RJH
CONSULTANTS, INC.

GEOTECHNICAL AND
WATER RESOURCES ENGINEERING

PRELIMINARY 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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July 2022
Project 16134

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LIST OF ABBREVIATIONS

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 Preliminary Design Report (Report) presents the results and conclusions of the preliminary design (i.e., 30-percent).

The preliminary (30-percent) design is documented in this Report and the 30-percent design drawings (Appendix J, bound separately). This Report and the drawings are complimentary to each other and combined represent the 30-percent design of the Project.

The preliminary design presented in this Report is based on hydrologic and hydraulic modeling, our current understanding of subsurface and groundwater conditions based on initial and preliminary design 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 preliminary design.

1.3 Scope of Services

RJH performed the following services for the preliminary design phase of the Project:

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 Phase II geotechnical investigations that included drilling, excavating test pits, performing a geophysical survey, performing field and laboratory testing, and preparing a Phase II Geotechnical Data Report.
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.
7. Performed a subsurface utility engineering (SUE) survey and incorporated utility information into Project base mapping.
8. Developed a Design Criteria Memorandum to identify and document 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 Preliminary Corrected Effective Model in the MIKE FLOOD program.
11. Performed hydraulic modeling in the MIKE FLOOD program to support the preliminary site layout and sizing of Project facilities.
12. Performed geotechnical analyses to support preliminary design of the embankment.

13. Performed geotechnical, geostructural, and hydraulic analyses to support preliminary design of the spillway and appurtenant structures.
14. Performed hydraulic and geotechnical analyses to support preliminary design of the outlet works.
15. Performed geotechnical analyses to support preliminary design of the barrier wall and groundwater conveyance system.
16. Developed design drawings to 30-percent complete level.
17. Prepared an American Society for Testing and Materials International (ASTM) E2516-11 Class 3 (ASTM, 2011) (i.e., budgetary level) opinion of probable project cost (OPPC) for the design.
18. Developed an Alternate Corrected Effective Model using the USACE HEC-RAS two-dimensional (2D) computer program to model the 100-year event. The Corrected Effective Model was developed in the MIKE FLOOD program.
19. Performed field mapping of existing Ute ladies' tresses orchid (ULTO) specimens for the 2021 survey season.
20. Prepared and submitted a Request for Jurisdictional Determination to USACE.
21. 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), E.I.
Samantha Guillies, P.E.
Adam Merook, P.E.

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)
RESPEC Company, LLC (RESPEC)

Environmental Permitting:	CORVUS Environmental Consulting, LLC (CORVUS) ERO Resources Corporation (ERO)
Surveying:	Flatirons, Inc. (Flatirons)
Subsurface Utility Engineering:	Surveying and Mapping, LLC (SAM)
Detention Excavation Layout:	Muller Engineering Company (Muller)
Tunnel Engineering:	Lithos Engineering (Lithos)
Cultural Resources:	PaleoWest

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.
City Dam Safety Advisor:	Kevin Clark, P.E.
MHFD Advisor:	James Watt, P.E. 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 preliminary 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 preliminary 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 preliminary 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 preliminary 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 that is 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:

- <u>Open Space - Other (OS-O):</u> 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.
- <u>Public (PUB):</u> 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.
- <u>Park, Urban, and Other (PK-U/O):</u> 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 SBC 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 SBC State Natural Area was designated by the state of Colorado in 2000 in recognition of the high-quality habitat and plant communities. The OSMP property is also used for cattle grazing seasonally, and portions 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.

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.

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 facilitate flood irrigation of OSMP property south of US36.

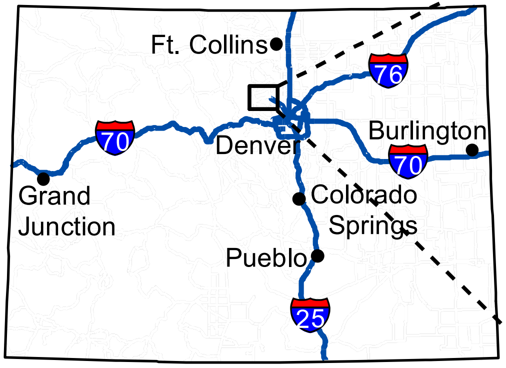
Numerous other irrigation ditches and smaller lateral irrigation channels (laterals) exist to distribute water throughout irrigated areas on OSMP. Based on information from OSMP and field observations by RJH, water is supplied to the OSMP fields using flood irrigation by placing check dams in irrigation 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

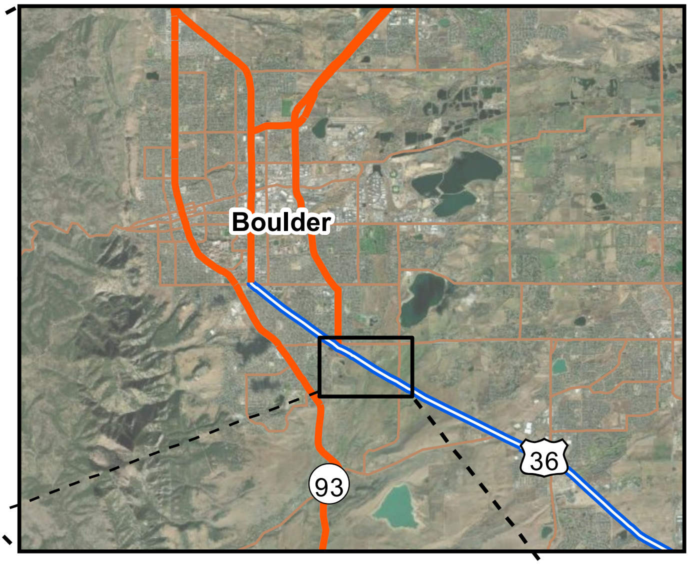
Based on site investigations performed by RJH (RJH, 2019; RJH, 2022b), our interpretation of the general subsurface profile at the Project site consists of fill or alluvium overlying bedrock of the Pierre Shale formation. In general, fill overlies bedrock throughout mined portions of the CU Boulder South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

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).

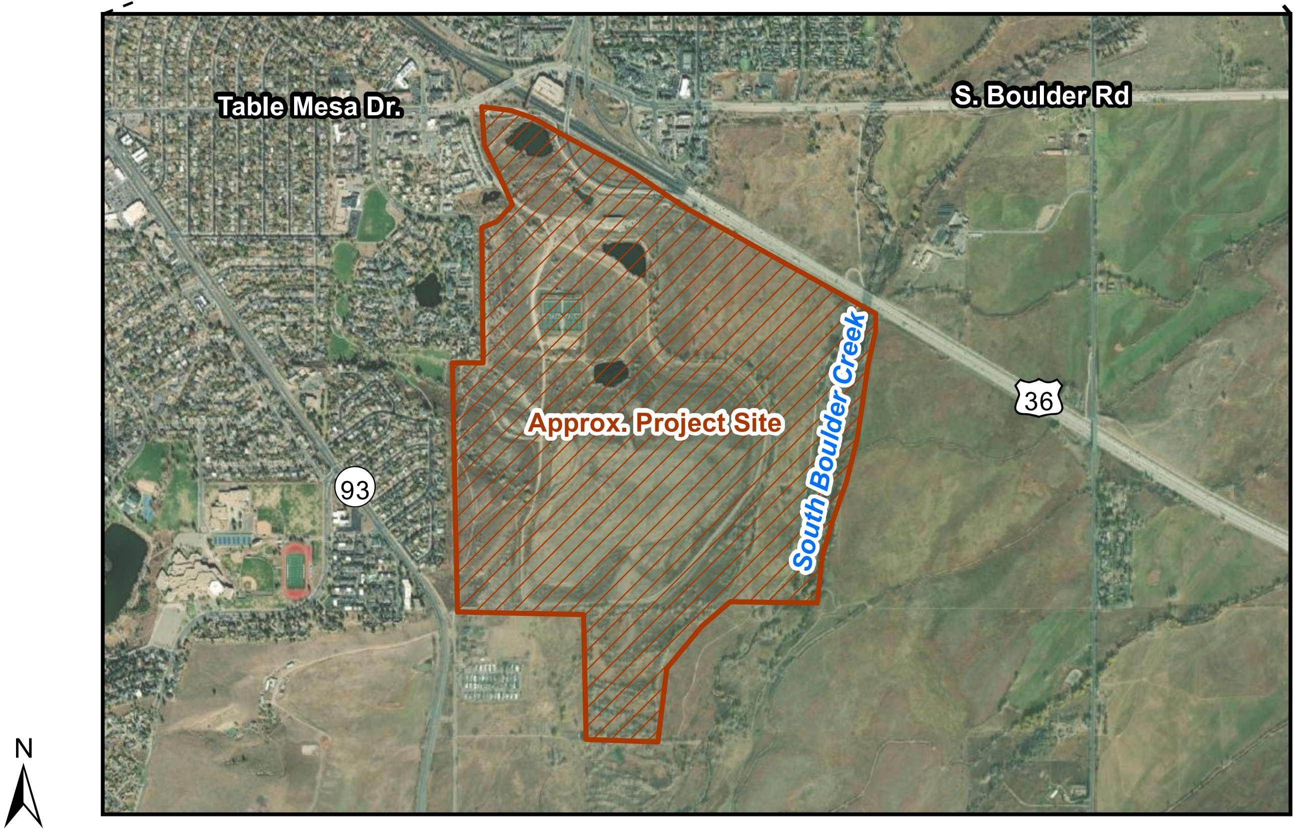
Based on site investigations and groundwater modeling performed by RJH, groundwater levels decline toward the north through the aquifer, which generally follows the slope of topography and the flow of SBC. Seasonal groundwater fluctuations are influenced by natural conditions through the hydrogeologic 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 May through July.



PROJECT VICINITY MAP
(NOT TO SCALE)



PROJECT LOCATION MAP
(NOT TO SCALE)



SITE LOCATION MAP
(NOT TO SCALE)

SOUTH BOULDER CREEK
REGIONAL DETENTION
PRELIMINARY DESIGN REPORT

SITE VICINITY
MAP

RJH CONSULTANTS, INC.

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Figure 3.1

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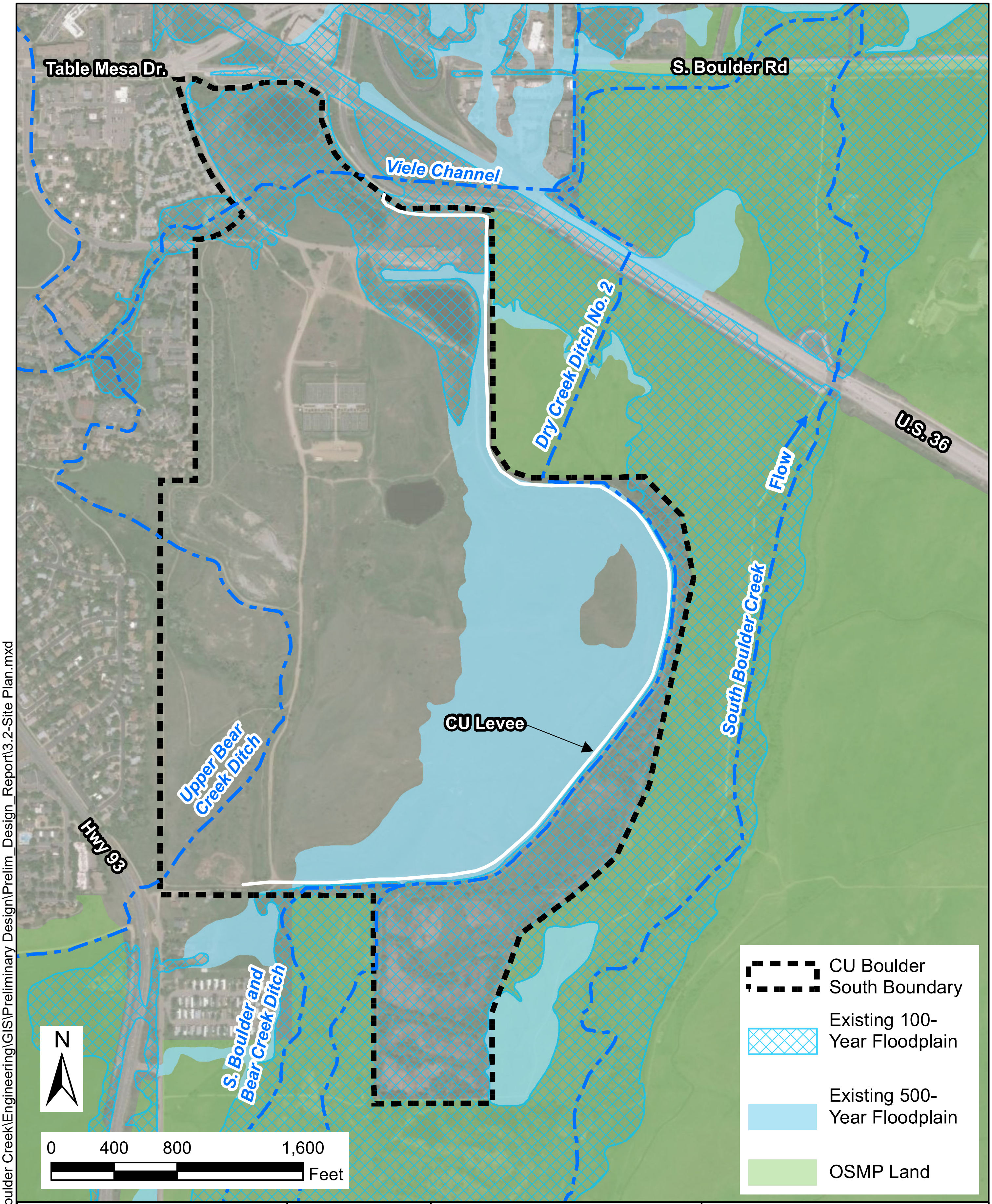

