For all analyzed conditions, the strength of the foundation material was based on lower-strength conditions of fill, which is conservative. Bank-full conditions in Viele Channel will maintain the phreatic surface higher, and be more conservative for stability analyses, than either the typical groundwater or empty Viele Channel condition. Results using materials properties that produced the most conservative conditions (i.e., highest phreatic surface) are presented in Table 10.2.

TABLE 10.2
SEEPAGE MODEL RESULTS

Loading Condition	Analysis Conditions	Exit Gradient(1)	Flow Rate(2) (gpm per foot)	Flow Rate into Toe Drain (gpm per foot)
Steady State - Empty Reservoir	Low-permeable foundation soils and bank-full conditions in Viele Channel	0.3(3)	$1.5x10^{-3}$	Not Applicable
Steady State - Full Reservoir	Low-permeable foundation soils and bank-full conditions in Viele Channel	<0.1(4)	0.04	$3.6x10^{-2}$

Notes:
1. Exit gradients are generally less applicable in fine grained materials.
2. The flow rate is calculated as all flow passing through a section that extends from the top of the embankment to the bottom of the bedrock in the model.
3. Exit gradient into the detention excavation.
4. Exit gradient 5 feet downstream of the embankment toe.

Stability analyses were performed based on the most conservative results of the seepage analyses for each loading condition. Stability results are presented in Table 10.3.

TABLE 10.3
SLOPE STABILITY MODEL RESULTS

Loading Condition	Computed Safety Factor		Required Minimum Safety Factor
	Upstream Slope	Downstream Slope	
Steady State - Empty Reservoir	2.2	2.3	1.5
End of Construction	1.5	1.6	1.3
Steady State - Full Reservoir	2.0	2.0	1.4
Rapid Drawdown	1.1	Not Evaluated	1.1

We conclude the following based on the model results:

- Acceptable seepage conditions will exist if steady state seepage occurred at the 100-year flood water surface elevation. The core, barrier wall, and toe drain effectively manage seepage and generally maintain the phreatic surface below the natural ground surface downstream of the dam (i.e., below the downstream shell).
- Seepage and stability conditions are predicted to be acceptable for both types of foundation soil (lower-permeable fill versus higher-permeable alluvium).
- Bank-full flood conditions in Viele Channel are not predicted to adversely affect seepage or stability performance of the dam. However, high water levels in Viele Channel could restrict the ability of the toe drain pipe to drain.
- Upstream and downstream slopes at 4H:1V are acceptable for all analyzed slope stability loading conditions.

A summary of the seepage and stability modeling is presented in Appendix C.4.

10.2.5 Seismic Deformation

We estimated seismic deformation using the Swaisgood procedure (Swaisgood, 2003), which is appropriate for non-liquefiable material. We expect the foundation soil to be non-liquefiable because the material is generally medium dense to dense and ranges from fine to coarse grained. The peak ground acceleration (PGA) was estimated for the design seismic event with a recurrence interval of 5,000 years (see Section 7), and the site adjusted PGA was 0.25g for very dense soil and soft rock. A conservative seismic hazard was evaluated using an earthquake magnitude of 6.0. The amount of settlement expected due to the design seismic event is about 0.2 inches. This amount of settlement is unlikely to result in a breach of the embankment and does not control the embankment design.

10.2.6 Camber

Design of embankment camber will be performed as part of the 90-percent Project design.

10.2.7 Upstream Slope Protection

The reservoir will typically be empty. In our opinion, erosion of the upstream slope from wave action is not anticipated to be a dam safety concern, and riprap or other hardened slope protection is not necessary. Based on previous discussion with the SEO, a permanent erosion control blanket will be installed along the upstream slope and will extend from the upstream toe to the embankment crest. The erosion control blanket will be buried, and the upstream slope will be vegetated with native grass.

10.2.8 Downstream Slope Protection

The alignment of the dam embankment has generally been located so that the downstream toe of the embankment is about 50 feet from the top of the right bank of Viele Channel. The embankment will need to be designed to safely withstand a PMF in Viele Channel. RJH performed a hydraulic evaluation to identify impacts to the embankment from flows in Viele Channel. Based on our analyses flows in Viele Channel during the PMF will not produce velocities and shear stresses that would cause erosion of the downstream slope. Therefore, downstream slope protection is not required. Additional information regarding the Viele Channel hydraulic analysis is discussed in Section 14.

In RJH's opinion, potential impacts to the dam embankment from an extreme flood in Viele Channel appear to be negligible, and a grass-covered slope should be adequate to maintain a stable embankment, and more robust erosion protection of the downstream slope is not required.

SECTION 11 - SPILLWAY

11.1 General

The spillway will consist of an above-ground concrete wall supported by secant piles that will provide structural support and below-ground seepage control that extends along the US36 corridor. The spillway will be approximately 2,650 feet long and will connect to the earthfill embankment at the west (left) end and to the US36 embankment at the east (right) end. A gravel access road will be located upstream of the spillway wall.

The alignment of the spillway was selected to avoid impacts to existing utilities within the CDOT ROW, facilitate construction access, and reduce permanent impacts to OSMP property to the greatest practicable extent. The location of utilities within the CDOT ROW varies along the US36 corridor; however, near the west end of the spillway, utilities are located near the southern edge of the CDOT ROW. The distance between the spillway wall and the CDOT ROW varies from about 16 to 50 feet. A gravel access road will be located upstream of the spillway wall.

11.2 Spillway Wall

11.2.1 Spillway Hydraulics

The spillway wall will consist of a vertical, reinforced concrete wall that varies in height above final grade from about 6 feet to 18 feet. For a majority of the spillway alignment, the top of wall will be set at El. 5364.8. This is one-foot above the 100-year water surface elevation. The 100-year water surface elevation was identified using the MIKE FLOOD model described in Section 9. The spillway wall will provide hydraulic control for flood events larger than the 100-year event. The spillway wall will be set at El. 5372.4 at the connection to the embankment dam (i.e., spillway left abutment) to prevent overtopping during the PMF and at El. 5365.8 at the connection to US36 (i.e., spillway right abutment) to reduce the frequency of overtopping during extreme events.

Reservoir and spillway routing for the PMF was performed for the spillway. A key part of performing spillway routing was identifying an appropriate weir coefficient. The hydraulic head over the spillway crest is relatively small (i.e., less than 10 feet) compared to the effective spillway length (i.e., 2,280 feet). As a result, spillway routing evaluations are especially sensitive to the selected spillway weir coefficient. RJH selected a weir coefficient of 3.2 based on the Kindsvater-Carter equation, which is commonly used to calculate weir coefficients for sharp-crested, rectangular weirs. In our opinion, this value is appropriately conservative for the purpose of dam safety permitting.

The spillway hydraulics are more complicated than a typical weir wall because of the following conditions:

- The existing ground generally slopes downward (east to west), and the height of the spillway wall generally increases from east to west. Flows will travel parallel to the spillway wall prior to overtopping the wall. Flows will overtop the wall non-uniformly.
- The area between the spillway wall and the US36 road embankment is a hydraulic constriction and will quickly fill with water once the spillway begins to discharge. This will create significant tailwater on the spillway apron.

As a result, RJH elected to use a two-dimensional, unsteady hydraulic model to model flow conditions at the spillway wall and the downstream channel. RJH developed a two-dimensional hydraulic model using HEC-RAS 6.3.1. RJH used the HEC-RAS model to route the PMF hydrograph through the spillway. The model predicted a peak discharge over the spillway of 71,870 cfs with a corresponding peak water surface elevation at El. 5371.4. A rating curve for the spillway is presented in Table 11.1. The rating curve was obtained from the HEC-RAS model and considers non-uniform overtopping of the spillway wall and tailwater in the downstream channel that eventually submerges the spillway.

TABLE 11.1
SPILLWAY RATING CURVE

Water Surface Elevation (ft)	Flow (cfs)
5364.8	0
5365.0	700
5366.0	4,440
5367.0	13,640
5368.0	23,020
5369.0	37,780
5370.0	50,540
5371.0	65,290
5371.4	71,870

Additional information regarding the spillway hydraulic evaluation is presented in Appendix D.1.

11.2.2 Spillway Wall Structural

RJH performed geostructural analyses to identify the required thickness of the wall. This evaluation was performed for the combined spillway wall and secant pile foundation system. RJH performed two-dimensional analyses using the DeepEX software program developed by Deep Excavation, LLC. Both 100-year and PMF hydraulic loads were evaluated. The model considered hydrostatic water conditions on each side of the wall (i.e., seepage beneath the secant pile wall was not evaluated). In addition, the model considered loads from earthfill placed along the downstream side of the wall for landscaping purposes (see Section 11.6). Based on the results of this model, a 1-foot-thick wall with appropriate steel reinforcement will generally be adequate for the spillway. If architectural treatments are approved for the wall, the thickness would be increased by the thickness of the treatments. For the 60-percent design, a reinforcement pattern for the spillway wall consisting of #7 bars each face and both ways at 12 inches was selected. The reinforcement pattern for the spillway foundation was modeled for every other secant pile with 11 #9 bars for vertical reinforcement and #5 hoops every 12 inches. Additional structural elements will likely be required at the base of the wall near the connection to the secant piles and pile cap. The reinforcement pattern and additional structural details will be developed in future stages of design. Additional information regarding the spillway geostructural evaluation is presented in Appendix D.2.

11.3 Spillway Foundation

11.3.1 Deep Foundation

Foundation conditions along the spillway consist of coarse-grained alluvium overlying Pierre Shale bedrock. Bedrock is expected to be about 21 feet below the ground surface near the west (left) end of the spillway and 8 feet below the ground surface near the east (right) end of the spillway. Foundation soils along the spillway contain cobbles and boulders, which will preclude installation of driven seepage control (e.g., sheet piles).

RJH initially considered multiple alternatives for a full cutoff for the deep foundation, including structural foundations (secant pile wall and diaphragm wall) and non-structural seepage barriers (sheet pile wall, soil-bentonite slurry wall, vibrating beam wall, soil-mixing, chemical/permeation grouting, jet grouting, and an earthen core trench). The secant pile wall was identified by RJH as the most desirable option based on technical and economic considerations.

The spillway foundation will consist of a secant pile wall that will extend through the alluvium and into bedrock. The purposes of the secant pile foundation are to provide structural support for the spillway wall. The secant pile foundation will also provide seepage management to restrict uncontrolled flow below the spillway through the coarse-grained alluvium during times of flood detention. Groundwater will be conveyed through secant pile foundation by the groundwater conveyance system in a controlled manner. A secant pile wall was selected because it can be installed in challenging subsurface conditions (i.e., cobbles and boulders) and provides more structural support compared to other types of cutoff walls (i.e., sheet pile, slurry wall, etc.). The secant pile wall will be constructed by drilling shafts and backfilling the shafts with reinforced concrete and non-reinforced concrete.

A reinforced concrete pile cap will be constructed at the top of the secant pile wall to transfer loads from the structural wall to the secant pile wall and to provide a level surface for installing forms for the structural wall.

RJH performed geostructural analyses to identify sizing, spacing, and embedment depth into bedrock for the secant pile foundation using the DeepEX model described in the previous section.

Based on this analysis, we concluded that the secant piles should extend about 8 feet below the top of the bedrock. Secant pile embedment should be measured from the top of competent bedrock that is generally moderately weathered to fresh and moderately fractured to unfractured. A secant pile diameter of 4 feet with center-to-center spacing of 7 feet will generally provide sufficient structural support for the spillway wall. To facilitate construction, every other pier will include reinforcing steel.

Additional information regarding the spillway foundation evaluation is presented in a technical memorandum in Appendix D.2.

11.3.2 Shallow Foundation

An alternative to the secant pile foundation (i.e., deep foundation) may be to construct a reinforced concrete spread footing (i.e., shallow foundation). The spread footing would be designed to provide sufficient structural support for the spillway wall; however, a seepage barrier to bedrock would not be included. A shallow foundation would be beneficial because it would allow groundwater to flow through the alluvium beneath the spillway during normal (non-flood) conditions. However, a shallow foundation would also allow seepage through the spillway foundation during flood loads, which would need to be safely managed.

RJH performed preliminary analyses to evaluate the feasibility of using a shallow foundation to support the spillway wall. We identified a) backward erosion piping and b) uplift of the spillway apron as being the two most credible seepage-related potential failure modes (PFM) for a spillway founded on a shallow foundation. We performed preliminary stability analyses to develop appropriate foundation geometries and then performed simplified seepage modeling to identify exit gradients, uplift pressures on the spillway apron, and flow rates that will need to be collected by a drainage system. Using results from the seepage modeling, we performed a simplified potential failure modes analysis to develop event trees and estimate the probability of failure for these two potential failure modes.

RJH identified that the shallow foundation alternative has the potential to reduce project costs by about $3 million but would increase the risk for several potential failure modes and may be challenging to permit. The new or increased risk failure modes include:

- <u>Differential settlement</u>: The spread footing would be inherently more susceptible to differential settlement than the secant pile wall because it would be founded on alluvial soils.
- <u>Uplift</u>: The spread footing would be more susceptible to uplift forces. Uplift forces would be caused by a defect in the barrier wall, excessive conveyance through the groundwater conveyance system, or by failure of the relief wells needed to manage uplift pressures.
- <u>Instability</u>: The spread footing would be more susceptible to instability if the spillway apron were to fail and there is excessive scour downstream of the spillway wall.

The unique configuration of the overtopping spillway with shallow foundation would present a permitting challenge. Overtopping dams/spillways founded on alluvial materials are very uncommon, and there is not established design criteria for this configuration. In addition, there does not appear to be a strong precedent for regulating this type of facility. The City and RJH eliminated this alternative because this concept would likely increase risk for catastrophic failure, may create a negative public perception, would likely be difficult to permit. In our opinion, the potential cost savings associated with the shallow foundation do not justify the disadvantages described above.

11.4 Spillway Apron

11.4.1 Hydraulic Evaluation

The spillway will discharge to the area between the spillway wall and the US36 roadway embankment. This area consists of OSMP property and the CDOT ROW and includes a regional multi-use trail. An energy dissipation facility is needed to reduce the likelihood of scour and erosion when the spillway is operating. The energy dissipation facility will consist of a reinforced concrete spillway apron immediately downstream of the spillway wall.

RJH performed hydraulic analyses to size the spillway apron. The spillway hydraulics are more complicated than a typical weir with an apron because:

- The existing ground generally slopes downward, and the height of the spillway wall generally increases from east to west (i.e., right to left). Flows will travel parallel to the spillway wall prior to overtopping the wall. Flows will overtop the wall non-uniformly. The spillway wall will initially be overtopped closest to SBC.
- The area between the spillway wall and the US36 road embankment is a hydraulic constriction and will quickly fill with water once the spillway wall begins to overtop. This will create significant tailwater on the spillway apron.

RJH performed a review of technical papers related to drop-spillway energy dissipation. The unique hydraulic conditions at the spillway do not facilitate the direct use of standard engineering reference documents to size the energy dissipation facilities. Most standard references for spillway and weir hydraulics were developed for shorter drop spillways, assumed uniform weir overtopping, and for an unsubmerged weir, and do not account for energy dissipation from high tailwater values.

We identified a technical report by the Reclamation, Technical Report (TR) REC-ERC-74-9 *Hydraulic Model Studies of Plunge Basins for Jet Flow* (TR 74-9) (Reclamation, 1974), that evaluated the influence of tailwater on energy dissipation from jet flow. This report focused on jet flow from a gate valve rather than an overflow weir. The nappe from an overflow weir will perform differently than jet flow from a gate valve when subjected to significant tailwater. However, we did not identify any other studies that evaluated the influence of significant tailwater depths on energy dissipation of a jet. We selected to use this approach for 60-percent design of the spillway apron and have endeavored a conservative application of this approach.

RJH evaluated spillway apron requirements using two methods:

- Theoretical hydraulic equations to estimate the trajectory of the nappe.
- An empirical evaluation based on hydraulic laboratory testing from Reclamation for jet flow impacted by tailwater.

Based on this evaluation, we conservatively selected apron lengths that vary along the lengths of the spillway. The apron length is 9 feet at the east (right) end of the spillway and increases to 20 feet at the west (left) end of the spillway.

It is possible that a more detailed evaluation could result in a decrease to the size of the concrete apron. This will likely require developing a computation fluid dynamic model or performing a physical model study. Either of these could be performed in the final design if the City desires to evaluate decreasing the size of the apron. However, the benefit-cost of the construction cost savings or the more rigorous engineering analysis should be evaluated.

Additional information regarding the spillway apron hydraulic evaluation is presented in Appendix D.1.

11.4.2 Seepage Management System

RJH performed seepage modeling using the computer program SEEPW to evaluate the need for a seepage management system along the spillway apron to relieve potential uplift pressures. SEEPW was used for this evaluation because this program is state-of-practice for evaluating seepage stability near dams and other hydraulic structures. A two-dimensional model is appropriate for this task because seepage through the conveyance system will predominantly be two-dimensional and will be dominated by the hydraulic gradient across the spillway wall. The localized SEEPWeep/W model allows for appropriate spatial refinement (5-foot elements), is suited for the simulation of short-duration (hours to days) hydraulic stresses, and includes built-in tools to evaluate model results related to dam stability (exit gradients, uplift pressures, etc.). Analyses were performed for one typical section that is generally representative of site conditions near spillway Station 16+89. The modeled section consisted of alluvium over Pierre Shale bedrock. US36 roadway fill was also included in the model downstream of the spillway; however, we do not expect the fill to significantly affect the model because the fill exists above the groundwater table. Material properties assigned to these materials were consistent with those used for MODFLOW groundwater modeling (RJH, 2021).

The groundwater conveyance system was simulated as aggregate-filled trenches on both sides of the secant pile wall and a pipe through the secant pile wall that connects the two trenches. Material properties assigned to these zones were high enough to not restrict flow through the system during normal (non-flood) conditions. The secant pile was excluded from the model because we consider it to be impermeable.

The model was used to simulate a 100-year flood. This flood was selected because the ponded (detained) water will reach the top of the spillway wall without flowing over the spillway, which represents the highest differential (critical) hydraulic load between the upstream and downstream sides of the spillway. The simulation consisted of initial steady state boundary conditions to represent normal initial conditions followed by a transient boundary condition to represent passage of the 100-year flood.

We considered normal groundwater conditions during the irrigation season (July). This condition was conservatively selected for analysis because groundwater levels will be higher during the irrigation season and, therefore would be more susceptible to produce seepage-related instability during a flood load. The initial boundary conditions were constant heads of El. 5360, about 1,200 feet upstream of the spillway, and El. 5341, about 1,200 feet downstream of the spillway. These boundary conditions produced an initial groundwater level and gradient near the spillway wall that were generally similar to existing conditions and those predicted by the MODFLOW model.

We then introduced transient boundary conditions into the model to simulate the 100-year flood. A constant head boundary condition (head versus time function) was assigned to the ground surface upstream of the spillway wall to simulate the changing flood waters. A constant head boundary condition (head versus time function) was also assigned downstream of US36 to represent the predicted 100-year flood stage in this area. The area beneath the apron slab was modeled as a no-flow boundary condition to represent impervious concrete. The area between the spillway apron and US36 was modeled as a seepage face boundary condition. This type of boundary condition allows water to exit the model as seepage if the calculated total head exceeds the elevation of the boundary.

Based on the model results, uplift pressures beneath the apron slab will be up to about 168 psf, and exit gradients at the end of the apron slab will be about 0.5. These pressures and exit gradients are higher than recommended values. Therefore, a seepage management system was added under the apron slab. Providing a seepage management system below a concrete apron downstream of a spillway is a common design feature for a dam. This system will consist of a 1-foot-thick filter/drain blanket below the concrete apron that connects to a slotted drain pipe at the downstream of the end of the apron. Solid pipes will be placed to convey any collected seepage to the drainage swale along US 36.

Additional information regarding the spillway apron hydraulic evaluation is presented in Appendix D.2.

11.5 Abutment Connection to US36

The spillway alignment at the right abutment will bend and extend perpendicular to US36. This section of the spillway will be set at El. 5365.8 (1 foot higher than the majority of the spillway) to reduce the frequency of overtopping during extreme events. The spillway will terminate in the US36 roadway embankment. The spillway wall and secant pile foundation will extend to the point where the top of the spillway wall is below the existing US36 embankment. A vertical soil-bentonite drilled shaft will be constructed at the edge of the spillway wall and secant pile foundation to reduce the likelihood of a seepage path forming along the connection.

The multi-use trail will extend over the east (right) abutment of the spillway. An earthfill ramp will be placed along both sides of the spillway wall at this location to accommodate the multi-use trail. Additional information regarding the multi-use trail is presented in Section 15.

The right abutment of the spillway will be higher in elevation than the spillway control section; therefore, the spillway abutment is not predicted to be overtopped during the design flood event (100-year event). However, the spillway abutment and US36 roadway are predicted to be overtopped during less frequent events. It is important that the stability of the spillway abutment is maintained during extreme flood events to protect against an uncontrolled release of the detained floodwaters and to meet SEO requirements.

RJH identified and evaluated four PFMs that could occur during extreme loading events and potentially compromise the spillway abutment:

- PFM #1: Abutment Breach from Spillway Flows. This failure mode will be caused by flows that overtop the spillway as intended and subsequently also overtop US36. These flows could cause erosion of the US36 roadway fill, and the abutment stability might be compromised if the erosion encroached too near the connection between the spillway and abutment.
- PFM #2: Abutment Breach from Abutment Overtopping. This failure mode will be caused by extreme flood events that overtop the right abutment of the spillway. These flows could erode soil from the abutment, which might result in a breach of the abutment if the erosion was severe enough.
- PFM #3: Abutment Breach from South Boulder Creek Flows. This failure mode will be caused by water that is retained upstream of the spillway and flows downstream through South Boulder Creek beneath the US36 bridge. These flows could cause erosion of the US36 roadway fill and a breach of the spillway abutment if flow conditions in this area were highly erosive.

- PFM #4: Seepage Instability of Abutment. This failure mode will be caused by seepage through the abutment (beyond the edge of the spillway) that develops during detention of floodwaters. This seepage could adversely affect the abutment if excessive seepage forces or uplift pressures develop downstream of the spillway.

RJH developed a simplified two-dimensional hydraulic modeling using the USACE HEC-RAS 6.3.1 software program to identify hydraulic loading conditions at select locations for each of the PFMs. PFMs #1, #2, and #3 were evaluated by developing an embankment erosion model using the Natural Resources Conservation Service WinDAM software program. PFM #4 was evaluated by developing a seepage model using the GeoStudio 2021 Seep/W software program.

Based on the evaluation performed by RJH, the four PFMs evaluated in this stage of design are not predicted to adversely affect the stability of the spillway abutment. Additional information regarding the abutment stability evaluation is presented in a technical memorandum in Appendix D.4.

11.6 Spillway Landscaping Architecture

The downstream face of the spillway wall will be visible from US36. The upstream face of the spillway wall will be visible from OSMP property. The City desires to provide aesthetic treatments for both sides of the wall and landscaping along the downstream corridor. Architerra developed initial concepts for aesthetic treatments for the concrete wall and surrounding landscape. Exposed faces of both the upstream and downstream spillway wall would include concrete stamping and possibly concrete coloring. Concrete stamping concepts could include artistic (i.e., animals, birds, trees, etc.), architectural (i.e., geometric), or natural (i.e., stone-like) patterns. Concrete colors and stamping patterns will be identified in the next stage of design.

Architerra developed two concepts for landscaping along the downstream spillway corridor. Both concepts include placing earthfill along the downstream face of the spillway wall to assist with concealing portions of the wall and limit opportunities for vandalism. The height of the earthfill will vary to provide visual variety.

- Landscape Concept 1: This concept involves placing earthfill extending from the spillway wall into the US36 ROW. The extents of earthfill within the CDOT ROW would be limited to maintain the existing drainage ditch. With this concept, the existing multi-use trail could be relocated closer to the spillway wall. This would enhance user experience on the trail by increasing the distance from vehicle noise on US36 and allow shrubs to be installed between the trail and US36.

- Landscape Concept 2: This concept involves placing earthfill extending from the spillway wall to the US36 ROW. Earthfill would not be placed within the CDOT ROW. This concept would not accommodate relocation of the existing multi-use trail.

Conceptual renderings for both landscape concepts are presented in Appendix D.5. For both concepts, the earthfill would be considered "sacrificial" (i.e., not required for the wall to be stable) and could potentially erode when the spillway is flowing. Also, the placement of the earthfill would not reduce the hydraulic capacity of the spillway. Additional erodibility evaluations will be performed in the next stage of design if this concept is advanced.

SECTION 12 - GROUNDWATER CONVEYANCE SYSTEM

12.1 General

The following Project components are anticipated to impact the natural flow of groundwater at the site:

- Barrier wall below the embankment dam.
- Barrier wall around the detention excavation.
- Secant pile wall below the spillway.

Groundwater conveyance systems will be included to mitigate the impacts from these project facilities and generally maintain groundwater levels and flow patterns that are similar to the existing (i.e., pre-construction) conditions. Groundwater conveyance systems will be installed at two locations: along the spillway and along the toe of the embankment dam at the west side of the site. The systems will be designed to operate passively (i.e., via gravity) without the need for routine operator intervention or pumping. However, some operator intervention will initially be required upon system commissioning to adjust gates and fine-tune system performance.

12.2 Spillway Groundwater Conveyance System

The purpose of the spillway groundwater conveyance system is to convey groundwater past the spillway alignment and mitigate impacts from the secant pile foundation. The system is designed to provide higher hydraulic capacity than the current hydraulic capacity of the aquifer so that the groundwater levels upstream and downstream of the spillway will naturally balance, and the groundwater system will generally continue to function consistent with historic conditions.

The spillway groundwater conveyance system will include the following key components:

- Collection trench on the upstream (south) side of the spillway and adjacent to the northeastern portion of the barrier wall. The purpose of this trench is to collect groundwater upstream of the secant pile wall and prevent the groundwater level upstream of the spillway from rising higher than its historic natural level. The collection trench will generally be located 11 feet upstream of the spillway and will consist of a 4-foot wide and 8-foot-deep trench with a 10-inch slotted PVC pipe surrounded by filter material. The filter material will be similar to ASTM C33 No. 8 coarse aggregate, which is filter compatible with the surrounding alluvium and is expected to have a hydraulic conductivity that is about 100 to 1,000 times greater than the alluvium. Pipe slot patterns will be designed during the next phase of design. Pipe slots will be compatible with the surrounding filter material and will have hydraulic capacity that exceeds the filter material. The invert of the pipe was set to be about 2 feet below the seasonally low groundwater levels so that the historic seasonally low groundwater levels can be maintained. The top of the trench will coincide with the surface of the spillway's temporary working platform, which will be about 5 feet above the seasonally low groundwater level.

- Distribution trench on the downstream (north) side of the spillway and downstream (northeast) side of the embankment dam. The purpose of this trench is to redistribute collected groundwater downstream of the secant pile wall and prevent the groundwater level downstream of the spillway from declining below its historic natural level. The distribution trench will generally be located 11 feet downstream of the spillway and will be configured similarly as described above for the collection trench.
- Connector pipes to convey water from the collection trench to the distribution trench. These pipes will be solid 10-inch PVC pipes 22 feet long and connect the collection trench pipe to the distribution trench pipe. The connector pipes will penetrate through the secant pile wall, and a low-permeable seal will be used to reduce seepage through these wall penetrations. An estimated nine connector pipes will be spaced at approximately 250 feet along the distribution and collection trench alignments. The collection pipes, distribution pipes, and connector pipes will have a hydraulic capacity that is orders of magnitude higher than the collection and distribution trenches; therefore, the proposed configuration of connector pipes is appropriate for conveying flows through the secant pile wall and maintaining similar groundwater levels on both sides of the wall.
- Trench Backfill Plugs. Both the upstream collection trench and the downstream distribution trench will include segments where a solid 10-inch PVC pipe is installed instead of a slotted pipe, and the trench is filled with low-permeable backfill (plugs) instead of filter material. These plugs are anticipated to be about 25 feet long and will be located at each manhole along the collection and distribution trenches. The purpose of the backfill plugs is to promote groundwater flow across the spillway alignment (i.e., through the connector pipes) instead of flowing along the length of the collection or distribution trenches.
- Manholes will be installed about every 250 feet along the collection trench and distribution trench at the location of each connector pipe. These manholes will provide access to the collection trench pipes and distribution trench pipes for inspection and maintenance of the system.

- Gates. Regulating gates will be installed in each manhole at both the upstream and downstream ends of the collection and distribution trench pipes. Similar to the backfill plugs, the purpose of these gates is to promote groundwater flow through the connector pipes instead of along the collection trench pipe. These gates will allow various segments of the groundwater conveyance system to be adjusted individually to accommodate potential local variations in alluvial properties or other characteristics of the hydrogeologic system.
- Monitoring wells. Additional monitoring wells will be installed upstream and downstream of the spillway alignment to record pre- and post-Project groundwater levels. The gates will be adjusted so the conveyance system generally mimics the existing groundwater system. We anticipate that some initial gate adjustments will be required to calibrate system performance immediately after construction. Locations of additional monitoring wells will be identified in next phase of design.