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SITE PLAN

PROJECT NO. 16134

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Figure 3.2

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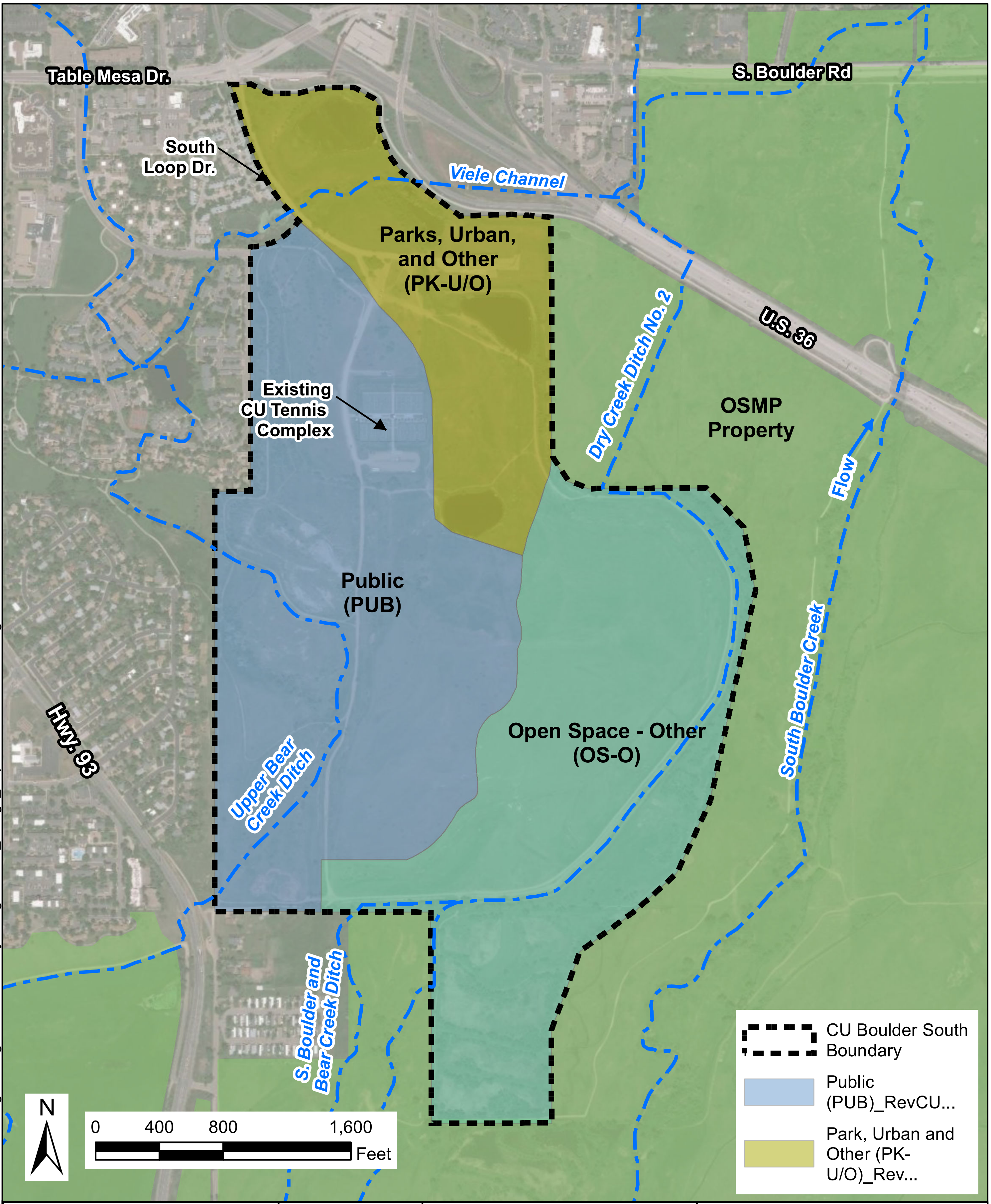

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SOUTH BOULDER CREEK
REGIONAL DETENTION
PRELIMINARY DESIGN REPORT

CU BOULDER SOUTH CAMPUS
AND LAND USE
DESIGNATIONS

PROJECT NO. 16134

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Figure 3.3

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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. 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.

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 habitat. 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 riser structure located at the upstream toe of the detention excavation. The front, top, and sides of the structure will be open and include steel trashracks.
- Conduit: The conduit will consist of a 60-inch diameter steel pipe located within a 96-inch diameter carrier pipe. The portion of pipe upstream of US36 will be installed in an open excavation and encased in reinforced concrete and tunneling will be performed to install the pipe below US36.
- Outlet Structure: The outlet structure will consist of reinforced concrete, baffled outlet structure 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 existing reinforced box culvert below US36 will be extended to the face of the spillway wall. This will accommodate future operation of the ditch without obstruction from the Project.
- Wildlife Crossing: The spillway alignment is located approximately 75 feet upstream of the face of the wildlife crossing. The wildlife crossing will be extended 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 approximately 65 feet upstream of the face of these culverts. Small low-flow openings will be installed in the spillway wall directly upstream of each smaller culvert and will convey irrigation flows and runoff from small events. The larger culverts for the wildlife crossing and DCD2 will be extended to the face of 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 maintain design capacity for flood mitigation.
- 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.
- 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: Existing levee will be partially removed to connect existing ground on both sides of the levee.
- 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.

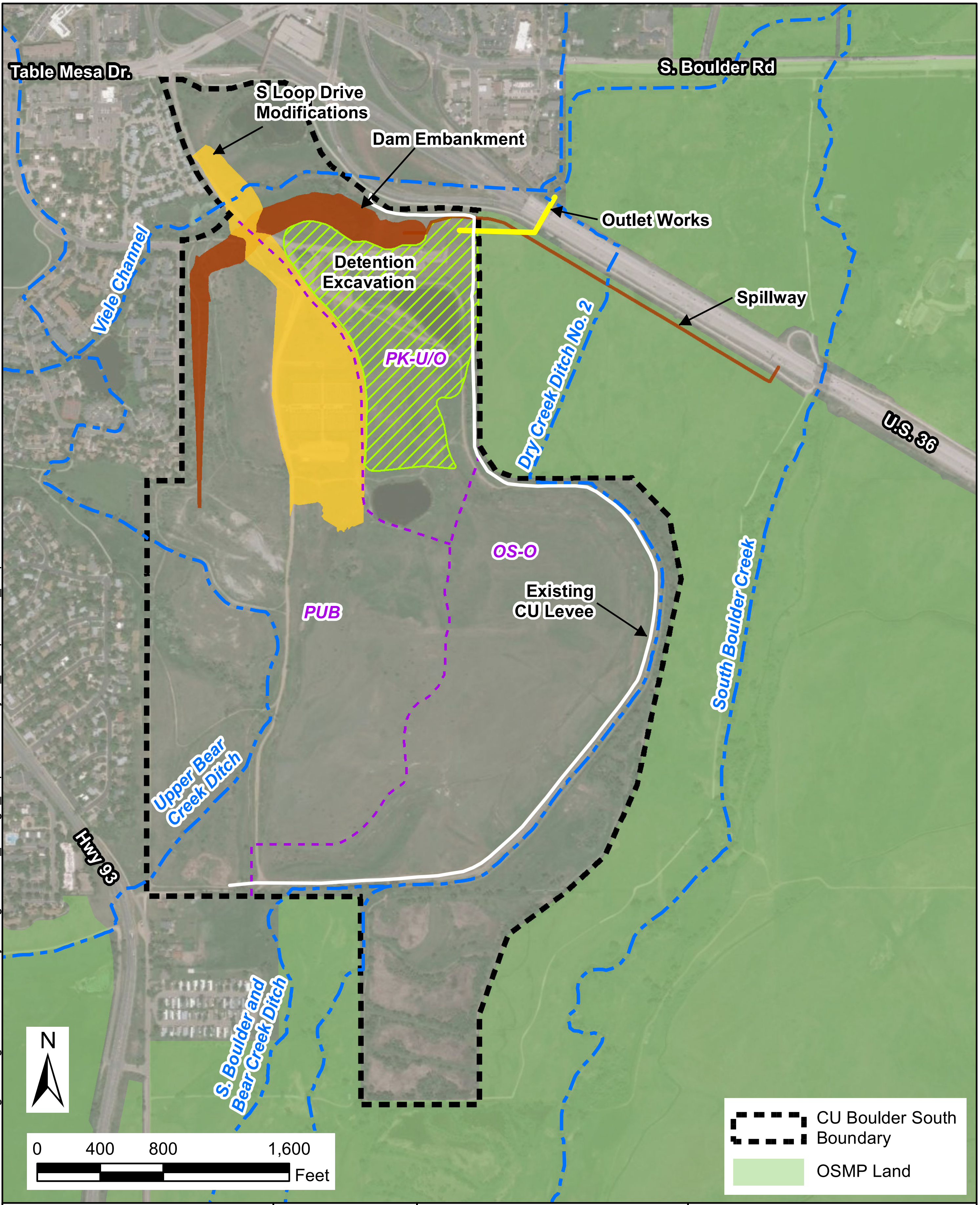
Additional information regarding the site grading and access design is presented in Section 15.

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 critical 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 critical habitat will be graded and revegetated to facilitate suitable habitat. The goals of the environmental mitigation and ecological restoration include:

- Removal of the existing levee embankment to reconnect the OS-O area to the SBC floodplain and riparian corridor.
- Development of new wetlands while maintaining current wetlands in the OS-O area.
- Development of new T&E habitat while sustaining current T&E habitat conditions in the OS-O area.

Concepts for environmental mitigation and ecological restoration have not been developed for the preliminary design.



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SOUTH BOULDER CREEK
REGIONAL DETENTION
PRELIMINARY DESIGN REPORT

PLAN OF PROJECT FACILITIES

PROJECT NO. 16134

July 2022

Figure 4.1

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SECTION 5 - DATA COLLECTION

5.1 General

Various types of data collection will be required throughout the Project to advance the design. During the preliminary 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

Flatirons performed topographic surveying in the winter of 2018 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. As work on the Project was being advanced, a discrepancy was identified in the survey data for the OSMP property south of US36. Flatirons resurveyed this area in November 2021 using conventional survey equipment and updated the base mapping. The limits of the 2018 and 2021 survey are presented on Figure 5.1.

Borings and monitoring wells from the Phase II geotechnical investigations were surveyed by Flatirons in June 2020 and October 2021. The locations of test pit investigations were recorded by RJH in August 2021 using a handheld Global Positioning System (GPS) device, and the ground surface elevations were estimated from the Flatirons topographic survey.

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 plan of the limits of the SUE survey is presented on Figure 5.2. The QL-D, QL-C, and QL-B work was performed in late 2021. The QL-A fieldwork is in progress. The Project base map was updated to include the QL-B SUE information. Results of the QL-B SUE investigation are presented on plan drawings. This information will be updated in the next phase of design when the QL-A survey is complete.

5.4 Geotechnical Investigations

RJH performed geotechnical investigations to obtain subsurface data needed to advance the Project design. The investigation was performed in two phases between 2018 and 2021. 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.

The geotechnical investigation included the following components:

- Performing geological mapping.
- Drilling 44 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 37 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 5 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) and *Phase II Geotechnical Report - South Boulder Creek Regional Detention* (RJH, 2022b). 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 that 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 by CORVUS 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 2019, 2020, and 2021, ULTO data from the City and Colorado Natural Heritage Program, wetland data from OSMP, and PMJM habitat data from the City and USFWS.

5.5.2 Wetland and T&E Surveys

CORVUS performed an environmental survey between September 11 and October 14, 2019, that included identifying channels, ditches, open water, and wetlands and assessing potential habitat for T&E species listed under the Endangered Species Act (ESA). 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, OSMP, and CU Boulder South Campus OS-O and P-U/O land use areas. CU Boulder independently retained CORVUS to perform a similar environmental survey of the CU Boulder South Campus PUB area.

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 required for Formal Consultation with the USFWS. USFWS requires surveys be performed during the flowering period, which is generally July 20 to August 31. ULTO surveys were completed during the 2020 and 2021 ULTO flowering seasons, and represent survey years 1 and 2, respectively. The 2020 ULTO survey was performed between August 10 and 18, 2020, and the 2021 ULTO survey was performed between August 9 and 11, 2021.

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.

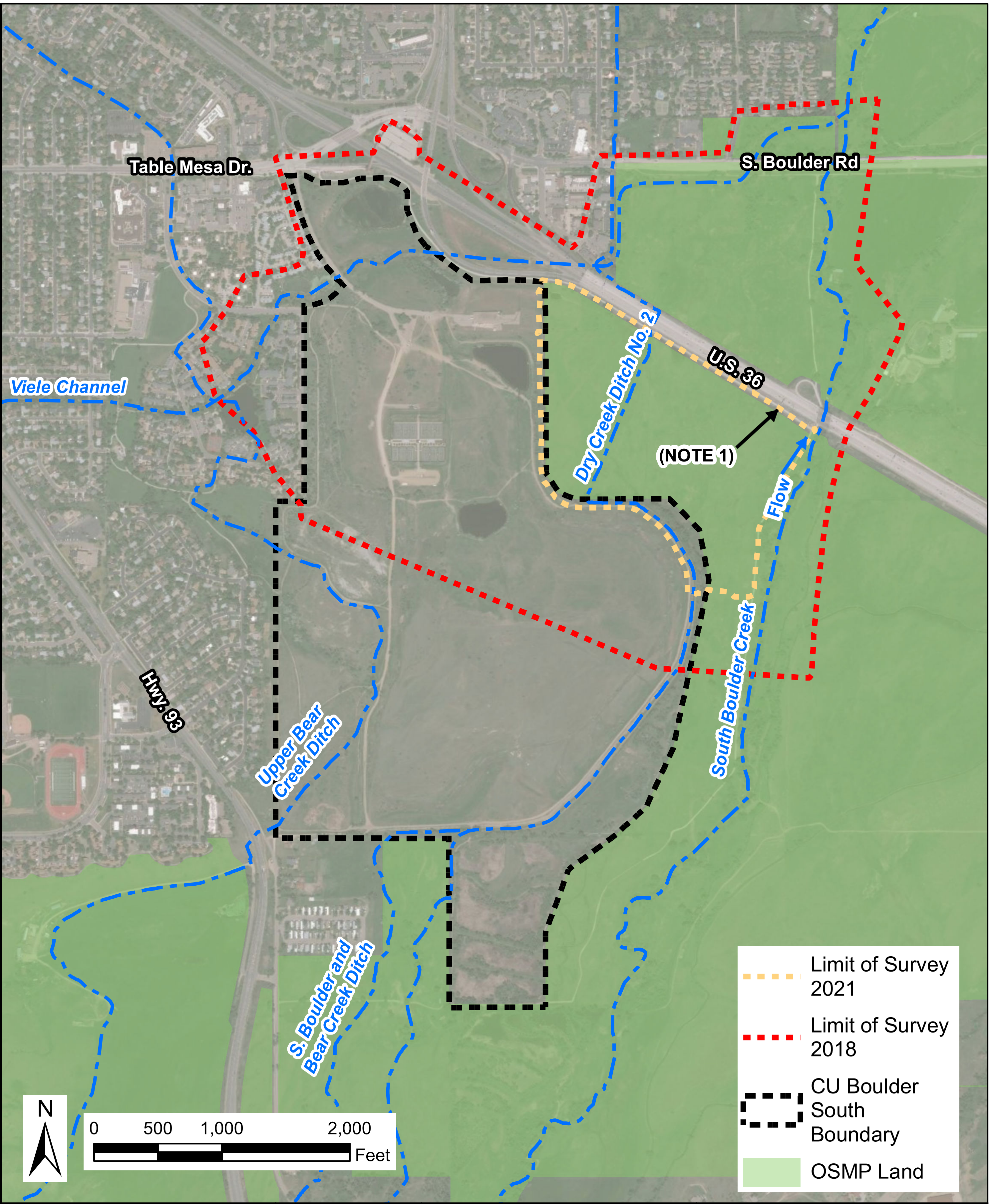
5.5.4 Cultural Resources

PaleoWest completed a Class I cultural resources evaluation of the Project site. The objectives of the Class I cultural resources evaluation included:

- Identifying the number and types of cultural resources that are or might be present within a 1-mile radius around the Project site.
- Providing a summary of the Project's potential to impact historic resources.
- Providing preliminary recommendations regarding any additional cultural resources work.

A representative of the Office of Archaeology and Historic Preservation performed an official file search of COMPASS for a 1-mile radius around the Project site. PaleoWest reviewed topographic maps and consulted Bureau of Land Management General Land Office records to identify any Historic-period resources.

PaleoWest identified one historic property that has been previously recorded within the 1-mile radius of the Project site and recommends completing a Class III cultural resources inventory. Pending results of a Class III inventory, the preliminary impact assessment is that the Project will have no adverse effect on the historic property, and the proposed Project will benefit the historic property by protecting it against future flood damage.



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NOTE:
1. 2021 SURVEY DATA INCORPORATED INTO PROJECT BASEMAP.

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PLAN OF
SURVEY LIMITS

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July 2022

Figure 5.1

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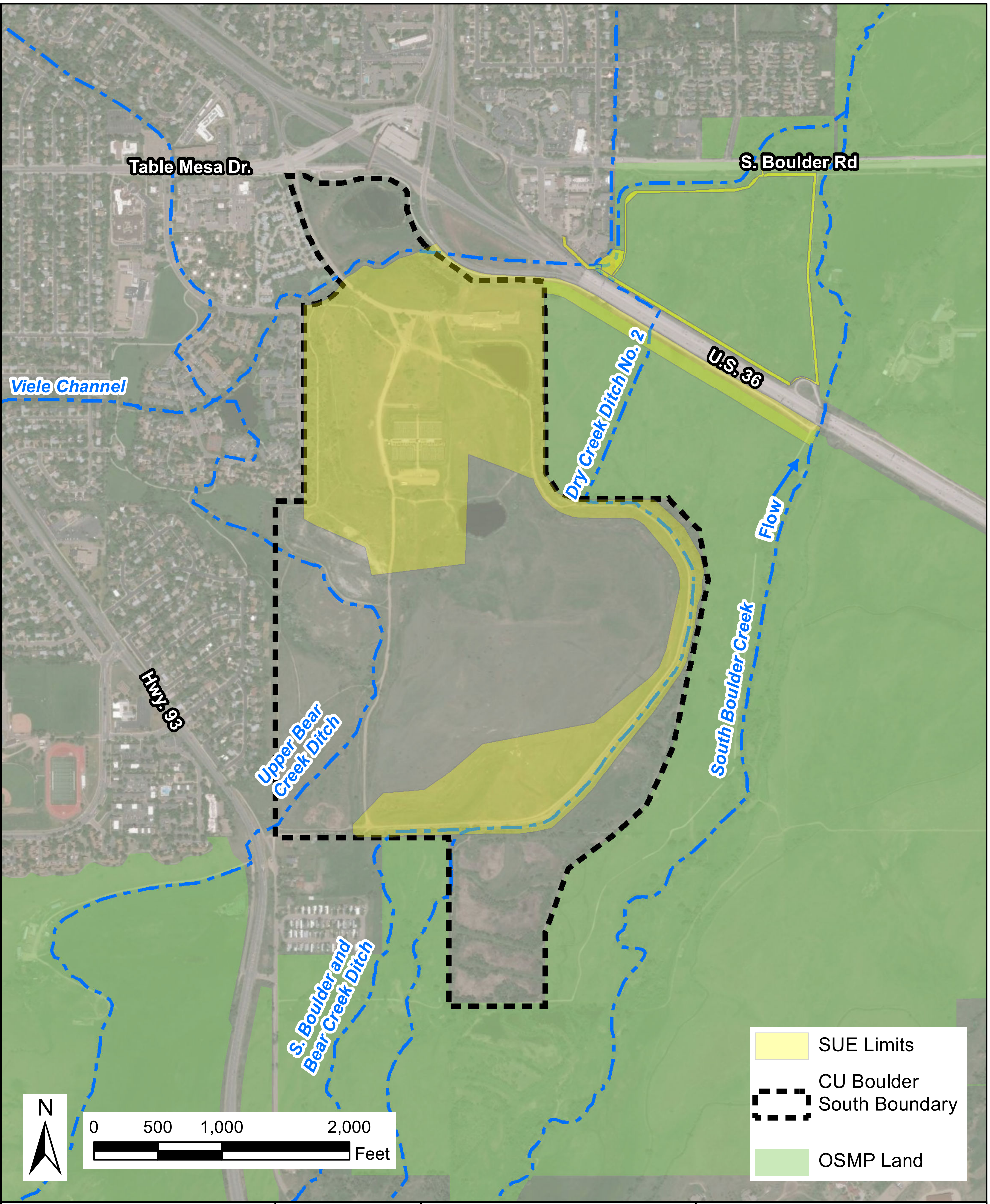

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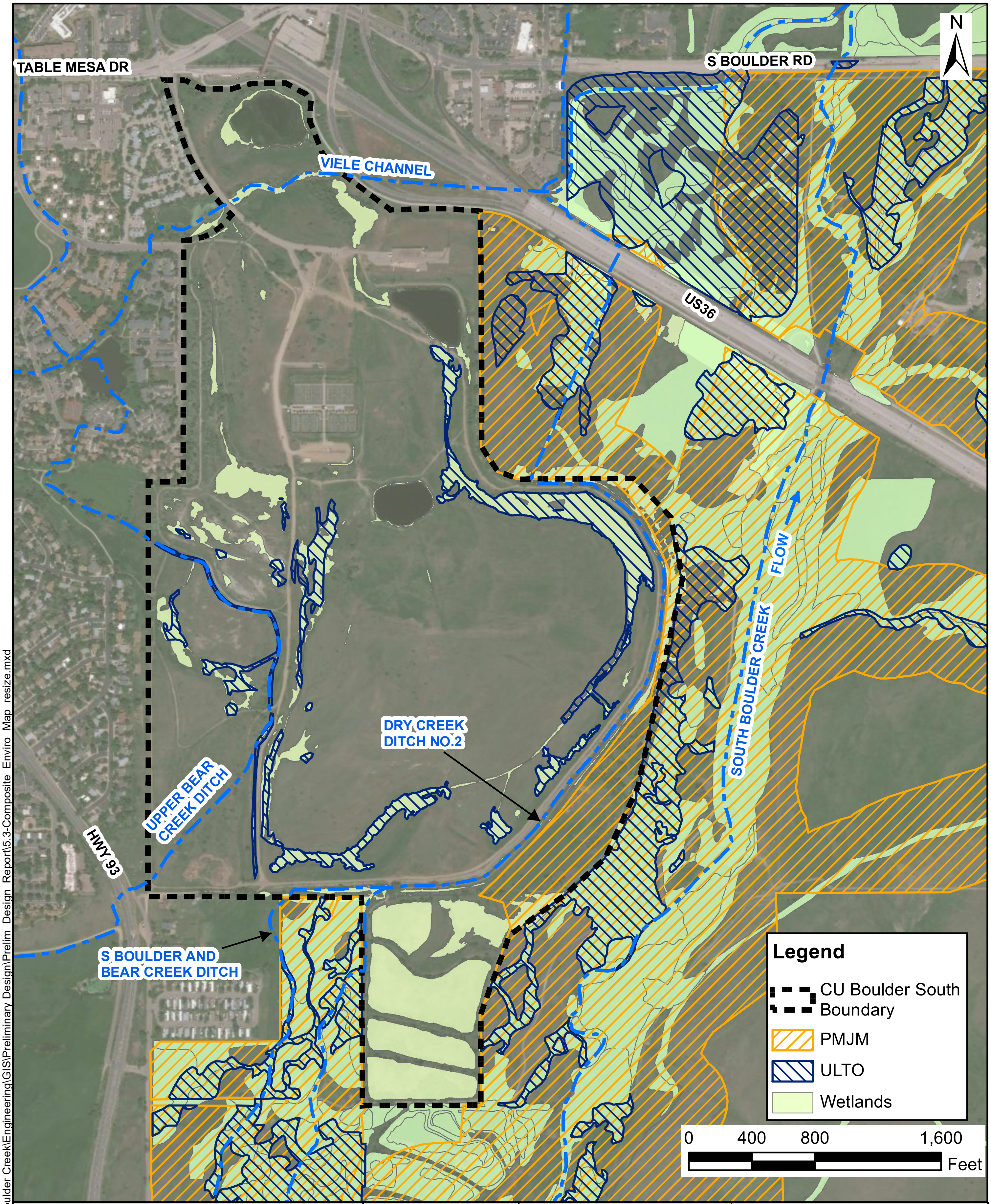
PLAN OF SUBSURFACE
UTILITY ENGINEERING
SURVEY LIMITS

PROJECT NO. 16134

July 2022

Figure 5.2



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REGIONAL DETENTION
PRELIMINARY DESIGN REPORT

PROJECT NUMBER 16134

COMPOSITE ENVIRONMENTAL RESOURCES MAP

July 2022

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 preliminary 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* (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.