12.3 Dam Embankment Groundwater Conveyance System

The dam embankment groundwater conveyance system will be the toe drain for the embankment dam. The toe drain will mitigate any rises in the groundwater elevations that will be caused by the barrier wall below the embankment dam. The toe drain will be installed near or slightly above the seasonally high groundwater table along the downstream side of the dam. Groundwater levels that rise above this historic level will be collected by the toe drain, and this water will be redistributed downstream of the embankment dam when it flows along segments of the toe drain pipe that are above the natural groundwater table. Additional information about the performance of this system is presented in the following section and in Section 10.1.

12.4 Groundwater Conveyance System Discharges

Groundwater and seepage collected in the downstream embankment toe drains will be distributed into the groundwater in a similar manner as the spillway groundwater conveyance system.

We anticipate that a Subterranean Dewatering Permit (General Permit Number COG603000) will be required from the Colorado Department of Public Health and Environment if the embankment toe drain pipe was to collect groundwater and discharge the water onto the ground surface or a surface water body. Requirements of this permit include monitoring of daily flow rates and water chemistry testing to demonstrate that the collected groundwater does not exceed the water quality standard for the receiving surface water body.

Because of these permitting requirements, in our opinion, it is not desirable to discharge collected groundwater onto the ground surface. Instead, we designed the toe drain pipe to redistribute collected groundwater within the subsurface. We anticipate that some of the collected groundwater will re-infiltrate along the length of the slotted toe drain pipe as it flows through locations where the pipe is above the natural groundwater table. Exfiltration areas will also be provided downstream of the embankment, where collected water can be reintroduced to the groundwater system. Weir boxes will be provided within vaults periodically along the toe drain alignment for flow monitoring. To provide redundancy, flows collected by the toe drain pipe that do not re-infiltrate within the drain system will be discharged into Viele Channel; however, in our opinion, surface water discharges will be highly unlikely.

12.5 Groundwater Modeling

12.5.1 Baseline Groundwater Modeling

RJH developed a Baseline Model to support the design of the groundwater conveyance system. The objective of the baseline groundwater modeling was to develop a model that (a) reasonably approximated the existing groundwater conditions near the site, (b) could be used to assess impacts to the natural groundwater conditions from proposed Project components, and (c) could be used to support the design of facilities to mitigate those impacts.

RJH developed a conceptual model of the hydrogeologic system based on subsurface information obtained during our Phase I Geotechnical Investigation (RJH, 2019) and used MODFLOW-USG to develop a numerical Baseline Model of the existing hydrogeologic system near the Project Site. The numerical model was calibrated to Site conditions measured in 2018-2019, and the unweighted scaled RMS error of the steady-state and transient model components were 1.2 and 1.1 percent, respectively, which are well below the acceptable value of about 5 percent (MDBC, 2001). We concluded that the Baseline Model provided a reasonable approximation of the existing groundwater system in the Project vicinity and was suitable for evaluating impacts of Project components and supporting the design of mitigation features.

Additional information about the Baseline Model is presented in the Baseline Groundwater Model Report (RJH, 2021).

12.5.2 Design Modeling

12.5.2.1 Model Evolution

Groundwater modeling performed to support design is described in the following sections, and additional information is provided in Appendix E.

During preliminary design, RJH modified the Baseline Model by decreasing the hydraulic conductivity of the bedrock based on additional data collected during the Phase II geotechnical investigation. This modification had negligible effects on model performance. The modified model that was used to simulate the natural hydrogeologic system during preliminary design is referred to as the preliminary design Pre-Project Model and was presented as part of the RJH, *Preliminary Design Report* (RJH, 2022c).

RJH made the following additional modifications to the preliminary design Pre-Project Model based on updated Site information:

- The modeled ground surface was updated based on new topographic data obtained by Merrick in 2023.
- The modeled bedrock surface was updated based on additional geotechnical data collected during the Phase II and Phase III geotechnical investigations.
- The SFR boundary condition (boundary condition used to represent surface water flow in South Boulder Creek) was updated based on new topographic data.
- Drain boundary conditions (boundary condition used to simulate areas where groundwater can exit the model as seepage into a ditch) were updated based on new topographic data.
- The foundation material at the north end of the CU levee was modeled as alluvium instead of fill to more closely match the expected subsurface conditions east of the existing maintenance building.

The model that includes these additional modifications is referred to as the 2018-2019 60% Pre-Project (PP) model (i.e., 60-percent design Pre-Project model with 2018-2019 hydrologic inputs).

RJH used the 2018-2019 60% PP model to simulate existing Site conditions from the 2018-2019 irrigation year and to affirm that model calibration remained satisfactory after making the changes described above. As described in Appendix E, the unweighted scaled RMS error of the steady state and transient model components were 1.3 and 1.2 percent, respectively, which are well below the acceptable value of about 5 percent (MDBC, 2001) and were very similar to what was originally achieved by the Baseline Model (RJH, 2021). Therefore, we concluded that the 2018-2019 60% PP model was a reasonable benchmark to simulate the natural hydrogeologic system in the Site vicinity, to evaluate the effects of proposed Project components on groundwater conditions, and to support design of Project facilities to mitigate impacts from project components.

12.5.2.2 Model Confirmation

RJH modified the 2018-2019 60% PP model to simulate conditions observed during a more recent (2020-2021) irrigation year. The purpose of this evaluation was to confirm that model performance remains acceptable when using more recent irrigation year data. The following changes were made to the 2018-2019 60% PP model to simulate the 2020-2021 season:

- Upstream and downstream constant head boundary conditions were updated to match observed far-field levels.
- SFR boundary condition was updated to match observed South Boulder Creek flows.
- Monthly background recharge (precipitation) was updated to match observed weather data.
- Irrigation recharge was updated based on ditch diversion records. The monthly irrigation rates on each irrigation zone were updated proportionately based on differences between the 2018-2019 diversion and the 2020-2021 diversion. The spatial irrigation patterns from the Baseline Model (RJH, 2021) were not changed.
- Evapotransporation was updated to match observed data published online by Northern Water.

The simulation of this new irrigation year is referred to as the 2020-2021 60% PP model (i.e., 60-percent design Pre-Project model with 2020-2021 hydrologic inputs).

The groundwater heads simulated by the 2020-2021 60% PP model were compared against the water levels measured during 2020-2021 in RJH's monitoring wells. As shown in Appendix E, the unweighted scaled RMS error of the steady-state and transient model components were 1.2 and 1.3 percent, respectively, which are well below the acceptable value of about 5 percent (MDBC, 2001) and were similar to those achieved by the original Baseline Model (RJH, 2021). Therefore, we concluded that the developed pre-Project model performed well for a different hydrogeologic year from which it was calibrated.

RJH then modified the 2018-2019 60% PP model to simulate conditions observed during a recorded period of drought (2001-2002) in the vicinity of the Project. The purpose of this simulation was to provide a benchmark for evaluating the effects of proposed Project components on groundwater levels during local drought conditions. Modifications to model inputs were similar in nature to those described above for the 2020-2021 60% PP model. The simulation of this observed drought year is referred to as the 2001-2002 60% PP model (i.e., 60-percent design Pre-Project model with 2001-2002 hydrologic inputs).

Simulated heads for the 2001-2002 60% PP model showed similar patterns to the preliminary design Pre-Project Model (RJH, 2022c) but were slightly lower in elevation, which is expected during a drought season. There are no reliable head calibration data in the Project vicinity prior to winter 2018, and therefore, head calibration statistics were not computed for the 2001-2002 60% PP model.

12.5.2.3 Proposed Conditions Scenario

RJH added the following proposed facilities into the 60% PP models to simulate the effects of Project components:

- Updated the ground surface topography to simulate earthwork fill for the dam and South Loop Drive roadway, removal of the CU levee, detention excavation, and miscellaneous Site grading.
- Added HFB boundary conditions to simulate the low-permeable soil-bentonite wall around the detention excavation and beneath the embankment foundation, and the low-permeable secant pile wall within the spillway foundation.
- Added drain boundary condition along the interior of the detention excavation HFB to remove water from the model and simulate how seepage and precipitation runoff that enters the detention excavation can flow out through the uncontrolled outlet works.
- Added high-permeable cells to simulate the groundwater conveyance system (collector trench, distributor trench, and connector pipes) along, and adjacent to, the spillway alignment.
- Added localized cells near the junction of the connector pipes to the collector and distributor trenches. These localized cells represent gates within manholes.

The configuration of the groundwater conveyance system is consistent with the layout and elevations shown on the 60 percent drawings. The high-permeable cells were assigned to simulate a drainpipe invert installed 2 feet below the seasonally-low groundwater table observed to date near the spillway alignment.

The model with 2018-2019 hydrologic inputs that includes these proposed Project components is referred to as the 2018-2019 60% Proposed Conditions (PC) model.

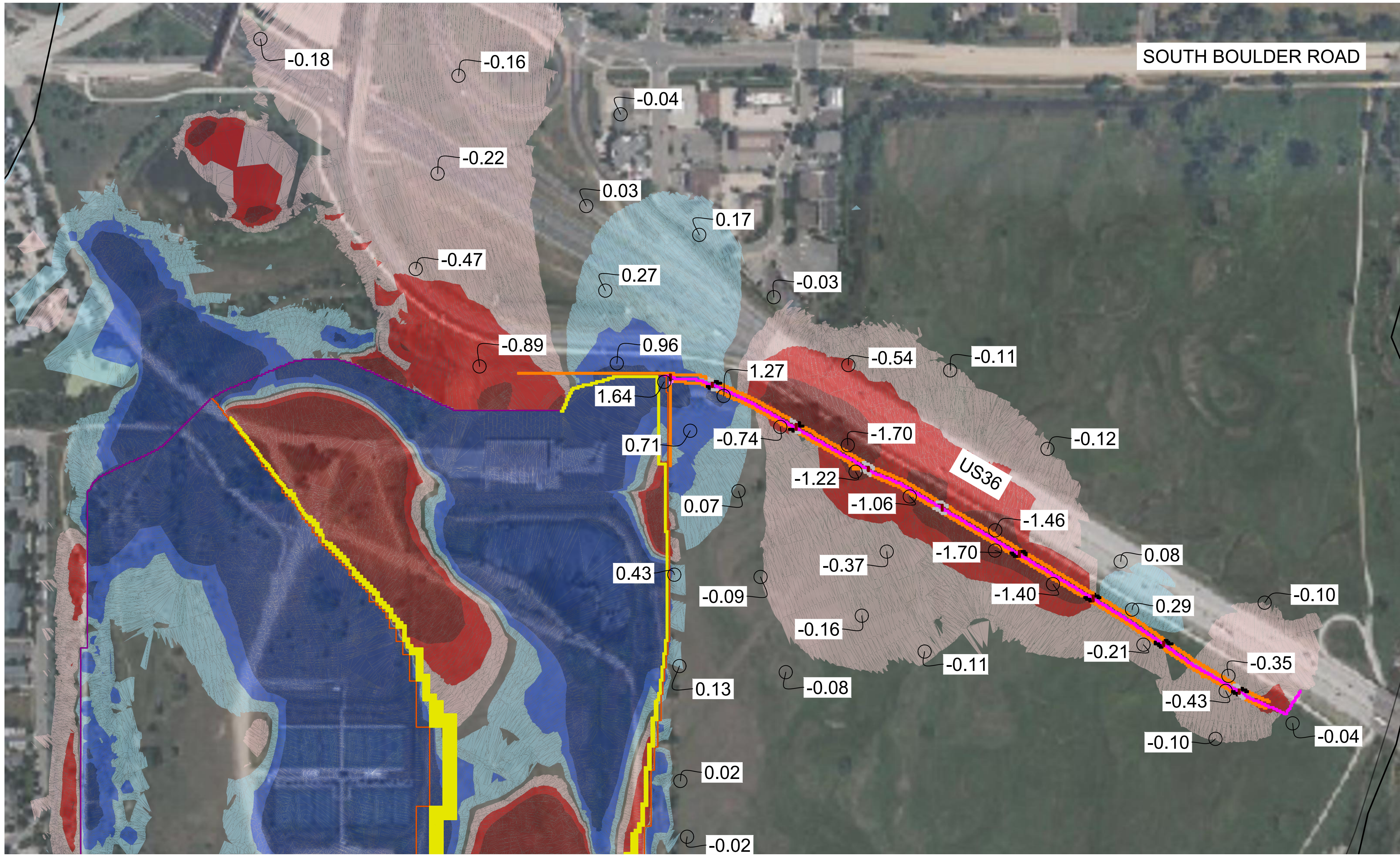
The extents of the modeled groundwater conveyance system trenches and the cell permeabilities used to represent gates were iteratively adjusted until the predicted heads were generally similar (within +/- 1 foot) to the groundwater levels predicted from the 2018-2019 60% PP model in the OSMP fields adjacent to US36. Modeled head-change results for the representative winter and summer months are shown on Figures 12.1 and 12.2, respectively. The iterative gate adjustment process demonstrated that some flow regulation will need to occur through gates in each manhole.

RJH then simulated the 2020-2021 irrigation year using the same gate permeabilities as those used to model the 2018- 2019 irrigation year. The purpose of this simulation was to verify that the proposed groundwater conveyance system would perform reasonably for a more recent irrigation year and to identify if the system would function effectively once the gates are adjusted, without further adjustment. This model is referred to as the 2020-2021 60% PC model. Modeled head-change results for the representative winter and summer months are presented in Appendix E and are also within about +/-1 foot of the groundwater levels predicted from the 2020-2021 60% PP model in the OSMP fields adjacent to US36.

We also simulated the 2001-2002 irrigation year using the same gate permeabilities as those used to model the 2018-2019 irrigation year. The purpose of this simulation was to verify that the proposed groundwater conveyance system would perform reasonably during drought conditions at the Site and to identify if the system would function effectively once the gates are adjusted, without further adjustment. This model is referred to as the 2001-2002 60% PC model. Modeled head-change results for the representative winter and summer months are presented in Appendix E and are also within about +/-1 foot of the groundwater levels predicted from the 2001-2002 60% PP model in the OSMP fields adjacent to US36.

Based on head-change results presented on Figure 12.1, Figure 12.2, and in Appendix E, we expect that additional seasonal adjustments to gate settings should not be needed after initial adjustments are made to achieve desired groundwater conveyance system performance.