6.3 Phase I and Phase II 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) and *Phase II Geotechnical Report - South Boulder Creek Regional Detention* (RJH, 2022b). 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 preliminary design criteria that has been used to develop this preliminary 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.
- 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

We anticipate the following stakeholders 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 will be provided an opportunity to review 30-, 60-, and 90-percent design submittals, and OSMP staff will 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 will need to be incorporated into the design. CU staff will be provided an opportunity to review 30-, 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.
- MHFD: MHFD will be a funding partner for construction and has been actively involved in development of the concept and preliminary designs for the Project. MHFD staff will review 30-, 60-, and 90-percent design submittals and provide input for development of the future phases of design.
- DCD2 Company: DCD2 is owned and maintained by the DCD2 Company. The spillway will intersect DCD2 approximately 75 feet upstream of US36, and ditch 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 of 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 anticipate the dam will have 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 preliminary 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. Additional analyses may be performed in the next stage of design to confirm the 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).

Embankment:

- 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 for preliminary design 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, the top of bank of Viele Channel, or the existing CDOT ROW, whichever is more restrictive.

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 have 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 will 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 during preliminary design based on the regulatory floodplain principle of generally not increasing downstream flood extents or depths. Additional discussions with FEMA will be required to identify more specific floodplain regulation requirements. We anticipate discussions will occur early in the next stage of design and will include MHFD, the City Floodplain Manager, and Boulder County Floodplain Manager. The Project team needs to identify a preferred modeling approach for the CLOMR prior to initiating discussions with FEMA (see Section 9).

7.5.2 City (Owner) Criteria

City requirements for the Project were based on requirements identified during preliminary 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. The City and OSMP have had on-going discussions throughout the development of the preliminary design and will continue to have discussions as the Project advances. The 30-percent design has been developed based on a 90-foot-wide construction corridor on OSMP property south of US36. We anticipate discussions regarding additional requirements for this corridor will continue.

<u>Hydraulic and Hydrologic</u>

- 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.

<u>Hydrogeologic</u>

- 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.

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.

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 to the site through South Loop Drive must be maintained. If modifications to the road are required, the road should be modified to maintain the existing level of service and overall condition, including paved to 24 feet in width. 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.
- 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

The preliminary design was advanced to accommodate future operations in DCD2 without obstruction from the Project and to ensure increases in flow and head through the US36 culvert will not negatively impact the long-term condition of the culvert. We anticipate discussions with DCD2 Company will occur early in the next stage of design.

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 $1x10^{-7}$ 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 2.0 to 26.0 feet in thickness and was underlain by alluvium or bedrock. The fill consisted of mostly clayey sand with gravel. Twenty-one sampler locations encountered refusal (50 blows for less than 6 inches). At 36 other sample locations, uncorrected SPT N-values ranged from 1 to 72 and averaged 23. The fill ranged from dry to wet, and very soft to very stiff. One sample wase nonplastic. Fifteen samples had liquid limits that ranged from 23 to 80 and plasticity indices that ranged from 6 to 54, with averages of 37 and 17, respectively. The maximum particle size was recovered in the test pits and was 18 inches. Pocket penetrometer results ranged from 0.25 to 3.0 tsf. Horizontal hydraulic conductivity measured from 11 rising head tests ranged from $3.4\text{x}10^{-6}$ to $3.6\text{x}10^{-4}$ cm/s, and the geometric mean was $5.4\text{x}10^{-5}$ cm/s. Vertical hydraulic conductivity was $2.6\text{x}10^{-5}$ cm/s for an intact sample of fill and $2.0\text{x}10^{-6}$ cm/s for a sample that was remolded to approximately 95 percent of the standard Proctor maximum dry unit weight at about 0 to 2 percent above the optimum moisture content. For samples of intact fill, RJH interpreted that the drained strength failure envelope can be represented by a drained friction angle of 36 degrees and no cohesion. The undrained strength failure envelope was interpreted to be an undrained friction angle of 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 two borings and ranged from about 6.4 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 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 two other sample locations, uncorrected SPT N-values were 13 and 33. The fill ranged from dry to moist, sands were generally dense to very dense, and clays were stiff to very stiff. For one sample, the liquid limit was 31, and plasticity index was 15. The maximum particle size recovered was 8 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, and therefore the current alluvial aquifer is constricted around the east side of CU Boulder South campus.

Alluvium was encountered below portions of fill west of the levee and at the ground surface in areas east of the levee. Alluvium ranged in thickness from 1.0 to 20.8 feet and was underlain by Pierre Shale bedrock. Alluvium predominantly consisted of a variety of coarse-grained material. 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 one test pit represent about 30 to 60 percent of the volume. Samples collected in the test pit better represent the coarser material, which was generally a gravel with silt, 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 46 percent of the SPT samples encountered refusal (50 blows for less than 6 inches). Uncorrected SPT N-values ranged from 7 to 73 and averaged 35. The alluvium ranged from dry to moist above the groundwater table and moist to wet below the groundwater table. The density ranged from loose to very dense. Four samples were nonplastic. Nine samples had liquid limits that ranged from 20 to 30 and plasticity indices that ranged from 2 to 13, with averages of 24 and 6, respectively. The maximum particle size was recovered in the test pit and was 20 inches. Horizontal hydraulic conductivity measured in 38 rising head and constant head tests ranged from 5.6×10^{-5} to 3.1×10^{-2} cm/s, and the geometric mean was 5.0×10^{-4} cm/s.

8.4 Pierre Shale

Bedrock of the Pierre Shale formation was encountered below alluvium and fill at depths that ranged from 3.7 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 along the southern portion of the detention excavation alignment at about 4 to 7 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. 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 62 and averaged 38. Recovered samples of Pierre Shale were mostly dry to moist. 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 29 Lugeons ($1.0x10^{-7}$ to $3.2x10^{-4}$ cm/s) and the geometric mean was 0.1 Lugeons ($1.8x10^{-7}$ cm/s). The unconfined compressive strength of 18 rock core samples ranged from 71 to 1,261 pounds per square inch (psi) and averaged 389 psi. The unconfined compressive strength of tested samples is generally higher along US36, which averaged 624 psi, and is generally lower toward the west side of the Project site (i.e., CU Boulder South campus), which averaged 153 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. 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 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 30-percent design submittal, the RJH Team developed a preliminary Corrected Effective Model and a Preliminary Proposed Conditions Model, which are described below.

9.2 Preliminary Corrected Effective Model

DHI developed a Preliminary Corrected Effective Model that was used as the baseline for comparisons with the Preliminary 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 Preliminary 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 Preliminary 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 Preliminary 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 Preliminary Corrected Effective Model is presented in the *Draft South Boulder Creek- MIKE FLOOD Corrected Effective Model Development Report* (DHI, 2020) that was previously submitted to the City.

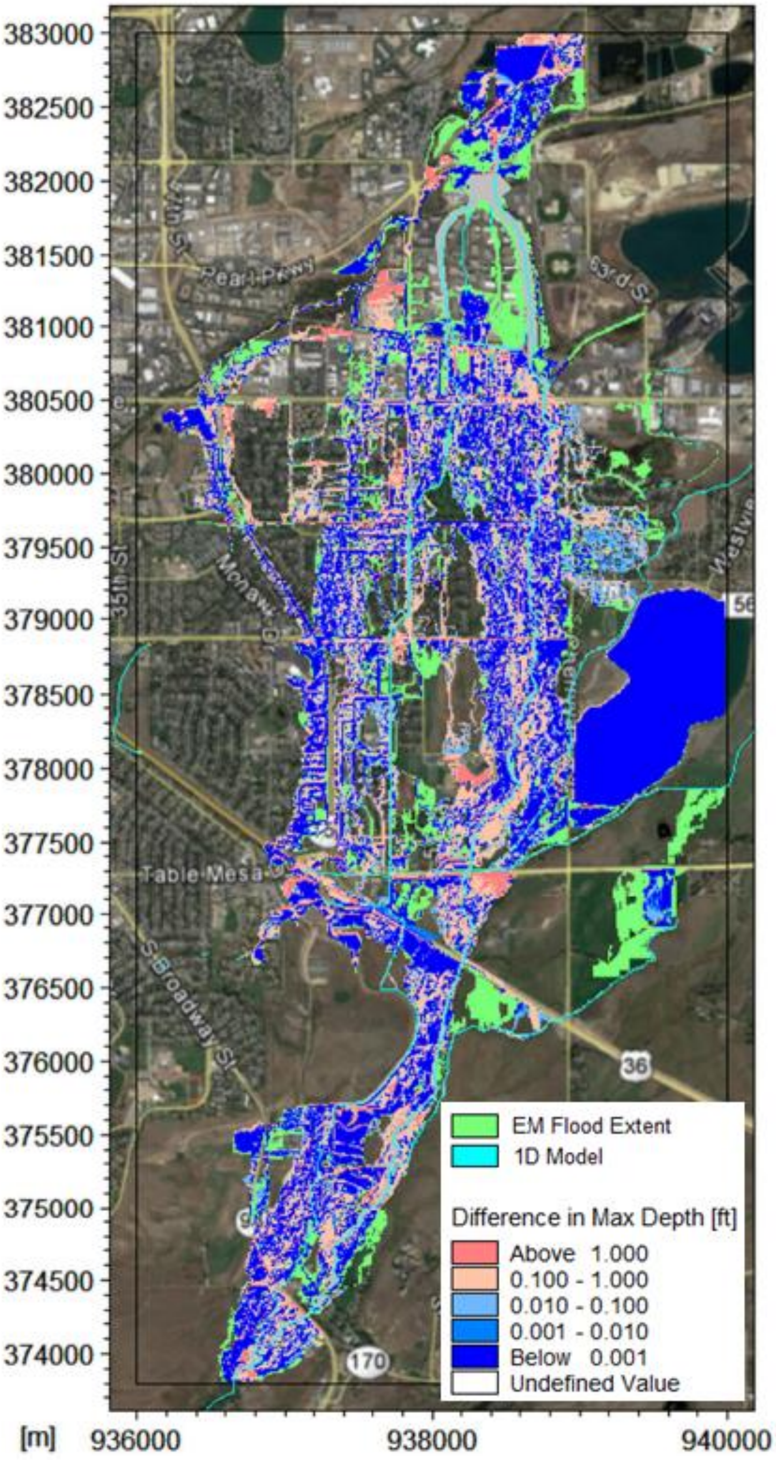


Figure 9.1: Difference in 100-year Flood Depth between Effective Model and Preliminary Corrected Effective Model

9.3 Preliminary Proposed Conditions Model

DHI developed a Preliminary Proposed Conditions Model by modifying the Preliminary Corrected Effective Model. Revisions generally included modifying the topographic terrain to reflect the dam embankment, detention excavation, spillway, and levee removal and modeling the outlet works as a culvert below US36. The Preliminary Proposed Conditions Model was developed to answer the following primary questions for the 30-percent design:

- Does the levee along the east side of the CU property need to be removed for the project to meet hydraulic and floodplain design criteria?
- What is the required hydraulic capacity of the outlet works?
- Does the wildlife crossing need to be modified to modify hydraulic capacity?
- What is the preferred storage volume and configuration for the detention excavation?

A series of model runs were performed to answer these questions. A summary of the model scenarios is presented in Table 9.1.

TABLE 9.1
HYDRAULIC MODELING SCENARIOS

Scenario	Levee Removed	Outlet Works Capacity$^{(1)}$	Wildlife Crossing Opening	Detention Excavation
Scenario 1	No	2	100%	Initial
Scenario 2	No	2	100%	Expanded
Scenario 3	Yes	2	100%	Expanded
Scenario 4	Yes	2	50%	Expanded
Scenario 5	Yes	3	100%	Expanded
Scenario 6	Yes	4	100%	Expanded
Scenario 7	Yes	1	50%	Expanded
Scenario 8	Yes	1	100%	Expanded
Scenario 9	Yes	1	100%	Refined

Note:
1. Number of 60-inch diameter pipes.

Additional information regarding key model variables is presented below.

- <u>Levee Removal</u>: For scenarios that included levee removal, the grading was developed assuming the levee is removed to native ground, which is generally the wet-side toe of the levee.
- <u>Outlet Works Capacity</u>: The maximum diameter of the outlet works will be limited to about 60-inches based on cover constraints below US36 and DCD2. For this

modeling, we considered outlet works configurations with a hydraulic capacity equal to one, two, three, and four 60-inch diameter pipes.

- Wildlife Crossing: We evaluated the impact of flows through the wildlife crossing on the performance of the Project by considering a) blocking one of the dual wildlife crossing culverts (i.e., 50-percent capacity) and b) maintaining full capacity of the culverts (i.e., 100-percent capacity).
- Detention Excavation: The initial detention excavation grading plan provided 60 ac-ft of storage at the top of the detention excavation. Based on initial modeling results, it appears that additional storage will be beneficial, so RJH developed an “expanded” grading plan that provided about 105 ac-ft at the top of the detention excavation. This grading plan was then refined to provide more desirable conditions for local drainage and vegetation establishment. The refined grading plan provides 73 ac-ft at the top of the detention excavation.

Additional information regarding the proposed conditions model is presented in Appendix B.1. A plan of difference in 100-year flood depths between the Preliminary Corrected Effective Model and the Proposed Conditions Model for Scenarios 8 and 9 is presented on Figures 9.2 and 9.3, respectively. A summary of proposed conditions model results is presented in Table 9.2. Based on the modeling results, Scenario 8, with the expanded detention excavation grading, does not increase downstream flood impacts compared to the existing condition. However, minor increases in downstream flood impacts (i.e., up to about 0.1 feet) are anticipated with the refined detention excavation grading plan (Scenario 9). Additional refinement to the detention excavation grading will be required in the next stage of design. The optimum storage volume of the detention excavation that will not cause downstream flood impacts is likely somewhere between the expanded grading (i.e., 105 ac-ft) and the refined grading (i.e., 73 ac-ft).

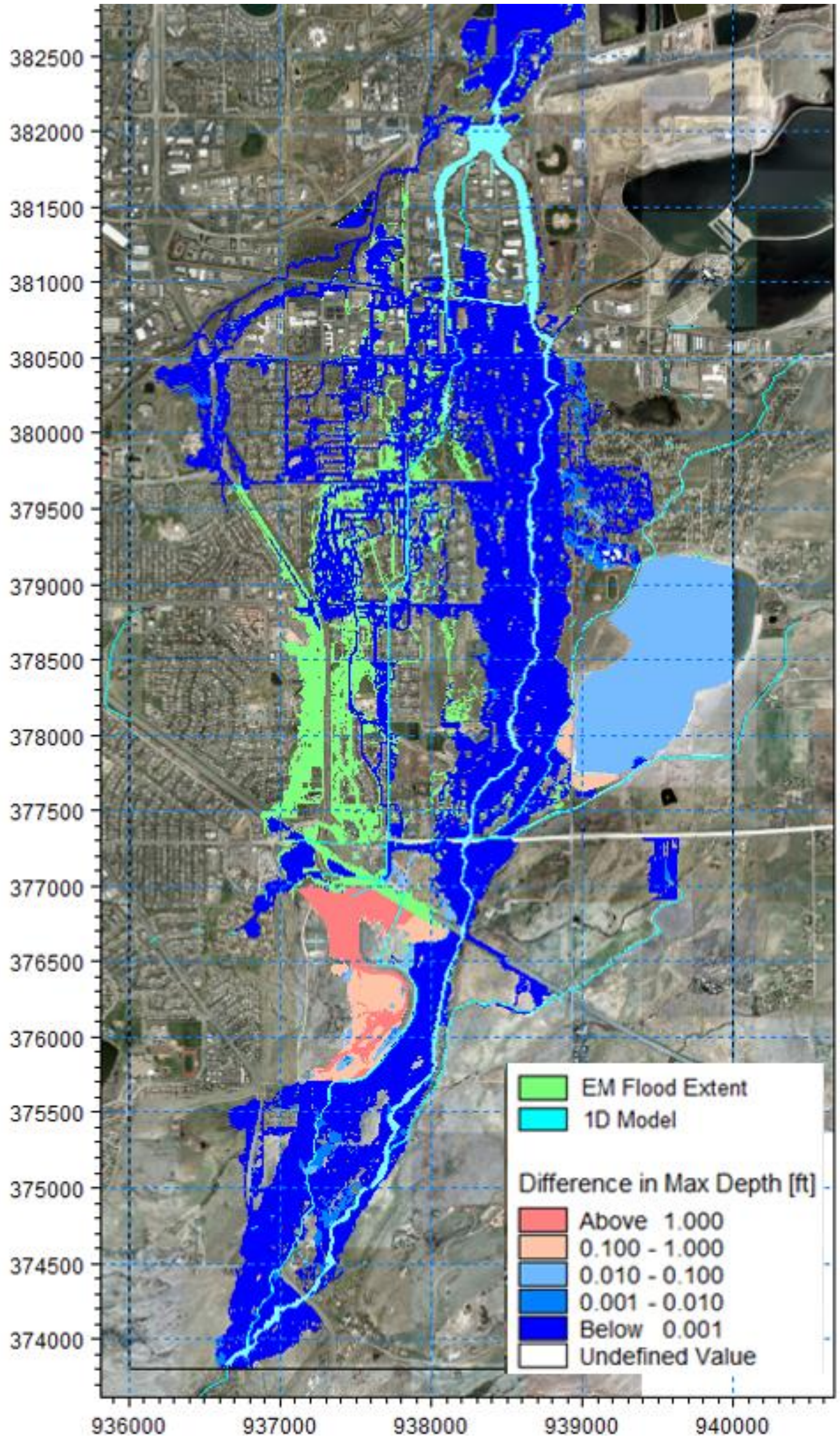


Figure 9.2: Difference in 100-year Flood Depth between Preliminary Corrected Effective Model and Proposed Conditions Model - Scenario 8

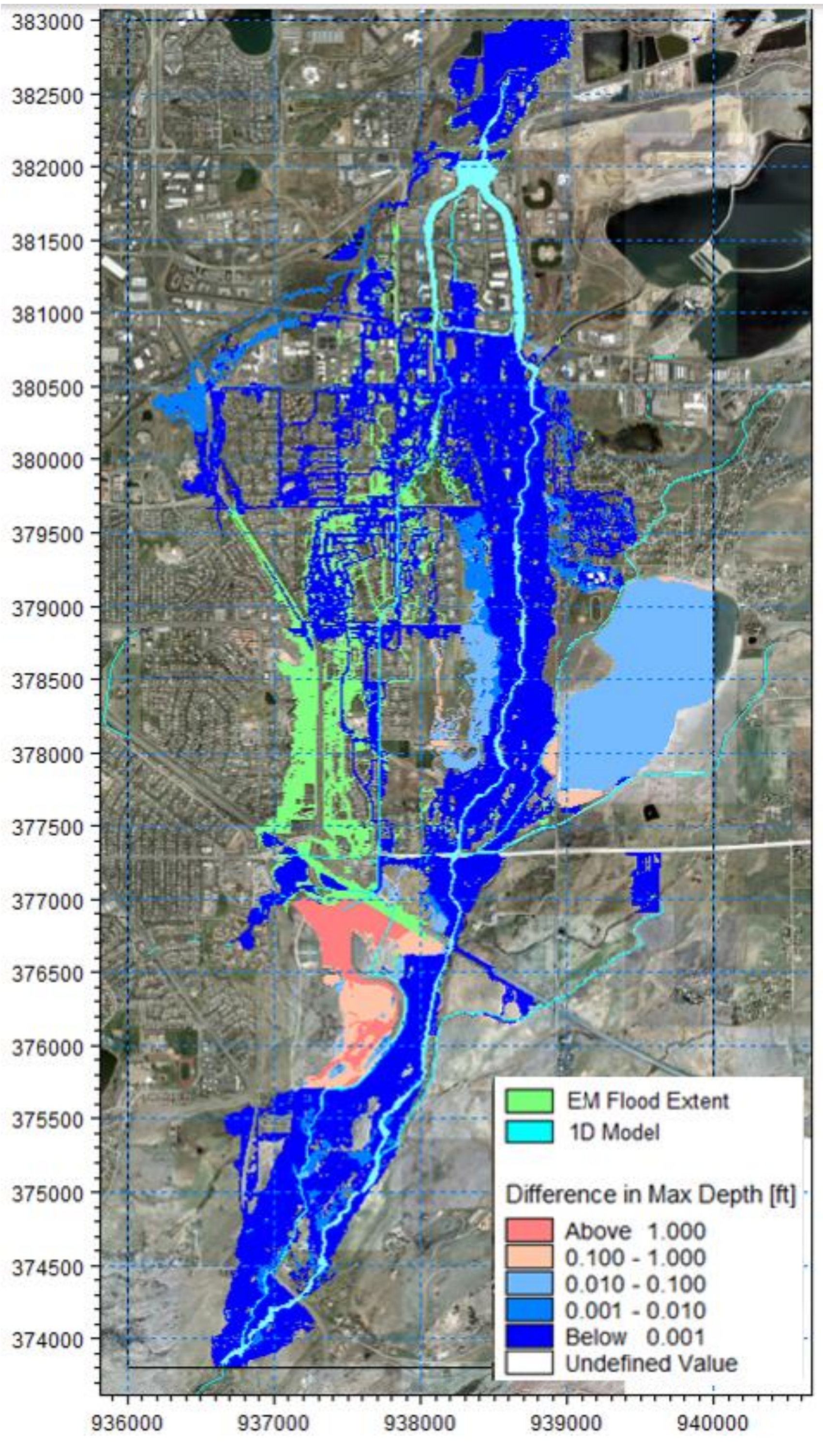


Figure 9.3: Difference in 100-year Flood Depth between Preliminary Corrected Effective Model and Proposed Conditions Model - Scenario 9

TABLE 9.2
HYDRAULIC MODELING RESULTS (100-YEAR)

Configuration	Max WSE at US36 Bridge (ft)	Max WSE in Pond (ft)	Peak Flow US36 Bridge (cfs)	Peak Flow S. Boulder Rd. (cfs)	Peak Outlet Flow (cfs)	Peak Wildlife Crossing Flow (cfs)
Existing	5361.5	N/A	3682	5477	0	810
Scenario 1	5362.0	5360.9	4128	6134	705	899
Scenario 2	5362.0	5359.5	4124	6061	677	899
Scenario 3	5361.3	5362.3	3378	5294	734	850
Scenario 4	5361.4	5362.9	3571	5088	746	421
Scenario 5	5361.3	5361.8	3376	5431	979	850
Scenario 6	5361.3	5360.9	3378	5609	1246	850
Scenario 7	5361.6	5363.4	3852	5135	351	444
Scenario 8	5361.2	5363.0	3374	5045	348	862
Scenario 9	5361.4	5363.3	3573	5293	350	865

Key takeaways from the proposed conditions modeling include:

- The levee needs to be removed for the Project to meet hydraulic and floodplain design criteria. If the levee is not removed, then a sufficient amount of water cannot be conveyed into the facility.
- The preferred outlet works configuration is a single 60-inch diameter outlet pipe.
- The hydraulic capacity of the wildlife crossing does not need to be modified.
- The expanded detention excavation grading plan (105 ac-ft) provides sufficient storage to meet hydraulic and floodplain design criteria. The refined detention excavation grading plan (73 ac-ft) will cause small water surface rises up to 0.1 feet at some areas. The optimum storage volume of the detention excavation is likely somewhere between the expanded grading (i.e., 105 ac-ft) and the refined grading (i.e., 73 ac-ft).