Additional documentation for the proposed conditions groundwater models, including information on model setup and simulated flows, is presented in Appendix E.



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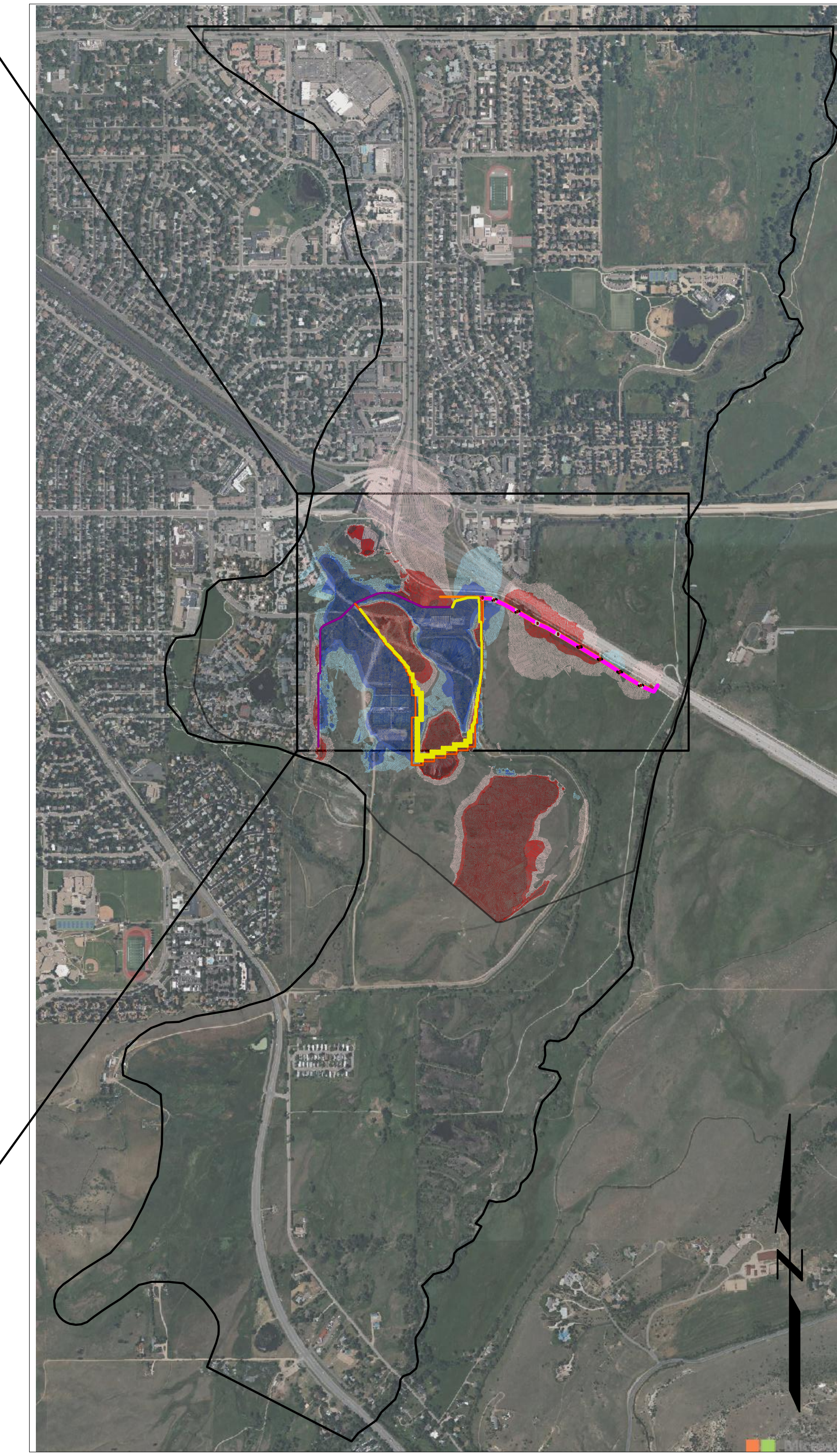
SCALE IN FEET



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SCALE IN FEET

ELEVATIONS TABLE

No.	Min. Elevation	Max. Elevation	Color
1	-1.00 OR LESS		
2	-1.00	-0.50	
3	-0.50	-0.10	
4	0.10	0.50	
5	0.50	1.00	
6	1.00 OR GREATER		

NOTES:
1. REPRODUCE IN COLOR.

PRELIMINARY
NOT FOR CONSTRUCTION

CITY OF BOULDER

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT

2018-2019 60% PC HEAD-CHANGE RESULTS - WINTER

PROJECT NO. 16134

February 2024

FIGURE 12.1

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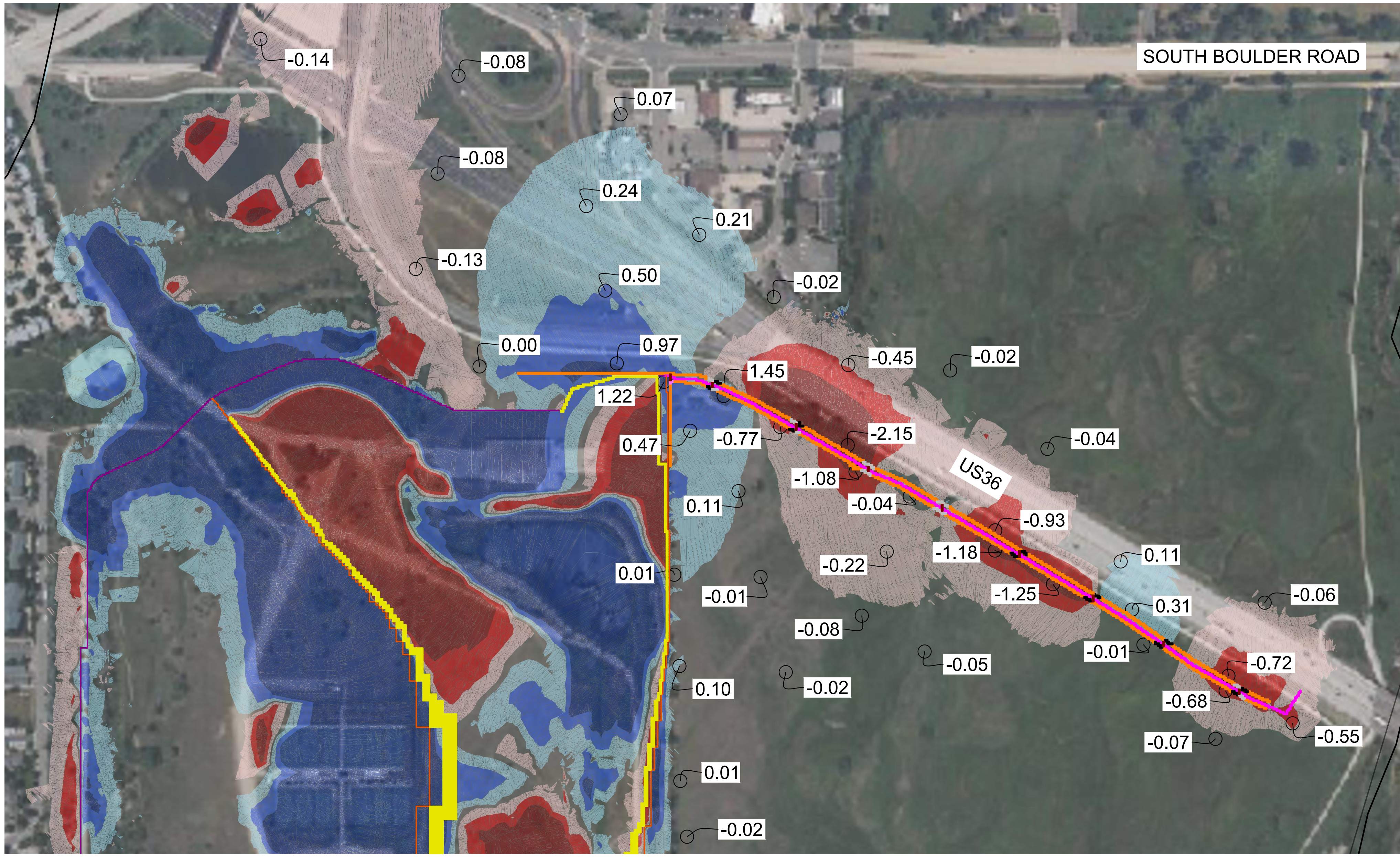

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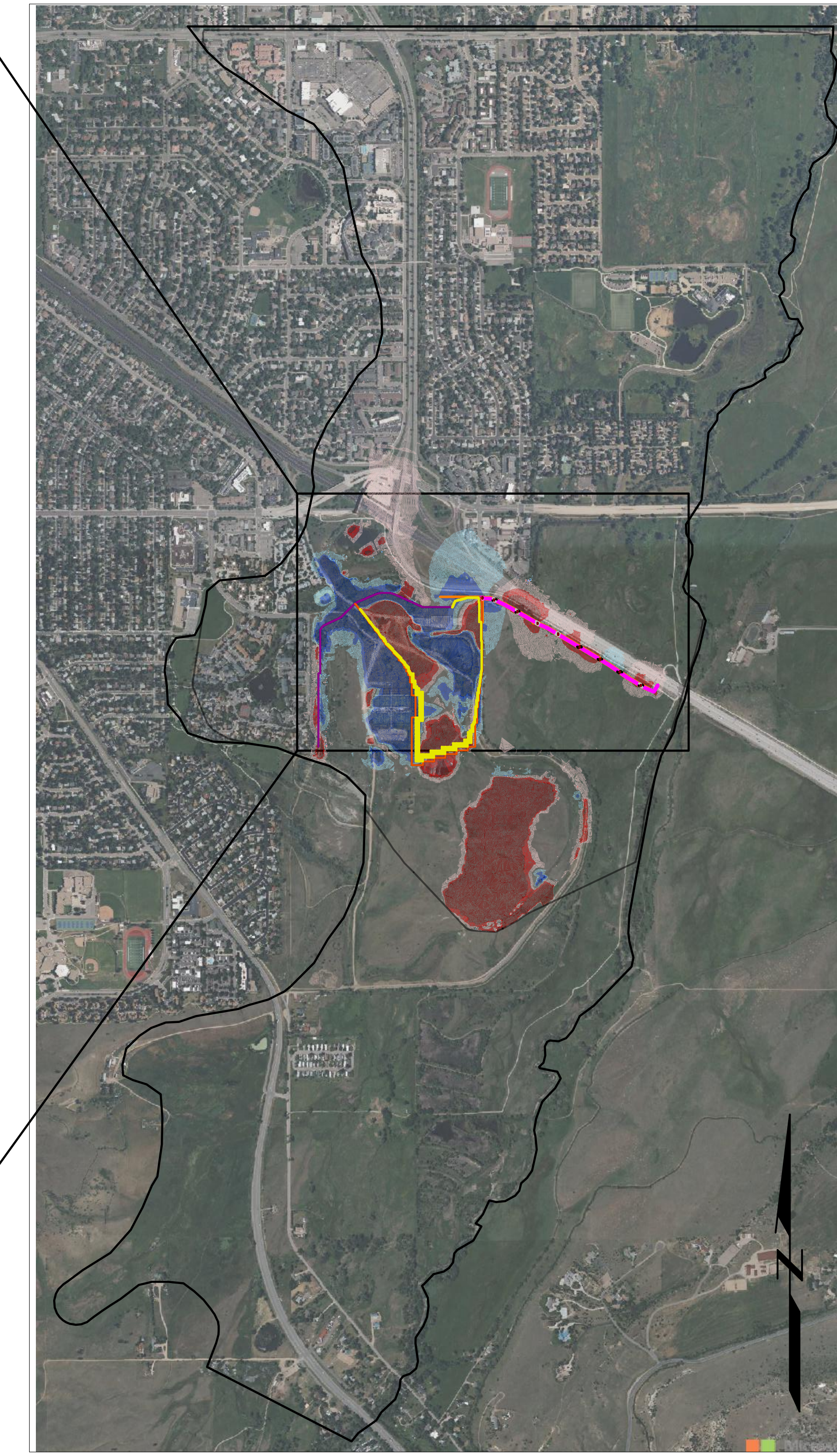

0 1000 2000 4000 6000

SCALE IN FEET

ELEVATIONS TABLE

No.	Min. Elevation	Max. Elevation	Color
1	-1.00 OR LESS		
2	-1.00	-0.50	
3	-0.50	-0.10	
4	0.10	0.50	
5	0.50	1.00	
6	1.00 OR GREATER		

NOTES:
1. REPRODUCE IN COLOR.

PRELIMINARY
NOT FOR CONSTRUCTION

CITY OF BOULDER

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION PROJECT

2018-2019 60% PC
HEAD-CHANGE RESULTS -
SUMMER

PROJECT NO. 16134

February 2024

FIGURE 12.2

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SECTION 13 - OUTLET WORKS

13.1 General

The outlet works will extend from the detention excavation south of US36 to Viele Channel north of US36. The outlet works will allow the lower portion of the reservoir pool to drain to meet SEO dam safety requirements and water rights requirements. The outlet works will include an inlet structure located at the toe of the detention excavation, an outlet structure located in Viele Channel, and an 875-foot-long conduit to convey flows between the inlet and outlet. The inlet of the conduit will be set at El. 5343.0, which is the bottom of the detention excavation. The outlet of the conduit will be set at El. 5339.5, which is the invert elevation of Viele Channel north of US36. The outlet works alignment was selected to reduce permanent and temporary disturbances on OSMP property, provide a perpendicular tunnel alignment to US36, and provide launching and receiving shafts and staging areas that avoid existing utilities to the greatest extent practicable. The outlet works conduit alignment includes a 90-degree bend and two 15-degree bends. Bends in outlet works conduits are not typical at high-hazard dams. However, bends in a conduit are not necessarily undesirable, but hydraulic impacts including head losses and thrust forces need to be considered in design.

13.2 Hydraulic Analyses

Hydraulic analyses for the outlet works included:

- Performing hydraulic routing to size the outlet works.
- Developing a theoretical rating curve for a full range of reservoir levels to identify velocities and pressures for steel pipe thickness sizing and trash rack sizing.
- Identifying hydraulic conditions at the outlet and sized the outlet structure.

The outlet works conduit needs to have sufficient hydraulic capacity to meet SEO and State water rights discharge requirements and prevent overtopping of US36. However, if the hydraulic capacity is too large, the existing 100-year floodplain downstream of the site will be negatively impacted. Because of the dynamic nature of outlet works discharges for this Project, outlet works routing was performed using the MIKE FLOOD software program (see Section 9). Based on the results of the modeling, the outlet works will consist of a single 60-inch diameter pipe with intake and outlet structures. This configuration will result in a peak discharge of 333 cfs through the outlet works during the 100-year flood event, which will not negatively impact the existing 100-year floodplain downstream of the site. The entirety of the reservoir will drain in approximately 20 hours, which meets both SEO and water rights criteria.

RJH developed an outlet works rating curve for a full range of reservoir levels using traditional weir and orifice equations and energy conservation analyses. For the purpose of sizing the steel pipe thickness and trash racks, the theoretical rating curve was conservatively developed, assuming no existing tailwater from runoff from the Viele Channel drainage basin. The theoretical rating curve is presented in Table 13.1.

TABLE 13.1
THEORETICAL OUTLET WORKS RATING CURVE

Water Surface Elevation (ft)	Flow (cfs)	Notes
5343.0	0	
5344.0	8	
5346.0	57	
5348.0	127	
5350.0	188	
5352.0	234	
5354.0	272	
5356.0	298	
5358.0	315	
5360.0	330	
5362.0	345	
5363.8	358	100-year WSE(1)
5364.8	365	Spillway crest

Note:
1. From the DHI MIKE FLOOD proposed conditions model.

The maximum theoretical flow through the outlet works pipe is about 365 cfs when the reservoir pool at the spillway crest, El. 5364.8 (i.e., design event). This corresponds to a velocity of about 19 fps in the 60-inch-diameter discharge pipe. Based on discussions with pipe manufacturers, polyurethane lining in a steel pipe can withstand velocities up to about 40 fps for infrequent periods.

RJH also considered hydraulic conditions through the outlet works during the Probable Maximum Flood (PMF). Based on hydraulic modeling performed by RJH, the PMF would have a maximum water surface elevation in the reservoir of El. 5372.4. The PMF would overtop the spillway and US36 and result in considerable tailwater at the outlet works outfall. The differential head between the reservoir pool and tailwater is greater during the design event than during the PMF so that the design event will control for design.

Additional information regarding the outlet works hydraulic evaluation is presented in Appendix F.1.

13.3 Intake Structure

The intake structure will consist of a 13.5-foot-high, reinforced concrete sloping riser structure. The intake structure will be located near the upstream toe of the detention excavation in the northeast corner of the detention excavation. The front and top of the structure will include openings covered by trashracks. The intake structure will have interior dimensions of 7 feet by 31.5-feet, which were selected to provide sufficient access for maintenance and to provide sufficient open area to meet SEO and MHFD trashrack velocity requirements, which limit velocities to 5 fps (50 percent clogged) and 2 fps, respectively. The trashracks along the front of the structure will be sloping to match the surrounding grade. The trashrack along the top of the structure will have a pentagonal prism shape. This may provide redundancy for trashrack clogging since it is possible that the different trashrack shapes will have different clogging mechanisms. Theoretically, the trashrack could be about 80 percent clogged before it will become a hydraulic restriction and impact drain time of the reservoir.

13.4 Conduit

The outlet works conduit will consist of an 880-foot long 60-inch diameter steel pipe. The pipe will be installed using a combination of open excavation and tunneling techniques. The portion of the conduit upstream of US36 conduit (620 linear feet) will be constructed in an open excavation and will be backfilled with low-strength concrete for corrosion protection. The portion of the conduit underneath US36 (260 linear feet) will be installed by tunneling. The tunneled portion of the pipe is currently envisioned to consist of a 60-inch diameter steel carrier pipe within a 96-inch diameter steel casing pipe. The size of the casing pipe was selected to facilitate advancement past potentially large boulders and to provide flexibility for installing the carrier pipe at the desired grade. The tunnel alignment will be oriented perpendicular to US36, which is typically preferred by CDOT. The annulus between the carrier pipe and casing pipe will be grouted.

The tunnel will require an approximately 20-foot by 40-foot launch shaft and a 20-foot by 20-foot receiving shaft. The tunnel will be installed from upstream to downstream to reduce the construction impacts north of US36 (size of work area, duration that work is being performed, number of traffic/deliveries, etc.). The launch shaft will be constructed in the CDOT ROW. The receiving shaft will be constructed on private property to the north of US36 and DCD2. The receiving shaft will also be used to provide a supported excavation for construction of the outlet structure.

The tunnel will extend below DCD2 north of US36. The vertical distance between the top of the tunnel and bottom of the existing ditch will be about 1 foot. The tunnel would either need to be installed during non-irrigation season when the ditch is not active, or a temporary bypass pipeline would need to be installed to safely convey ditch flows over the tunnel crossing during construction.

Ground conditions along the outlet works profile are very challenging for tunneling because of shallow groundwater and potential obstructions (cobbles and boulders) within the alluvium. These challenging ground conditions are further complicated by the presence of US36 over the tunnel alignment (i.e. lack of construction access and high consequences from overexcavation and settlement) and the desire to maintain a specified grade so the outlet works can drain the detention excavation via gravity. Lithos performed a tunnel feasibility evaluation based on the geotechnical data available during Preliminary Design (RJH, 2022) and did not identify any fatal flaws that would preclude construction of a tunnel. Lithos evaluated the feasibility, advantages, and disadvantages of several tunneling methods and identified pipe ramming as the preferred tunneling method with the least overall project risk.

A new boring (B-327) was completed near the launch shaft after completion of the Preliminary Design Report and identified that bedrock is higher in elevation than previously anticipated and is within the limits of the tunnel profile. It is therefore reasonable to consider that some portions of the tunnel alignment will encounter mixed face conditions (i.e., a combination of alluvium and bedrock). Mixed face conditions are adverse to conventional tunneling techniques, practically preclude the use of the preferred method (pipe ramming), and create major construction risk.

Two additional borings (B-328 and B-329) are proposed early in the 90-percent design phase to better define subsurface conditions near the midpoint and downstream end of the tunnel alignment. After this additional information is collected, workshops between the City, RJH, and Lithos are planned to discuss potential tunneling approaches, the associated risks of each, and the preferred approach to advance into final design. Potential alternatives include:

- <u>Lowering the tunnel alignment</u>: This alternative would consist of lowering the tunnel profile to be completely within bedrock beneath US36. Construction costs would decrease and a wider variety of construction techniques would become viable. This approach would greatly improve the chances of success and would practically eliminate risks associated with obstructions and overexcavation/settlement within the alluvium. A major disadvantage of this approach is that it is expected to collect sediment and require routine maintenance to maintain pipe capacity.

- Use GEONEX excavation method with multiple carrier pipes: GEONEX is a horizontal hammer system that is capable of excavating the anticipated ground conditions within alluvium with minor risk of overexcavation or failure. However, this equipment has a maximum size of 4 feet. This method could use a combined 48-inch diameter thick wall steel casing pipe/carrier pipe or could use separate casing and carrier pipes, which would result in a 36-inch carrier pipe. This would require multiple pipes to achieve the required hydraulic capacity. Guidance of this excavation method is also difficult, and deviation of about 1 foot could occur at the receiving end. Limited room also exists at the downstream end for discharging multiple conduits into Viele Channel.

- Dewatering the alignment to allow for open-face tunnel methods: This approach would allow for maintaining the desired alignment and removing boulder obstructions. However, dewatering will be very challenging in the highly permeable alluvium with limited surface access to the tunnel alignment. Changes to the groundwater elevations from dewatering would extend significantly beyond the tunnel alignment. Some overexcavation and settlement/sinkholes could occur because of incomplete dewatering. Construction costs will be relatively high.

- Upsize the tunnel to allow for different tunneling approaches: Earth Pressure Balance Machine (EPBM) is one example of a tunneling technique that can handle mixed face conditions and large boulders. The minimum required tunnel diameter would be about 15 feet, which would significantly increase construction cost. The top of the EPBM tunnel would need to be near the top of the casing pipe to maintain appropriate cover between the tunnel and US36. The bottom of the tunnel would extend into bedrock. This would also require a robust system to install and maintain the grade of the carrier pipe when the annulus is grouted.

- Attempt the tunnel as currently designed using pipe ramming: This approach would involve major construction risk and the City would likely need to remove risks from the contractor for tunneling contractors to be willing to bid the work. The City would likely need to assume financial and schedule risks for dewatering interventions for obstructions, possible closure of US36 for emergency tunnel face access, and grouting to alleviate settlement/sinkholes.

13.4 Outlet Structure and Discharge Channel

Energy dissipation at the outlet works outfall will be influenced by tailwater in Viele Channel. Significant tailwater is anticipated in Viele Channel when the outlet works is flowing near design capacity. Design procedures for commonly used energy dissipation structures at dams, such as impact basins and baffled chutes, do not account for significant tailwater.

RJH developed a two-dimensional hydraulic model of the outlet works outfall area using HEC-RAS 6.3.1. A terrain model was developed with data from the 2023 topographic survey. The model was conservatively developed, assuming no existing tailwater in the channel from runoff in the Viele Channel drainage basin.

RJH modeled several different outlet structure configurations. The preferred configuration consists of a 17-foot-wide, 24-foot-long horizontal concrete apron with vertical training walls. The concrete apron will transition to a 17-foot-wide, trapezoidal discharge channel with 3H:1V side slopes. The discharge channel will subsequently flow into Viele Channel. Velocities in the concrete apron will not exceed 10 feet per second, which is acceptable for concrete. The discharge channel and adjacent portion of Viele Channel will be lined with riprap. Velocities in the riprap area will not exceed about 8 feet per second. Additional information regarding the outlet structure hydraulic evaluation is presented in Appendix F.1.

SECTION 14 - SITE DRAINAGE

14.1 General

The Project facilities will impact Viele Channel, DCD2, US36 wildlife crossing, and site drainage under US36. A discussion of impacts to site drainage and potential solutions is presented below.

14.2 Viele Channel

14.2.1 Impacts on Dam Embankment

Viele Channel extends through the northwest portion of the CU Boulder South campus. The alignment of the embankment dam has generally been located so that the downstream toe of the embankment is about 50 feet from the top of the right bank of Viele Channel. RJH performed a hydrologic and hydraulic evaluation to identify the impacts of flooding in Viele Channel on the dam embankment.

The Viele Channel watershed at the Project site is approximately 1.2 square miles. The watershed extends southwest of the Project site through multiple residential neighborhoods and into Shanahan Hill. Viele Lake is located in approximately the center of the watershed. Viele Lake is formed by a low-hazard, jurisdictional dam. Viele Lake Dam consists of an approximately 24-foot-high embankment dam with an approximately 90-foot-wide excavated earthen spillway through the right abutment.

RJH performed hydrologic analyses to identify peak flow rates in Viele Channel at the Project site during the PMF. The hydrologic analyses were performed in accordance with the SEO's *Guidance for Hydrologic Modeling and Flood Analysis* (SEO, 2022) and the SEO Rules and Regulations (SEO, 2020a). Viele Lake and spillway do not have sufficient hydraulic capacity to rout the PMF, and the dam will breach during the PMF. A breach of Viele Lake was included in the hydrologic analysis. Based on the analysis performed by RJH, the controlling PMF event in Viele Channel at the Project site is the 2-hour Local storm. The peak flow rate for this event is 6,280 cfs.

Additional information regarding the Viele Channel hydrologic evaluation is presented in a technical memorandum in Appendix G.1.

RJH performed hydraulic analyses of the PMF in Viele Channel at the Project site. The embankment will need to be designed to safely withstand a PMF in Viele Channel. The segment of Viele Channel adjacent to the dam embankment varies and consists of a combination of the following: open channels, a detention pond, and culverts under roadway crossings. The channel and culverts are not sized for an extreme flood event like the PMF and will overtop. A portion of the overtopping flows will discharge onto the downstream slope of the dam embankment.

The flow regime beyond the main channel of Viele Channel will consist of shallow overland flow. RJH developed a two-dimensional hydraulic model using HEC-RAS 6.3.1. An inflow hydrograph was used for the boundary condition at the upstream end of the model and consisted of the Viele Channel PMF hydrograph developed by RJH with a peak flow rate of 6,280 cfs.

Based on the results of the HEC-RAS model, velocities along a majority of the downstream slope of the embankment will be less than 0.5 feet per second during the PMF. These velocities will not be expected to cause erosion of grass-covered earthfill materials. There is an approximately 150-foot-long segment of the downstream slope where the velocities will be between about 4 to 6 fps. The flow depths in this area will be less than 2 feet. These velocities will likely not cause erosion of grass-covered earthfill materials if the grass cover was moderately dense. If grass cover is not dense, then minor erosion will be expected. We do not anticipate that minor erosion in this area will be a dam safety risk.

In RJH's opinion, potential impacts to the dam embankment from an extreme flood in Viele Channel appear to be negligible, and a grass-covered slope should be adequate to maintain a stable embankment, and more robust erosion protection of the downstream slope is not required.

Additional information regarding the Viele Channel hydraulic evaluation is presented in a technical memorandum in Appendix G.2.

14.2.2 South Loop Drive Crossing

Viele Channel currently extends below South Loop Drive through dual 8-foot-wide by 6-foot-high corrugated metal pipe (CMP) culverts. About 15-feet of earthfill will be placed above these culverts to ramp South Loop Drive over the dam embankment. CMPs are prone to corrosion and have a shorter design life than other types of culvert materials. We are unsure when the CMPs were installed or if they have significant corrosion damage. Also, we are unsure if the existing CMPs have sufficient structural strength to withstand additional earth loads. For these reasons, the existing CMPs will be demolished and replaced with a single 12-foot-wide by 6-foot-high reinforced concrete box culvert. The culvert dimensions were selected to provide similar hydraulic capacity to the existing CMP culverts.

14.3 Dry Creek Ditch No. 2

DCD2 is owned and maintained by the DCD2 Company. Flows in the ditch are diverted from SBC approximately 1.8 miles upstream of the Project site. DCD2 consists of an earthen ditch from the point of diversion through OSMP property to the Project site. Multiple turnout structures are located along this segment of the ditch to facilitate flood irrigation of OSMP property south of US36. The capacity of the ditch varies significantly by location. The approximate ditch capacity directly upstream of US36 is approximately 22 cfs.

DCD2 extends under US36 through a 6-foot by 4-foot RCBC. The culvert discharges into a 6-foot-wide by 3-foot-high rectangular concrete-lined channel downstream of US36. The concrete lined channel dimensions transition to a 5.25-foot-wide by 2-foot-high concrete-lined channel approximately 85 feet downstream of the culvert. The concrete lined channel transitions to an approximately 7-foot-wide earthen ditch 375 feet downstream of the culvert outlet.