We conservatively selected a 100-year water surface elevation of El. 5363.8 to use for setting the top of the spillway wall and dam crest.

As the design progresses, we anticipate continuing to refine the Preliminary Proposed Conditions Model. This model will serve as the basis for future regulatory floodplain permitting activities. We also plan to develop a “local” proposed conditions model. The local model will be a truncated model extending from about Highway 93 to Baseline Road. The local model will allow for smaller grid sizes and provide more detailed hydraulic information at key locations to inform the selection of design features. For example, we will use the local model to evaluate velocities along the upstream face of the spillway wall to identify if erosion protection is required.

9.4 Alternate Existing Conditions Model

The Effective Model in MIKE FLOOD has historically been challenging for the City to use as a regulatory model because MIKE FLOOD is a proprietary software that is not widely used in industry for regulatory floodplain models. The City is interested in the possibility of using the USACE HEC-RAS software program for future FEMA floodplain updates to SBC. HEC-RAS is a public domain hydraulic modeling software program that was been widely adopted by municipalities, regulators, consultants, developers, and floodplain managers.

RESPEC developed an Alternate Existing Conditions Model using HEC-RAS. This model was developed to facilitate comparison to the MIKE FLOOD Preliminary Corrected Effective Model discussed above. This comparison will be used to inform the City's decision of whether to transition the Project hydraulic modeling and permitting to HEC-RAS.

The Alternate Existing Conditions Model was developed using as much information from the Preliminary Corrected Effective Model as reasonably possible for consistency. Model input obtained from the MIKE FLOOD model included Manning's n grid, digital elevation model, hydraulic structures information, and inflow hydrographs and locations.

While both MIKE FLOOD and HEC-RAS are based on hydraulic principles of conservation of mass and momentum, there are differences in computational algorithms, equations, and approaches that will result in differences in flow depths and extents between the models. A significant difference between the two models is that the Alternate Existing Conditions Model was developed using a topographic terrain grid cell size that ranged from 25-feet by 25-feet upstream of the US36 crossing to 150-feet by 150-feet in the upstream portions of the model domain, while the MIKE FLOOD model used grid cell sizes that ranged from 1 to 2 meters (3.28 to 6.56-feet) around US36 and 4 meters (13.12 feet) elsewhere. A significantly smaller cell size can be used for the MIKE FLOOD model because DHI has computers with processing capabilities that are more powerful than those utilized by most consultants.

Based on a comparison of the Alternate Corrected Effective Model and Preliminary Corrected Effective Model results, overall characteristics for the 100-year flood are similar. However, there are significant differences in flow depths and extents at specific locations. The mean difference in water depths between the two models is 0.00048 feet with a standard deviation of 0.56 feet. Differences in water surface elevation (flood depth) between models are shown on

Figure 9.4. There is an average difference of 1.1 feet between the models overtopping US36 into the West Valley with MIKE FLOOD having higher water depths. The discrepancy is likely the result differences in cell size in this hydraulically complex area.

Differences in flow rates between the models are presented in Table 9.3. The largest differences occur at culverts, and calibration of culvert hydraulics could be performed to decrease the discrepancy between the two models.

TABLE 9.3
PEAK FLOW RATE DIFFERENCES

Location	HEC-RAS (cfs)	MIKE FLOOD (cfs)	Difference (%)
Approach Highway 93	7,125	6,643	6.8
Approaching US36	7,370	7,159	2.9
US36 Bridge	3,906	3,811	2.4
US36 Wildlife Crossing	751	856	-14.0
US36 Flow Split	2,795	2,704	3.2
US36 Overtopping	2,423	2,338	3.5
South Boulder Road	2,899	2,734	5.7

In RESPEC’s opinion, the floodplain delineation and resulting 100-year water surface elevations developed using HEC-RAS compare reasonably to those developed using MIKE FLOOD, and HEC-RAS will be a reasonable model to use for future floodplain modeling of SBC.

Additional information regarding the Alternate Corrected Effective Model is presented in Appendix B.2.

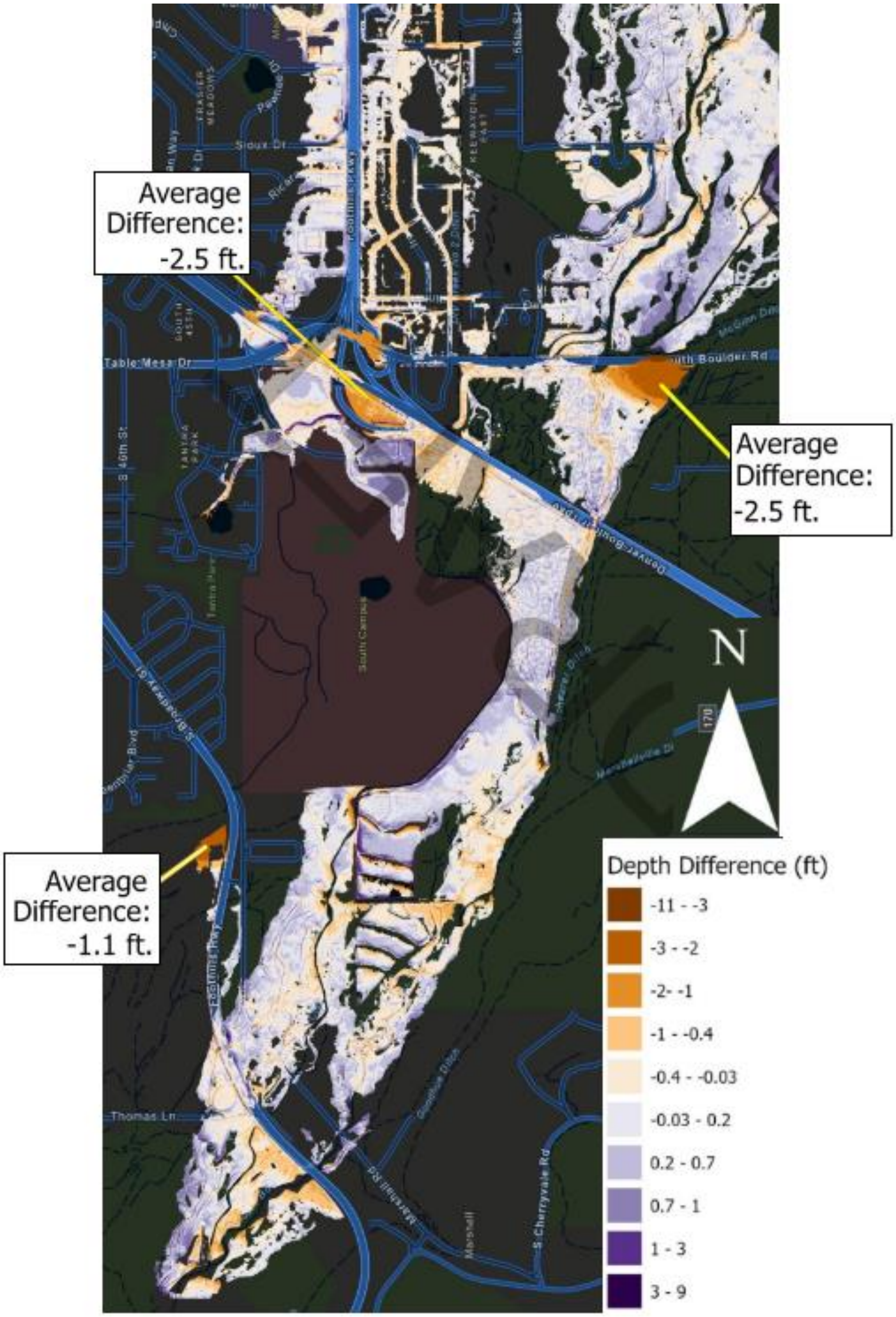


Figure 9.4: Difference in 100-year Flood Depth between Alternate Corrected Effective Model (HEC-RAS) and Preliminary Corrected Effective Model (MIKE FLOOD)

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 west (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 reduce seepage losses. 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 4H:1V to reduce long-term maintenance and provide improved aesthetics. The embankment crest will be at elevation (El.) 5371.2. This provides 2 feet 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 will likely 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 5 feet 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 preliminary design of the dam embankment. These analyses included an evaluation of wave runup and freeboard, material properties, slope stability, seepage, and seismic deformation and 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:

- IDF Pool (El. 5370.8) plus runup and setup from a wind velocity exceeded 10-percent of the time, which is 15 mile-per-hour (mph).
- 100-Year Flood Pool (El. 5363.8) plus runup and setup from a 100- mph wind velocity.

- Earthfill embankment at a 4 horizontal to 1 vertical (H:V) slope covered with native grasses.

Based on these analyses, the computed wave runup was less than 2 feet of freeboard selected by RJH.

A spillway routing analysis was performed using the weir equation to identify the IDF pool. The IDF pool was computed to be at El. 5369.2 for a 2,200-foot-long spillway.

For preliminary design, RJH selected a dam crest El. 5371.2 based on the spillway routing analysis plus 2 feet of freeboard to be conservative and account for changes due to hydraulic modeling. We anticipate the design elevation of the dam crest to be modified during future design phases based on additional hydraulic modeling.

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 for preliminary design, 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.1.

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).
- 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.

Evaluating steady state conditions is conservative because steady state conditions are not anticipated to develop during short-term reservoir impoundments. 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 10.1.

TABLE 10.1
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)	1.5×10^{-3}	Not Applicable
Steady State - Full Reservoir	Low-permeable foundation soils and bank-full conditions in Viele Channel	<0.1(4)	0.04	3.6×10^{-2}

Note:
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 10.2.

TABLE 10.2
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.2.

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 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 60 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 2175-feet long and will connect to the earthfill embankment at the west (left) end and to the US36 embankment at the east (right) end.

The alignment of the spillway for the 30-percent design was selected to avoid impacts to existing utilities within the CDOT ROW and facilitate construction. The location of the utilities within the CDOT ROW varies along the US36 corridor; however, near the west end of the spillway, the utilities are located near the southern edge of the CDOT ROW. The centerline of the spillway will need to be about 45 feet from the CDOT ROW at this location to maintain the existing utilities and provide reasonable room for construction of below-ground portions of the spillway and groundwater conveyance system. For the 30-percent design, we maintained a consistent offset of 45 feet from the centerline of the spillway to the CDOT ROW. The offset distance could be reduced where feasible to reduce impacts to OSMP based on the locations of existing utilities within the CDOT ROW.

11.2 Spillway Wall

The spillway wall will consist of a vertical, reinforced concrete wall that varies in height above final grade from about 6 feet to 10 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 spillway wall will be set at El. 5371.2 at the connection to the embankment dam to prevent overtopping during the PMF; and at El. 5365.8 at the connection to US36 to reduce the frequency of overtopping during extreme events.

Reservoir and spillway routing for the PMF was performed using the MIKE FLOOD model described in Section 9. During the PMF, flows will travel along the upstream face of the spillway wall prior to overtopping the wall. Flows will overtop the wall non-uniformly. The spillway 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. Tailwater will eventually submerge the spillway wall during the PMF.

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). Based on the results of this model, a 1-foot-thick wall with appropriate steel reinforcement will generally be adequate for the spillway. For the 30-percent design, the reinforcement pattern for the spillway wall was #7 bars each face and both ways at 12 inches. 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 wall evaluation is presented in a technical memorandum in Appendix D.1.

Various architectural treatments could be considered to the spillway wall for improved aesthetics in future stages of design. Some options include concrete staining or stamping, architectural trellises to facilitate plant growth, curvilinear alignment, etc.

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 and to provide a seepage barrier to restrict flows through the coarse-grained alluvium during times of flood detention. 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.

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.

Additional information regarding the spillway foundation evaluation is presented in a technical memorandum in Appendix D.1.

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 will be designed to provide sufficient structural support for the spillway wall; however, a seepage barrier to bedrock will not be included. A shallow foundation will be beneficial because it will allow groundwater to flow through the alluvium beneath the spillway during normal (non-flood) conditions. However, a shallow foundation will also allow high seepage rates through the spillway foundation during flood loads, which will 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.