The RCBC below US36 and the downstream concrete channel were installed in 2016 by the Colorado Department of Transportation as part of the US36 widening project. Based on information presented in the CDOT as-built drawings, the culvert and channel were designed for a peak flow of 123 cfs. The 123 cfs design flow was considered the sum of the DCD2 deeded flow (69 cfs) and the local 100-year stormwater runoff (54 cfs).

It appears that the 100-year stormwater runoff identified by CDOT only considered runoff from the contributing drainage basin directly upstream of the culvert. However, 100-year flows in SBC overtop the left bank of SBC, and a portion of these flows will be conveyed through the DCD2 culvert. Based on Corrected Effective modeling of the SBC floodplain performed by DHI, the peak 100-year flow through the existing DCD2 culvert would be about 290 cfs during a 100-year event on SBC.

The spillway alignment intersects DCD2 approximately 44 feet upstream of US36. The DCD2 culvert will be extended to the upstream face of the spillway wall to maintain flows in DCD2 below US36. The culvert extension will consist of a 4.5-foot (rise) by 4-foot (span) RCBC. The culvert extension was sized to provide sufficient hydraulic restriction to limit flows to less than 290 cfs with an increased hydraulic head during the Project design event while not restricting deeded and CDOT design flows.

Hydraulic modeling of the culvert system was performed using the U.S. Army Corps of Engineers HEC-RAS Version 6.3.1 hydraulic modeling program. Steady-state, one-dimensional models were developed to simulate the following conditions:

- Existing conditions: The existing conditions model simulates hydraulic conditions through the existing 6-foot by 4-foot DCD2 culvert.
- Proposed conditions: The proposed conditions model simulates hydraulic conditions through the DCD2 culvert with a restricted culvert extension at the inlet.

RJH evaluated a range of different sizes for the culvert extension. Based on the results of the hydraulic models, a 4.5-foot-wide by 4-foot-high RCBC will meet the Project design criteria as summarized below:

- The DCD2 deeded flow (69 cfs) can be conveyed through the culvert system under open channel conditions, and water surface elevations upstream of US36 would not be raised compared to existing conditions.
- The CDOT design flow rate (123.1 cfs) can be conveyed through the culvert system under open channel conditions, and incremental increases in water surface elevations upstream of US 36 would be less than 6 inches.
- The proposed culvert extension would convey approximately 280 cfs when the water surface elevation upstream of US36 is at the spillway crest (El. 5364.8), which is less than the existing 100-year flood peak flow rate of 291 cfs from the Corrected Effective Model.

A concrete apron will be installed on the upstream side of the culvert extension to transition the grade from the bottom of DCD2 to the culvert extension invert. Modifications to DCD2 facilities north (i.e., downstream) of US 36 will not be required because the Project is not increasing the peak 100-year flow through the culvert system. Calculations for the DCD2 extension are provided in Appendix G.

14.4 Wildlife Crossing

A wildlife crossing extends under US36 approximately 400 feet west of SBC and consists of a dual 4-foot by 10-foot RCBC. The wildlife crossing was installed as part of the US36 widening project implemented by CDOT in 2016. An approximately 9-acre area south of US36 drains directly to the wildlife crossing. This area was previously drained to an adjacent culvert below US36 and to SBC prior to the installation of the wildlife crossing. The wildlife crossing will also discharge flows from SBC during a flood event.

The spillway alignment is located approximately 53 feet upstream of the face of the wildlife crossing. The wildlife crossing will be extended to the upstream face of the spillway wall to facilitate wildlife access. Based on hydraulic modeling performed by the Project team (see Section 9), the Project will decrease the 100-year flow through the wildlife crossing by 212 cfs (from about 847 to 635 cfs).

14.5 US36 Culverts

The OSMP property south of US36 drains through a series of culverts below US36. The total drainage area is approximately 60 acres, not including the OSMP area that drains to SBC. These culverts include the US36 wildlife crossing and DCD2 crossing discussed above and multiple smaller culverts. The smaller culverts include two 18-inch by 24-inch elliptical RCP, one 24-inch by 36-inch elliptical RCP, and one 24-inch RCP. These culverts discharge to OSMP property on the north side of US36 and are used to convey stormwater runoff during flood events and routine irrigation flows.

The spillway alignment is located approximately 25 to 40 feet upstream of the face of these culverts. Flows from the areas south of the spillway will be obstructed by the spillway wall. We understand that the culverts need to convey routine irrigation flows to maintain historic irrigation patterns on both sides of US36 and flow from routine rainfall and snow melt events. The culverts will be extended to the face of the spillway wall through the landscaping fill. Risers will be installed at the inlet of the existing culverts to accommodate drainage of the area between the US36 embankment and the spillway wall.

14.6 CU Boulder

The entirety of the OS-O land use area and about 55 acres of the PUB land use area on the CU Boulder South campus property will drain into the detention excavation. Approximately 70 acres of the PUB land use area will drain into the area between the dam embankment and roadway embankment, which will not freely drain to an adjacent drainageway. A culvert will be installed through the South Loop Drive earthen ramp to drain this area into the detention excavation. A flap gate will be installed on the culvert outlet to prevent water in the detention area from entering the PUB land use area. It may be desirable to discharge this culvert into Viele Channel instead of the detention area, and should be evaluated in the next stage of design.

SECTION 15 - SITE GRADING AND ACCESS

15.1 General

Site grading and site access improvements will be required to support the Project facilities discussed in the proceeding sections and to meet Project design criteria. Site grading will include detention excavation, levee removal, OS-O inflow rundown, and miscellaneous grading needed to promote site drainage. Site access modifications will be required for South Loop Drive and the multi-use trail. The Project will also include construction of new access roads through the site to provide access for maintenance and operation.

15.2 Site Grading

15.2.1 Detention Excavation

15.2.1.1 Grading

To ensure that the Project does not cause additional flooding on the main stem of SBC downstream of US36, the Project must be configured to maintain or reduce flows downstream of South Boulder Road for the design event. Based on hydraulic modeling (see Section 9), between 73 to 105 ac-ft of detention storage is required below the existing ground to achieve hydraulic and floodplain design criteria.

The detention storage will be achieved by excavation on the northern portion of the CU Boulder South campus. The detention excavation grading plan was developed to include the following features:

- The bottom of the excavation will be at El. 5344 to facilitate drainage to Viele Channel on the north side of US36. This will prevent the formation of a permanent pool in the detention excavation, which is undesirable because it could promote mosquito habitat and cattail and wetland vegetation.
- Low-flow channel approximately 2-feet-deep and at a 0.5-percent slope. This should provide sufficient drainage to prevent stagnation in the low-flow channel and will keep areas outside of the low-flow channel relatively dry during routine conditions. This should promote the growth of desirable riparian and upland vegetation outside of the low-flow channel rather than cattails and other wetland vegetation. It will also allow a majority of the bottom of the excavation to be relatively dry for maintenance access.

- Inflow rundown consisting of a grass-lined open channel rather than a concrete or grouted riprap chute. The inflow rundown will be graded at a 0.5-percent slope to reduce the risk of erosion during a large flood event. The inflow rundown will direct flows toward the south end of the excavation rather than the north end to reduce the likelihood of sediment and debris being deposited directly at the outlet works intake structure.
- Side slopes no steeper than 4H:1V for aesthetic and maintenance considerations.

Additional refinement of the grading will be performed in the next stage of design to appropriately incorporate the detention excavation area into the project features.

Since the excavation will be below existing groundwater elevations, a barrier wall is needed to keep the excavation from collecting groundwater. The barrier wall will be similar to the barrier wall described above for the embankment dam but will extend around the perimeter of the detention excavation.

Precipitation and stormwater runoff that enters the detention excavation will discharge through the outlet works or infiltrate. Soils within the detention excavation below the invert of the outlet works will typically be saturated because any infiltrated water will essentially be confined by the barrier wall. This will likely result in the bottom of the detention excavation being consistently moist and/or soft. This may also result in the growth of cattails and other wetland vegetation along the bottom of the detention excavation.

The detention excavation will be within 200 feet of the upstream toe of the dam, which violates the SEO's Rules and Regulations. Our embankment analyses considered this excavation, and it does not pose a stability risk to the embankment. A waiver will be requested from the SEO for this variance.

15.2.1.2 Perimeter Barrier Wall

RJH performed seepage analyses to support design of the soil-bentonite barrier wall along the perimeter of the detention excavation. Two-dimensional seepage analyses were performed using Seep/W, which is part of the GeoStudio 2021 software package. Seepage analyses were performed to estimate the quantity of groundwater seeping into the detention excavation either through or beneath the barrier wall, and to support selection of the depth the barrier wall should extend into bedrock. Analyses were performed at six sections along the barrier wall, which represent various material properties and groundwater conditions.

The modeled sections include alluvium, fill, fresh bedrock, and weathered bedrock. Along the alignment of the barrier wall, depth to top of bedrock ranges from 2 to 20 feet. Groundwater data was based on field measurements in existing monitoring wells and data from the Project groundwater model. The hydraulic material properties were developed based on information collected during RJH's geotechnical investigations, calibrations of the Baseline Groundwater Model, published data, and engineering judgement.

We evaluated seepage conditions that would exist during steady state seepage when the groundwater levels outside of the barrier wall are at the highest projected level and groundwater levels within the detention excavation are at the seasonal low level, which represents the most extreme seepage scenario. Based on the seepage analyses performed, RJH selected to extend the barrier wall 3 feet into bedrock, which is consistent with the current state of practice.

The selected design criteria are based on the SEO's, *Guidelines for Lining Criteria For Gravel Pits* (SEO, 1999). These guidelines define the design standard and performance standard for maximum allowable leakage rate into the detention excavation. The guidelines were written for gravel pits but are also applicable to the detention excavation. Based on the design standard, the calculated maximum allowable leakage into detention excavation is 4.0×10^{-2} cubic feet per second (cfs) or 18 gallons per minute (gpm), which is significantly less than the SEO performance standard maximum allowable leakage of 1.2×10^{-1} cfs or 54 gpm.

Additional documentation for the soil-bentonite barrier wall seepage evaluation is presented in Appendix H.

15.2.2 Levee Removal

The earthen levee that extends along the south and east boundaries of the CU Boulder South campus will be removed as part of the Project. The levee is approximately 7,500 feet long and varies in height, with a maximum height of about 14 feet. The levee will be excavated down to existing ground, which we have defined as the top of the adjacent DCD2 channel. The intent of the levee removal is to maintain DCD2 at its current alignment, condition, and flow capacity. Following levee removal, the existing ground along the levee alignment will be between 8 to 20 feet higher than the bottom of the reclaimed gravel pit, and the slope to the bottom of the gravel pit will be maintained. Earthen material excavated from the levee will be stockpiled for use in the embankment and other earthfill. The riprap excavated from the levee will be stockpiled for use as lining of the OS-O inflow rundown.

15.2.3 OS-O Inflow Rundown

After the levee is removed, the OS-O land use area will be inundated by SBC flood flows for events greater than about the 10-year flood event. Because the OS-O land use area was developed by gravel mining, it is about 20 feet lower than the surrounding SBC floodplain area. SBC flood flows need to be conveyed into the OS-O land use area in a controlled manner to limit the potential for erosion and subsequent sediment deposition in the detention excavation or outlet works facilities. The large changes in elevation between the SBC floodplain and the old gravel mining area create challenges for hydraulic conveyance of flows into the CU Boulder South campus site. Also, a large "knob" of unmined material protrudes along the south slope of the old gravel pit creating a non-uniform surface for overtopping flows.

An inflow rundown will be constructed at the south end of the OS-O land use area to facilitate controlled conveyance of overtopping SBC flood flows. The inflow rundown will be constructed by excavation and fill. The excavation will extend into the knob of unmined material. The inflow rundown will have a longitudinal slope of 5 percent.

RJH performed hydraulic modeling to evaluate the performance of the inflow rundown. Peak velocities during the 100-year event were calculated to be up to 9 fps in the rundown channel and 11 fps along the slopes of the old gravel pit. The inflow rundown and portions of the existing gravel pit slope will be lined with loose placed, buried riprap, which will provide sufficient erosion protection for flows up to the 100-year flood event. Riprap was sized in general accordance with guidance from MHFD Urban Storm Drainage Criteria Manual Volume 1 – Chapter 8: Open Channels for steep slopes (i.e., slopes exceeding 2 percent slope) (MHFD, 2017a). Type L riprap with a median rock size of 9 inches is acceptable. It may be possible to use the existing riprap along the wet-side of the levee for this application. Additional measurements of this riprap will be performed during the next stage of design to confirm.

During a 100-year event, we anticipate most of the surficial soil and vegetation in the inflow rundown would be eroded down to the top of riprap; however, the riprap should remain stable (i.e., not transported). Following the flood event, we anticipate widespread erosion repairs would be required, which would involve placing fill over exposed riprap and revegetating.

RJH also performed hydraulic modeling for the 10-year and 50-year flood events. Based on these results, we anticipate some minor to moderate erosion rilling would occur during these events, and some regrading and revegetation may be required after the flood. Additional information regarding the inflow rundown hydraulic evaluation is presented in Appendix H.

15.2.4 Miscellaneous Site Grading

Miscellaneous site grading will be required adjacent to the primary Project facilities. The site grading will be developed to drain to the closest respective drainageway (SBC, Viele Channel, detention excavation, etc.).

15.3 Site Access

15.3.1 South Loop Drive

South Loop Drive will be the primary access point to the site for permanent (post-construction) access. The alignment of the embankment dam extends across South Loop Drive. South Loop Drive will need to be reconstructed to provide access over the embankment dam.

An earthen roadway ramp will be constructed that extends over the embankment dam. The top width of the earthen roadway ramp will be 80-feet wide in accordance with the Annexation Agreement between the City and CU (City and CU, 2021). A 24-foot-wide paved asphalt road will be constructed on top of the earthen roadway ramp as part of this Project. CU will be responsible for future improvements to South Loop Drive.

The ramp north of the dam embankment will be at less than a 4-percent slope, which was selected based on roadway design criteria presented in the City of Boulder's Design and Construction Standards (Boulder, 2020). To the south of the dam embankment, the earthfill roadway ramp will extend along the western edge of the detention excavation. The top of the earthen roadway ramp at this location was set at the same elevation as the 500-year flood water surface elevation (El. 5368.0). Any excess excavation generated by the Project could be placed in the reservoir pool between the earthen roadway ramp and the dam embankment. This would not impact flood storage for the 100-year event and would provide cost savings compared to exporting excess excavated materials.