In our opinion, a shallow foundation is technically feasible and will likely be less expensive, but inherently is slightly higher risk than a deep foundation with full seepage cutoff. If the City elects to move forward with the shallow foundation, additional analyses are needed to confirm that the increase in risk falls below tolerable risk levels identified by the City and SEO.

11.4 Spillway Apron

The spillway will discharge to the area between the spillway wall and the US36 roadway embankment. This area consists of both 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 preliminary design of the spillway apron and have endeavored a conservative application of this approach.

Based on this evaluation, we conservatively selected apron lengths that vary along the lengths of the spillway. The apron length is 12 feet at the east (right) end of the spillway and increases to 18 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 evaluation is presented in a technical memorandum 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, and 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 the PMF event. 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 PFM that could occur during extreme loading events and potentially compromise the spillway abutment:

- <u>PFM #1</u>: 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.
- <u>PFM #2</u>: 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.
- <u>PFM #3</u>: 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.
- <u>PFM #4</u>: 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 5.07 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.3.

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.

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. 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 be located 11 feet upstream of the spillway and will consist of a 4-foot wide and 4-foot-deep trench with a 10-inch slotted PVC pipe surrounded by filter material. The filter material will be filter compatible with the surrounding alluvium and also with the pipe slot widths. The hydraulic conductivity of the collection trench is about 91 times greater than the alluvium. The trench will extend from about 2 feet above to 2 feet below the seasonally low groundwater level. The invert of the pipe was set to be at about 10 inches below the seasonally low groundwater levels so that the historic seasonally low groundwater levels can be maintained. Extending the trench deeper than 2 feet below the seasonally low groundwater level would provide added reliability to the system and the benefit-cost should be evaluated during future design phases. The top of the trench will coincide with the surface of the spillway's temporary working platform, which will be about 2 feet above the seasonally low groundwater level.

- Distribution trench on the downstream (north) side of the spillway. 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 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 that are 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 eight connector pipes will be spaced at approximately 260 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, and therefore the proposed configuration of connector pipes is appropriate for conveying flows through the spillway 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 intermittent 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 20 feet long and spaced about every 260 feet 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 260 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 the manholes in the collection trench at the upstream end of the connector pipe and at the discharge end of the collection 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 future phases 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. 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 Preliminary Design Modeling

12.5.2.1 General

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

RJH modified the Baseline Model (RJH, 2021) slightly prior to beginning preliminary design modeling. We decreased the hydraulic conductivity of bedrock to 2x10^{-7} cm/s throughout the model based on packer tests performed during our Phase II geotechnical investigation (RJH, 2022b). This value is about 2 to 3 orders of magnitude lower than the hydraulic conductivity used in the Baseline Model for weathered and unweathered bedrock, respectively, and in our opinion, is reasonable based on site data and our experience with the Pierre Shale.

RJH then used the modified Baseline Model (i.e., Pre-Project Model) to simulate the following scenarios and support the preliminary design:

- <u>No Conveyance System Scenario</u>. We simulated proposed facilities to evaluate groundwater effects if a groundwater conveyance system is not installed. We used horizontal flow barriers (HFBs) to simulate the effects of the barrier walls along the embankment and around the detention excavation and the secant pile wall along the spillway. We increased the hydraulic conductivity of cells within the detention excavation area to simulate removal of soil from this portion of the site. We also added a drain boundary condition within the detention excavation area to simulate how water that accumulates in this area can flow out through the uncontrolled outlet works conduit. The No Conveyance System scenario groundwater levels were predicted to be up to about 9 feet higher than Pre-Project levels upstream of the spillway and up to about 9 feet lower than Pre-Project levels downstream of the spillway. No Conveyance System scenario groundwater levels were also predicted to be about 3 feet higher than Pre-Project levels downstream of the embankment dam near the west side of the CU Boulder South campus. These predicted groundwater effects are not acceptable, and the model results demonstrate that some type of groundwater conveyance system is required to maintain pre-Project groundwater conditions at the Site.
- <u>Proposed Conditions Scenario</u>. We added proposed conveyance facilities into the model to mitigate the groundwater effects described above. We used a drain boundary condition along the west side of the model to simulate the effects of the embankment toe drain pipe. We used highly permeable cells to simulate the effects of a collection trench, distribution trench, and connector pipes along the spillway. We iteratively adjusted the configuration of the proposed facilities until the conveyance facilities appropriately mitigated the groundwater effects. The final Proposed Condition Scenario developed during the preliminary design modeling consisted of segmented collector and distribution trenches. Each segment was modeled as 300 feet long and 300-foot gaps of natural alluvium were simulated between each segment. One connector pipe was simulated to connect each segment of the collector trench to each segment of the distribution trench. Segmented

collector and distribution trenches were required to prevent excessive flows towards the northwest along continuous collector and distribution trenches. The simulated Proposed Conditions configuration is shown on Figure 12.1.

12.5.2.2 Proposed Conditions Scenario - Head Results

Although the groundwater model was developed using monthly stress periods to represent a complete hydrogeologic season, for preliminary design RJH only evaluated head conditions during one typical non-irrigation month when groundwater is generally low (November) and one typical irrigation month when groundwater in irrigated areas is generally high (June).

The simulated changes to the groundwater levels between the Pre-Project and Proposed Conditions scenarios for November and June are shown on Figures 12.2 and 12.3, respectively. Blue shaded areas on Figures 12.2 and 12.3 represent areas of simulated groundwater mounding (e.g., Proposed Conditions groundwater levels are predicted to be higher than Pre-Project levels), and red shaded areas represent areas of simulated groundwater decline (Proposed Conditions groundwater levels are predicted to be lower than Pre-Project levels). Darker colors represent greater mounding or decline. Areas that are within +/-0.10 foot of Pre-Project groundwater levels are not shaded to improve clarity. The numerical magnitudes of mounding or decline are also shown by callouts at selected locations on the plan figures.

The head results on Figures 12.2 and 12.3 show the following, which in our opinion demonstrates that groundwater conveyance facilities will adequately maintain Pre-Project groundwater levels after construction of Project facilities:

- Along the west side of the site, the predicted groundwater impacts are typically limited to less than +/- 1 foot and generally occur along the embankment dam alignment on CU property.
- Upstream and downstream of the spillway, groundwater levels are generally predicted to change by less than +/-0.25 feet throughout the OSMP North and South fields.
- Adjacent to the spillway alignment and north of the detention excavation, the groundwater level is predicted to change more than +/-0.25 feet and is predicted to decline up to about 1 to 2.6 feet in localized areas; however, these changes are acceptable in our opinion because they typically occur within the CDOT ROW and developed areas where changes in groundwater level are not anticipated to cause adverse effects.

12.5.2.3 Proposed Conditions Scenario - Flow Results

We evaluated the amount of groundwater flow that is predicted to occur beneath US36 for the Proposed Conditions Scenario. Simulated flows were extracted from the model using the same techniques as described in the Baseline Groundwater Model Report (RJH, 2021). The predicted Proposed Conditions flow beneath US36 simulated minor redistribution and decrease of flows ranging from 0.2 to 14 percent; however, the total flows beneath US36 were within about 2 percent of the Pre-Project flow rates for every month of the model simulation, which in our opinion is negligible and within tolerable limits.

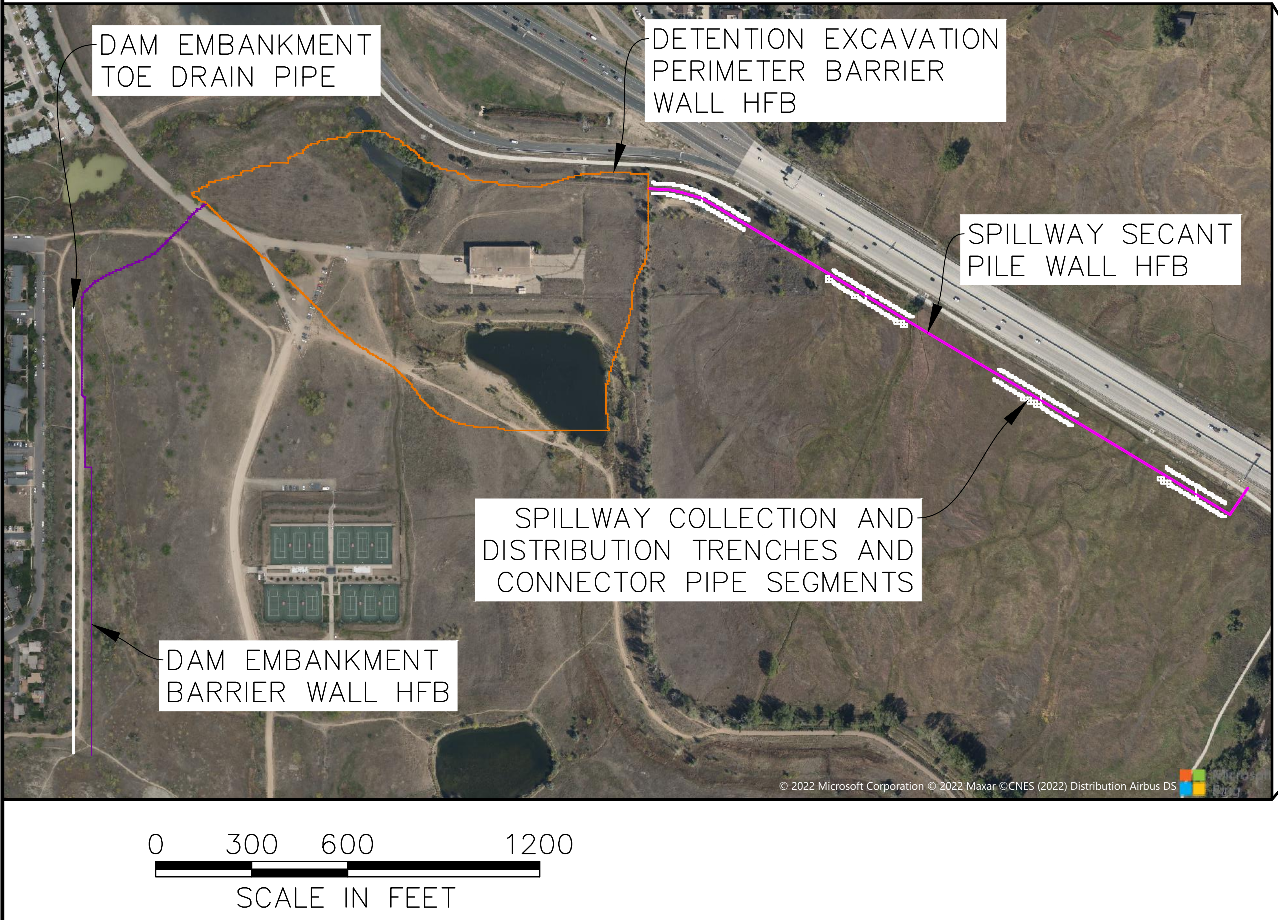
The highest flow rate modeled through any of the proposed connector pipes was about 14 gallons per minute (gpm). A 22-foot-long 10-inch PVC pipe can convey 14 gpm under negligible head (much less than 0.1 foot), and therefore the connector pipes are anticipated to have adequate capacity for conveying flows across the spillway alignment.