15.3.2 Site Access Roads

Permanent access roads will be required to provide access to Project facilities for future maintenance activities, and to access CU property. An aggregate access road will be installed along the crest of the dam. Access to the embankment dam crest will be from South Loop Drive and include a vehicle turnaround near the right end of the embankment.

As the design progresses, the City should consider the need to install permanent access roads at the following locations:

- Along the downstream toe of the embankment between Viele Channel and the embankment. This will provide access to toe drains for vegetation removal/maintenance along the downstream slope.
- Along the upstream toe of the embankment. This will provide access for vegetation removal/maintenance along the upstream slope.
- Along the upstream side of the spillway wall. This will provide access to groundwater conveyance system manholes.
- Into the detention excavation. This will provide access for the removal of sediment and debris.

15.3.3 Multi-Use Trail

The alignment of the spillway connection to US36 extends across the existing multi-use trail. An approximately 300-foot-long segment of the existing multi-use trail will be demolished and reconstructed at this location. An earthfill ramp will be placed along both sides of the spillway wall at this location to accommodate the multi-use trail. The slopes of the multi-use trail earthfill ramp will be at no more than 5-percent based on criteria from Boulder Parks and Recreation Design Standards Manual (Boulder, 2021). The width of the reconstructed multi-use trail was set to match the width of the existing trail.

If the Landscape Concept 1 (see Section 11) is selected, then an approximately 2,100 linear feet of the multi-use trail would be relocated closer to the spillway wall to improve use experience and overall aesthetics.

SECTION 16 - ENVIRONMENTAL PERMITTING, MITIGATION, AND RESTORATION

16.1 Environmental Permitting

16.1.1 Clean Water Act

The City will need to obtain a CWA Section 404 permit to construct the project because of anticipated impacts to wetlands. USACE will be the lead regulatory agency for this permit. The RJH Team and the City performed a site walk with Matt Montgomery with USACE on August 17, 2021. Based on this site walk, USACE provided the following preliminary opinions:

- The wetlands along the US36 corridor and north end of the CU Boulder South Campus will likely not be considered jurisdictional wetlands because they are not directly connected to SBC and lack inundation in a typical year.
- Wetlands along Viele Channel may be considered jurisdictional. If these wetlands are jurisdictional, then the South Loop Drive modifications south of US36 and the outlet structure north of US36 will impact jurisdictional wetlands.
- The work in Viele Channel north of US36 could likely be permitted under Nationwide Permit (NWP) 7 for Outfall Structures.
- The work in Viele Channel south of US36 for South Loop Drive modifications could likely be permitted under NWP 14 for Linear Transportation.
- The Area of Potential Effect will likely only be defined to include areas along Viele Channel. However, USACE will likely review the Biologic Assessment for the entire Project site.

USACE requested that the City submit a request for jurisdictional determination.

The RJH Team submitted a request for jurisdictional determination to USACE on November 11, 2022. The USACE provided an Approved Jurisdictional Determination letter on May 20, 2022. The USACE determined that Viele Channel and DCD2 meet the definition for waters of the United States. Work impacting Viele Channel and DCD2 will require a Section 404 permit. A copy of the request for jurisdictional determination and Approved Jurisdictional Determination letter are provided in Appendix I.

USACE will also require the development of a Biological Assessment and Cultural Resources Class III Report. Drafts of both the Biological Assessment and Cultural Resources Class III Report are in progress and will be finalized early in the next stage of design.

16.1.2 City of Boulder Wetland Permit

A City Wetland Permit will be required to construct the Project because of anticipated impacts to wetlands and regulated buffer areas under the City's jurisdiction. The City Wetland Permit will be based on Project impacts to delineated wetlands, not just those deemed jurisdictional by USACE. The City Wetland Permit will require an approved CWA Section 404 permit and a Compensatory Wetland Mitigation prior to approval by the City. An initial meeting was held with City Planning Department staff to discuss permit requirements and process. Additional work for this permit has not been advanced.

16.2 Environmental Mitigation and Ecological Restoration

The environmental mitigation will be constructed on-site in the OS-O portion of the CU Boulder South campus and will be performed in conjunction with a larger ecological restoration of this area. The goals of the environmental mitigation and ecological restoration include:

- Avoid impacts to and improve, where appropriate, existing wetlands, native grassland, and T&E species habitats.
- Increase ecological connectivity between the restoration area and SBC.
- Ensure the long-term sustainability of wetlands and uplands considering Site hydrology.
- Minimize impacts to existing wetlands and buffer zones.
- Avoid impacts to existing irrigation ditches.
- Reduce impacts to and protect T&E species and habitats.

A key part of the ecological restoration will include removing the existing levee located along the eastern portion of the OS-O land use area to reconnect the OS-O land use area to the SBC floodplain.

Westervelt and Headwaters developed a 60-percent design for ecological restoration. The existing Site topography, hydrology, and vegetation required advancing the ecological restoration design with different design concepts for the two distinct areas within the ecological restoration footprint:

- Interior of levee: The area on the interior (i.e., dry side) of the levee generally consists of a reclaimed gravel pit. The vegetation in this area is primarily upland grassland/shrubland. A drainage ditch is located along the toe of the levee and contains mapped ULTO specimens, ULTO-suitable habitat, and wet meadow/palustrine emergent wetlands. The ecological restoration design concept for the interior of the levee primarily consists of wetland re-establishment and wetland rehabilitation. The re-established wetland area will include seasonally inundated swales that are expected to be saturated-to-ponded in the early part of the growing season. The swale areas are expected to function similar to the existing drainage ditch along the toe of the levee. Connecting existing ULTO habitat on the exterior of the levee to created habitat on the interior of the levee will facilities ULTO seed transport and potentially germination.
- Exterior of the levee: The area on the exterior (i.e., wet side) of the levee generally consists of riprap vegetation that is native to the SBC riparian corridor. This area also contains three constructed ponds (aka "southern ponds") at the south end of the OS-O land use area. The southern ponds contain Typha-dominated wetlands (i.e., cattails) with mapped ULTO specimens and ULTO-suitable habitat along the banks. The ecological restoration design concept for the exterior of the levee is to restore this area to resemble the historical SBC floodplain. A key part of this work will include reducing the overall saturated hydroperiod of the ponds and increasing hydraulic connectivity between the ponds to encourage growth of more diverse wetland species. The ponds will be excavated to remove at least 1 foot of cattail-dominated soil and regraded to provide more diversity of elevations within the ponds. The earthen berms between the ponds will be partially excavated to increase hydraulic connectivity.

Upland buffer zone mitigation will be utilized throughout ecological restoration area to create a mosaic of uplands and wetlands to provide habitat connectivity between the interior and exterior of the levee. Two wetland types have been targeted as part of the design: wet meadow/PEM and wet meadow/PEM swale. PMJM compensatory mitigation measures for occupied habitat will be discussed with USFWS during the next phase of design but will likely incorporate shrub scrub plantings throughout portions of the restoration area. ULTO compensatory mitigation will occur through the creation of new habitats and enhancement of existing habitats and salvaging and replanting of sod from the permanent impact area to the mitigation area. Removing cattail monocultures and noxious species such as crack willow and Russian olive will help promote ULTO expansion throughout wetland and upland transition areas.

The 60-percent design does not require permanent irrigation to be sustained. However, the design has incorporated the possible use of DCD2 irrigation water during site establishment or for future adaptive management. The ecological restoration grading will connect with DCD2 and incorporate stop logs or turnout structures to divert water if desired.

The ecological restoration will meet or exceed all environmental mitigation requirements. A summary of mitigation requirements and anticipated mitigation generation is presented in Table 16.1.

TABLE 16.1
SUMMARY OF MITIGATION ACRAGES

Environmental Resource Type	Flood Project Impact Area (acres)	Proposed Mitigation Ratio	Proposed Mitigation Area (acres)	Mitigation Project Habitat Generation (acres)	Area in Excess of Required (acres)
Boulder Wetlands (Permanent Impacts)	7.4	2:1	14.8	36.7	21.9
Boulder Wetlands (Temporary Impacts)	16.7	1:1	16.7*	0.0	0.0
Boulder Wetland Buffer (Permanent Impacts)	28.6	1:1	28.6	32.1	3.5
Boulder Wetland Buffer (Temporary Impacts)	12.3	1:1	12.3*	0.0	0.0
PMJM Critical Habitat (Permanent Impacts)	0.3	2:1	0.62	0.6	0.0
PMJM Critical Habitat (Temporary Impacts)	0.3	1.5:1	0.45	0.5	0.0
PMJM Occupied Habitat (Permanent Impacts)	2.8	2:1	5.6	78.5	72.9
PMJM Occupied Habitat (Temporary Impacts)	10.7	1.5:1	16.1	16.1	0.0
ULTO Habitat	12.3	1:1	12.3	42.3	30.0

The Ecological Restoration 60-Percent Design Report is provided in Appendix J.

SECTION 17 - CONSTRUCTABILITY CONSIDERATIONS

17.1 General

RJH identified anticipated construction activities and Site conditions that are expected to impact the construction of Project facilities. Constructability items, along with a brief discussion of key issues and possible methods to address each issue, are provided below. Additional constructability evaluations will be performed in the next stage of design.

17.2 Contractor Staging

Construction activities require staging areas for contractor trailers, equipment, imported materials, and stockpile areas. It is generally desirable to locate contractor staging areas outside of the construction footprint if possible. For this Project, this will be possible if the entirety of the staging area was located on the CU Boulder PUB land use area. For 60-percent design, we have estimated dimensions for the contractor staging area based on our experience with similar projects and have assumed the contractor staging area will be located on the CU Boulder PUB land use area south of the earthen roadway ramp.

If the PUB land use area is not available for contractor staging, then the contractor staging area will need to be located on either the PK-U/O land use area or the OS-O land use area. The PK-U/O land use area is generally located throughout the footprint of the proposed detention facility, and the OS-O land use area is the location of proposed environmental mitigation and ecological restoration. Staging in either of these areas will likely require sequencing construction to accommodate contractor staging and relocating the staging area at some point during construction.

Coordination with CU Boulder will be required in the next stage of design to evaluate whether the PUB land use area can be used for contractor staging.

17.3 Earthwork Balance

Primary onsite borrow sources for the Project include the detention excavation, CU Boulder levee, CU Boulder west berm, ecological restoration, and inflow rundown. Primary fill areas include the dam embankment, inflow rundown, and earthen roadway ramp. We anticipate that the embankment core (Zone 1) will be obtained from fine-grained soil in the CU Boulder west berm, whereas material from the remaining excavations could be used for the embankment shell (Zone 2) and the earthen roadway ramp. A summary of the anticipated earthwork balance for the Site is presented in Table 17.1.

TABLE 17.1
SUMMARY OF EARTHWORK BALANCE

Work Item	Excavation (cy)	Earthfill [(1)] (cy)
Detention Excavation	119,000	39,000
Dam Embankment	175,000	237,000
S. Loop Drive Ramp	0	150,000
Spillway and Groundwater Conveyance System	40,000	33,000
Levee Removal	85,000	0
OS-O Inflow Rundown Fill	0	8,000
Ecological Restoration	149,000	Imported
Total	**568,000**	**467,000**
Excess Excavation	**101,000**	

Note:
1. Earthfill that could be generated using material excavated from on-site sources.

If landscape berms along the downstream spillway wall are selected to be included as part of the Project, then excess excavation could be utilized to construct the berms. Landscape Concept 1 and Concept 2 are anticipated to require about 14,000 cy and 7,500 cy of earthfill, respectively.

Earthwork balance will be affected by material shrinkage (i.e., new fill is compacted to a higher unit weight than material in the borrow areas) and removal of oversized particles (described in the next section), which will be further evaluated in the next phase of design. Also, only two borings were drilled in the levee, and one boring encountered some debris. It is currently unknown if this debris is localized or throughout the levee, which could impact the volume that can be used for fill material.

We considered that specially graded aggregates (embankment Zone 3 and Zone 4 and backfill for the groundwater conveyance system trenches) will be imported to the Site from commercial sources.

The current design does not include an allowance for an onsite waste area. Future stages of design should evaluate which miscellaneous materials generated during the work (bentonite slurry, bentonite-amended soil, material excavated from secant pile shafts, cobbles and boulders, etc.) will be suitable to incorporate into permanent fill, if these materials could be left onsite or if they will need to be disposed of offsite.

Depending on the landscape concept that is selected, we anticipate excess excavation could vary from about 85,000 to 95,000 cy. We anticipate CU would desire to use this material for future development of their site. On option would be to place this material between the dam embankment and the S. Loop Drive berm. The excess excavation would encompass the entirety of the area between the dam embankment and S. Loop Drive if it is placed to El. 5368.0 to match to top of the S. Loop Drive berm. Other options for placing the excess excavation could include thin spreading at the bottom of the mined gravel pit within the PUB land use area or stockpiling in a mound at a location designated by CU.

17.4 Oversized Particles

Oversized particles (i.e., cobbles and boulders) are expected to exist throughout the alluvium. Oversized particles will also be encountered within fill soils onsite; however, based on current data, the fill is expected to contain smaller-sized and less frequent oversized particles than the alluvium. The proposed construction techniques (e.g., pipe ramming for the tunnel and secant piles for the spillway foundation) were selected because these are preferred for handling oversized particles.

Oversized particles in the alluvium are expected to preclude the use of scrapers to excavate materials. Also, depending on where the excavated alluvium will be used for fill, screening of the material may be required to remove oversized particles. The approximate quantity of oversized materials that may need to be stockpiled or removed will be developed in the next stage of design. We anticipate that existing fill materials will be able to be excavated using scrapers and will be able to be placed directly as fill without screening.

Oversized materials excavated from the barrier wall alignments will also need to be selectively removed from soil-bentonite backfill. Supplemental fine-grained soil may need to be incorporated into soil-bentonite backfill if the materials excavated from the barrier wall trenches are too coarse.

17.5 Construction Water

Construction water will be needed for moisture-conditioning earthen fill, mixing bentonite slurry and soil-bentonite backfill, dust suppression, and other uses. We anticipate construction water will be provided by the City from a nearby hydrant. The contractor will be responsible for transporting or conveying water from the source to the site. The logistics associated with using City-supplied construction water should be further evaluated in future stages of design. Obtaining construction water from onsite sources such as existing water stored in ponds, runoff, groundwater, or groundwater that is dewatered from other Project alignments should also be evaluated. Key issues will be water rights, water quality, and water quantity.

17.6 Construction Space Constraints

The spillway and groundwater conveyance system will be constructed on OSMP property south of US36 to avoid impacts to existing utilities within the CDOT ROW. The City desires to reduce permanent and temporary impacts to OSMP property to the greatest extent reasonable using practical/common construction methods. The width of the corridor required to construct the spillway and groundwater conveyance system will depend on the construction techniques, equipment, sequencing, and duration selected by the construction contractor.

To assist the Project team with developing a reasonable estimate for this construction corridor, RJH retained a qualified contractor to identify key constructability considerations and potential construction sequences, equipment, and techniques. Based on discussions with the contractor and RJH's experience with similar projects, we have identified the following general sequence of construction activities required to construct the spillway and groundwater conveyance system:

- Excavating a working platform. The working platform will provide a level surface for equipment transportation and construction of facilities.
- Installing the secant piles. The working platform will need to be wide enough to accommodate a secant pile drill rig, concrete trucks, concrete pump trucks, and equipment for transporting and placing rebar and removing spoils. Sufficient space will be required for maneuvering the drill rig and for the concrete trucks to turn around since driving in reverse in a long, narrow corridor is impractical and unsafe.
- Dewatering below the working platform. This will likely require the installation of wellpoints.
- Installing the collection and distribution trenches. This will require an excavator to excavate the trenches and dump trucks to haul excess excavation and imported aggregate and trucks to deliver the pipe. Sufficient space will be required for the excavator and dump trucks to turn around and for some stockpiling of materials. Excavation for the trenches will require the use of trench boxes.

- Installing the reinforced concrete spillway wall. This will require concrete trucks, concrete pump trucks, and equipment for transporting and placing rebar and forms.
- Placing backfill to restore ground to existing grades and installing the reinforced concrete spillway apron and seepage management system.

Based on discussions with the contractor, we identified that a 64-foot-wide working platform with 1.5H:1V side slopes would provide reasonable working space to perform the required work tasks. This is based in part on assuming a) 8-foot-deep trenches boxes will be utilized to construct the groundwater conveyance and distribution trenches and b) access over the secant piles will not be feasible once they are constructed because of protruding rebar.

RJH and the City are continuing to work with the contractor to refine estimates for the construction corridor. Potential options could include an asymmetrical working platform (i.e., different widths on each side of wall) and installing a ramp over the secant piles to accommodate vehicle turnaround.

If irrigation of the OSMP fields is planned to occur during construction of the spillway and groundwater conveyance system, then additional space would be required for a collection ditch and pumping facilities at the south side of this construction corridor to manage this irrigation water.

Additionally, safe access needs to be provided for non-contractor personnel including City and RJH staff, materials testing personnel, and regulators. This would ideally be located adjacent to the main work corridor and we have currently included a 12-foot drive lane at the south side of this corridor.

The outlet works outlet structure will be constructed within a 30-foot-wide corridor on the north side of US36 between DCD2 and Viele Channel. An approximately 20-foot by 20-foot braced excavation will be used for both construction of the outlet structure and as a receiving pit for the tunnel, which will reduce the area of impact and reduce the risk to impact DCD2.

Construction of the soil-bentonite barrier walls require a flat to gradually sloped platform and a work area along one side of the trench, which has a minimum width that is equal to the depth of the trench (i.e., about 20 feet) for stockpiling and mixing backfill material. The 30-percent design accommodated this need.

17.7 Demolition

Demolition activities will include demolishing the CU Boulder tennis courts, CU Boulder maintenance building, and miscellaneous existing site utilities, fencing, and trails. Demolished facilities will need to be hauled off-site and disposed of by the contractor. It is possible that the CU Boulder maintenance building could include asbestos or other potentially hazardous materials that will require specialty procedures for handling. The material that needs to be demolished and disposed of will be evaluated in the next phase of the design.

Other facilities to be demolished include utilities, fencing, a portion of the CU cross country trail, and a portion of the concrete multi-use trail. We anticipate demolition and disposal of these facilities should be straightforward.

17.8 Tunneling

Construction challenges to tunneling below US36 include:

- High groundwater levels and high-permeable soils: Tunneling will likely occur below the groundwater table. Substantial dewatering operations will likely be required to accommodate shaft construction at each end of the tunnel alignment, and the inability to reliably dewater the tunnel alignment also affects the feasibility of various tunneling techniques.
- Cobbles and boulders along the tunnel alignment: The tunnel will predominantly extend through alluvial materials above the Pierre Shale bedrock. Based on geotechnical investigations performed for the Project, the tunnel will likely encounter wet sand, gravel, cobbles, and boulders. During test pit excavation, frequent cobbles up to about 3-4 inches in diameter and intermittent boulders greater than 12 inches in diameter were observed; however, there is a high degree of uncertainty of the size and frequency of the cobbles and boulders along a majority of the tunnel alignment. We have selected pipe ramming as the preferred technique because this method is best suited for advancing through the anticipated ground conditions.
- Presence of US36 and DCD2 above tunnel: The top of the casing pipe will be about 8 feet and 2 feet below the US36 roadway and DCD2, respectively. Construction risks associated with tunneling below these facilities are the formation of settlements and sinkholes from overexcavation, or ground heave caused by the displacement and repositioning of boulders as the tunnel is advanced. We anticipate that both CDOT and the DCD2 Company will require a monitoring and instrumentation plan for construction, and it is possible lining a short section of the ditch may be required.

- Elevation of alluvium-bedrock interface: The alluvium-bedrock interface may be located within the extents of the tunneling section. This would be problematic because tunneling through different types of material is challenging, and preferred equipment for tunneling through alluvial materials is often different than preferred equipment for tunneling through bedrock. Additional subsurface investigations will be performed in the next stage of design to better identify the elevation of the alluvium-bedrock interface.

During the next phase of design, a Geotechnical Baseline Report will be prepared to document subsurface conditions that the contractor should expect during tunnel construction, and allowable construction approaches. Instrumentation and monitoring plans will also be developed during design to monitor ground response near US36 and DCD2 during tunneling.

Construction of the tunnel will likely be sequenced at a time when DCD2 is not flowing water or a section of the ditch will need to be temporary conveyed by a pipe over the tunnel alignment. Additional coordination with the DCD2 Company will be required as the design advances.

17.9 Irrigation and Farming Operations

DCD2 and numerous smaller laterals exist to distribute water throughout irrigated areas on OSMP. Ideally, construction of the spillway will occur in non-irrigation season (i.e., generally November through March). Construction of the spillway is expected to occur during the irrigation season (i.e., generally April through October); therefore, temporary facilities will need to be installed to convey irrigation flows past the work area and into the culverts below US36. Construction of the spillway will also be less complicated if irrigation on OSMP South fields was reduced or eliminated during the construction period.

South Boulder and Bear Creek Ditch and DCD2 convey flow near the levee embankment. The contractor will be required to avoid disturbing ditch operations during removal of the levee.

The City and OSMP will need to coordinate with the farmers about the anticipated cattle grazing patterns during construction. If cattle will be on pasture on OSMP South during construction of the spillway, sturdy temporary fencing will be required to keep cattle out of work areas. The construction easement is narrow, and the installation of temporary fencing beyond the construction easement (i.e., further onto OSMP) may be required if farming activities are anticipated to continue during construction.

17.10 Flood Protection

Potential flooding sources that could impact the work include SBC, Viele Channel, and local drainages. It may be desirable to require the contractor to construct temporary facilities to protect the work from precipitation, runoff, and flood events. Temporary facilities such as cofferdams could be installed upstream of work areas and then periodically pumped to dispose of the accumulated water. The size of the storm (return interval) used to size the stream diversion facilities is a function of risk and cost. Selection of criteria and an approach to govern flood protection will need to be developed based on how the City decides to manage risk and cost.

One approach for selecting a return interval for flood protection is to consider the construction duration. A general rule of thumb in the industry is to provide flood protection for a return interval of approximately three times the construction duration. For example, if the anticipated construction duration is 18 months, flood protection for a 5-year flood event will be required. Another approach would be to allow the contractor to select the level of flood protection. This would transfer the risk to the contractor, and the cost for assuming this risk will be incorporated into the overall Project cost. The City should also consider the potential for catastrophic flooding of the construction site. We understand that SBC overtops its banks at about the 10-year flood event. It may be possible that the existing risk of Site flooding from SBC overtopping is acceptable without any flood protection facilities.

Regardless of the level of flood protection that is required, construction sequencing and specification requirements could be used to reduce the risk of flood damage. This could include:

- Constructing the spillway, outlet works, and ecological restoration prior to the embankment. This will allow the levee to remain in place for a longer duration to protect the OS-O land use area. Also, constructing the outlet works before the embankment will provide some protection during embankment construction because flows overtopping SBC could be conveyed through the outlet works.
- Requiring the contractor to move equipment out of the floodplain when major flooding is forecasted.
- The construction corridor along the spillway is not large enough to provide cofferdams around the spillway work area. Construction of the spillway will need to be sequenced when the risk of SBC flooding is low.

17.11 Groundwater and Dewatering

Lowest groundwater levels are typically during the winter months of November through February. Highest groundwater levels typically occur April through July. Low groundwater levels are desirable during excavation, and dry conditions are desirable for earthwork and material processing. Dewatering is expected to be required and will likely consist of pumping wells, sumps, pipes, and ditches. Dewatering will be required for the following:

- Spillway: We expect that well points will be used during construction of the spillway to maintain groundwater below the working platform. Construction should be sequenced such that excavations for the groundwater conveyance system is performed during fall and winter to avoid high groundwater conditions and increased pumping requirements due to seasonal high groundwater levels and irrigation activity.
- Site Ponds and Detention Excavation Area: Water will need to be removed from the site ponds for the detention excavation and from within the detention area once the perimeter barrier wall is constructed and for construction of the dam embankment. It is likely that wells will be needed around the ponds to lower the groundwater table to remove the muck and fill.
- Outlet Works: Substantial dewatering will be required to accommodate shaft construction at each end of the tunnel alignment. We expect that wells will be required for this work.

Construction of the dam embankment toe drain should be performed during low groundwater levels, or dewatering will likely be required. Groundwater is not expected to significantly impact barrier wall construction. However, at areas of shallow groundwater, the working platform will need to be elevated to several feet above groundwater to maintain stability of the excavation.

17.12 Other Construction Sequencing

The work will need to be sequenced to accommodate the potential constructability issues described above. Additional sequencing considerations are:

- The barrier wall around the detention excavation should be installed prior to the detention excavation to reduce groundwater inflows and to permit this work to be performed in the dry.
- The barrier wall below the dam embankment will need to be installed prior to embankment construction.

- The narrow construction corridor of the spillway will require the contractor to carefully sequence construction of the groundwater conveyance system, secant piles, and spillway wall.
- Several intersecting structures will need to be sequenced appropriately. Intersecting structures include the groundwater conveyance system and secant piles; the outlet works tunnel and secant piles; the dam embankment barrier wall and secant piles; the detention excavation barrier wall; the dam embankment barrier wall; and the secant piles; and the connection of the dam embankment and spillway.
- Selection of the construction sequencing should also consider the timing of site reclamation and reseeding. It is desirable to reseed disturbed areas shortly after the completion of site work to reduce erosion of exposed soil, and it is also desirable to reseed in the spring or fall to facilitate germination and establishment of the vegetation.
- Sod and topsoil on OSMP property along the spillway corridor will be stockpiled and replanted. The removal of sod and topsoil should be performed during a period that would be beneficial for timely replanting and when receiving sites have been prepared.
- The following sequencing requirements were identified during development of the Biological Assessment by ERO and will likely be required as part of the environmental permits:
  - Impacts on designated critical habitat should be mostly avoided during PMJM active season with the majority of construction within the critical habitat areas occurring during PMJM hibernation period (August 15 to May 31), if feasible.
  - Any shrubs located in all mapped PMJM habitat areas disturbed during construction should be removed outside the hibernation window (shrub removal to occur between June 1- Aug 15).
  - During the PMJM active season, which is between June 1 and August 15, work located within mapped PMJM habitat should occur only during daylight hours.

We anticipate some of the construction sequencing will be at the discretion of the contractor, and others will be required in the specifications.

17.13 Traffic Control and Site Access

South Loop Drive is the primary access point to the site. South Loop Drive will be closed to the public during construction but will be maintained for construction access. The alignment of South Loop Drive will need to be modified during various stages of construction to facilitate continuous use for construction access. Other site access points, including from Tantra Drive and Marshall Road, will need to be closed during construction. The contractor will need to provide fencing around the site/work areas for public safety and contractor security.

Temporary (construction) access roads will be needed by the contractor for the movement of equipment and materials. The locations and details for these roads will be left to the contractor, provided that the temporary roads meet the safety requirements for construction access roads as described by Occupational Safety and Health Administration regulations and that ditch flow is maintained in all irrigation ditches (i.e., DCD2, South Boulder and Bear Creek Ditch, and Upper Bear Creek Ditch). Upon completion of construction activities, the contractor will be responsible for the reclamation of temporary access roads.

The multi-use trail will need to be closed during construction. The duration of closure will depend upon the selected landscape architecture concept along the spillway and the City's preferred detour route. At minimum, the trail will need to be closed when the spillway connection to US36 is being constructed. Potential trail detour options include:

- Existing bike lanes along S. Boulder Road and Cherryvale Road. These lanes are part of the Boulder bikeway system. This is the longest of the potential trail detour options.
- The existing gravel trail along SBC north of US36 and S. Boulder Road bike lanes. The gravel trail would likely be difficult to use for road bicycles.
- Following construction of the US36 connection, the existing multi-use trail could be utilized. This would allow traffic on the multi-use trail when a majority of the spillway is being constructed. Safety fencing would be required.

A preferred trail detour plan needs to be identified in the next stage of design.

Portions of the gravel trail on OSMP north and south fields will be temporarily closed to the public during key points of construction.

17.14 Site Reclamation

Near-surface material will be stripped from the footprints of the dam embankment, reconstructed South Loop Drive, spillway, and detention excavation prior to earthwork in these areas. The stripped material will be used as topsoil to reclaim disturbed areas, and the material will need to be stockpiled in the contractor staging unless it can be used to reclaim portions of the Site as work progresses.

We do not anticipate that imported topsoil will be required for reclamation of flood control project components; however, additional topsoil may be required for ecological restoration. We will need to coordinate with OSMP and environmental consultants about whether there are special requirements for selectively removing and replacing topsoil and wetland vegetation along the spillway alignment separately from general site topsoil.

SECTION 18 - REFERENCES

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