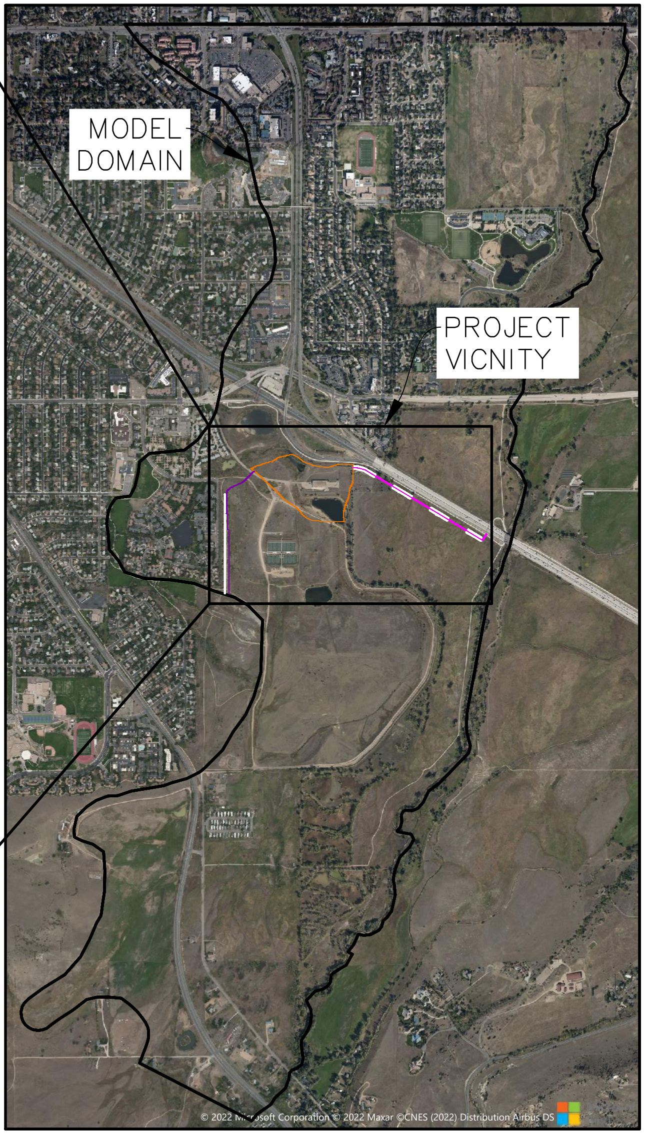
12.5.3 Groundwater Modeling - Conclusions

We conclude the following from the results of groundwater modeling:

- The Baseline Model provides a reasonable approximation of the existing groundwater system in the Project vicinity and is suitable for evaluating impacts of Project components and supporting design of mitigation features.
- Project components are anticipated to create unacceptable changes to existing groundwater conditions if mitigation features are not installed.
- The Proposed Conditions Scenario illustrated on Figure 12.1 generally maintains Pre-Project groundwater levels and is an acceptable design solution. Groundwater level impacts are predominantly limited to areas on CU Boulder South campus, CDOT ROW, and developed areas immediately adjacent to Project facilities. The modeled scenario is useful for supporting the preliminary design of groundwater mitigation systems.
- Collection and distribution trenches along the spillway will contain periodic backfill plugs and gates, and multiple connector pipes that will facilitate operational flexibility for regulating the distribution of flow through the system and restrict groundwater from flowing along the lengths of the trenches.
- Components of the groundwater conveyance system have ample hydraulic capacity. A 10-inch pipe was selected to accommodate access for long-term inspection and maintenance.

- The preliminary design of the groundwater conveyance system is reliable and suitable for 30 percent design. Configurations of Project components will be refined as the design progresses. Also, in future stages of design we will use the groundwater model to confirm that the design solution performs acceptably under a range of operating conditions, including variability in aquifer properties, precipitation, and evaporation rates, etc.
- Additional monitoring wells will need to be installed near proposed facilities to collect existing (i.e., pre-construction) groundwater data and allow for monitoring the effectiveness of conveyance facilities.





NOT TO SCALE

NOTES:

1. REPRODUCE IN COLOR

PRELIMINARY
NOT FOR CONSTRUCTION

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SOUTH BOULDER CREEK
REGIONAL DETENTION
PRELIMINARY DESIGN REPORT

SIMULATED PROPOSED CONDITIONS SCENARIO

PROJECT NO. 16134

July 2022

Figure 12.1

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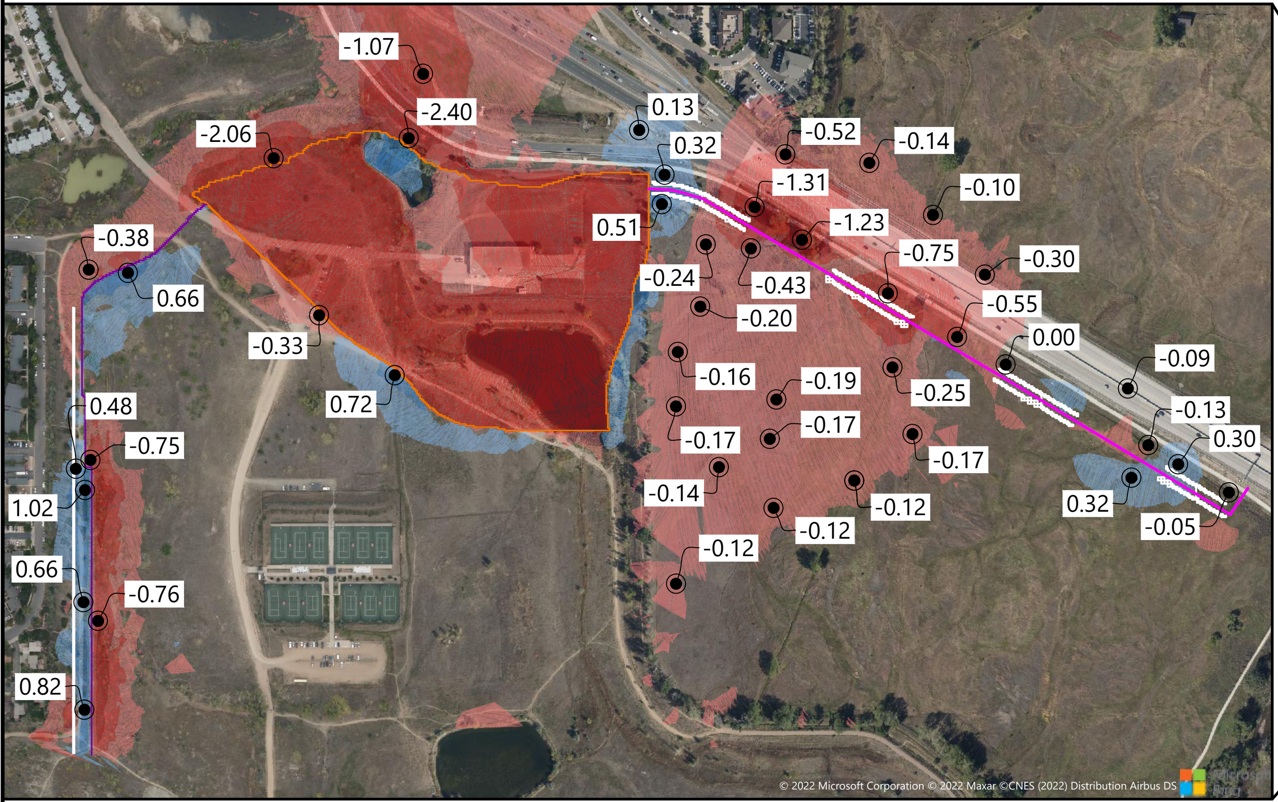

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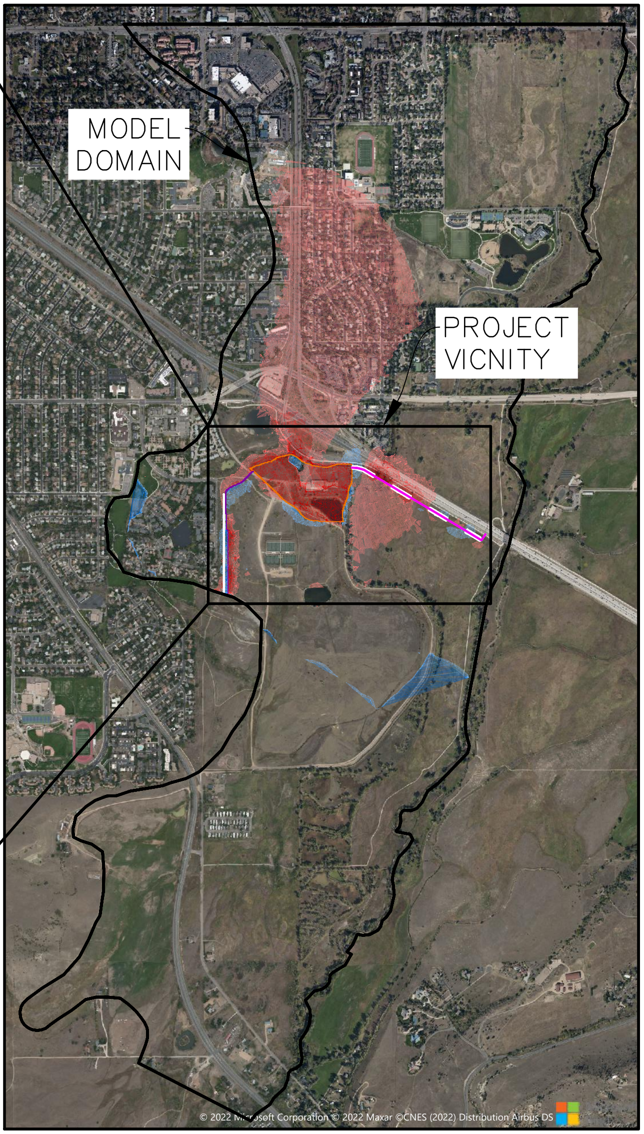

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Elevations Table

Number	Minimum Elevation	Maximum 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		🟦

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PRELIMINARY DESIGN REPORT

PROJECT NO. 16134

PROPOSED CONDITIONS
SCENARIO HEAD RESULTS
- NOVEMBER

July 2022

Figure 12.2

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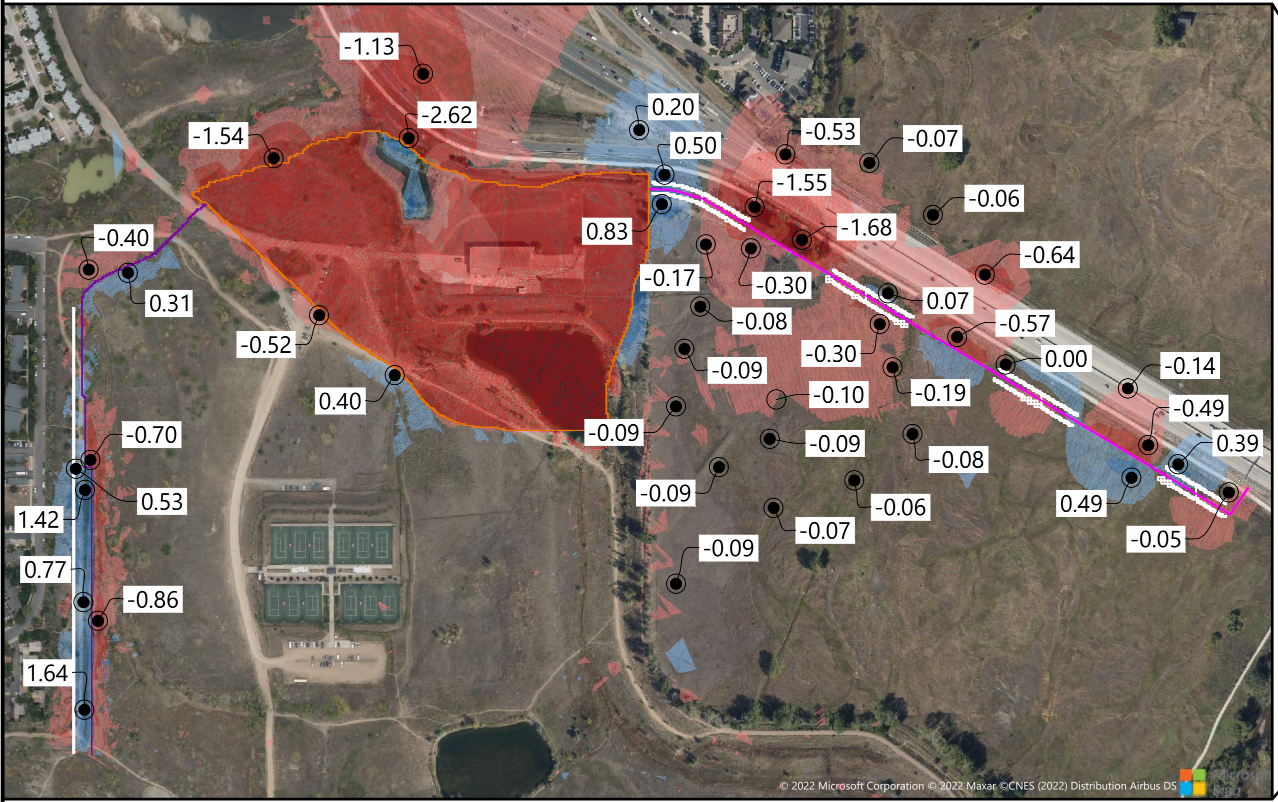

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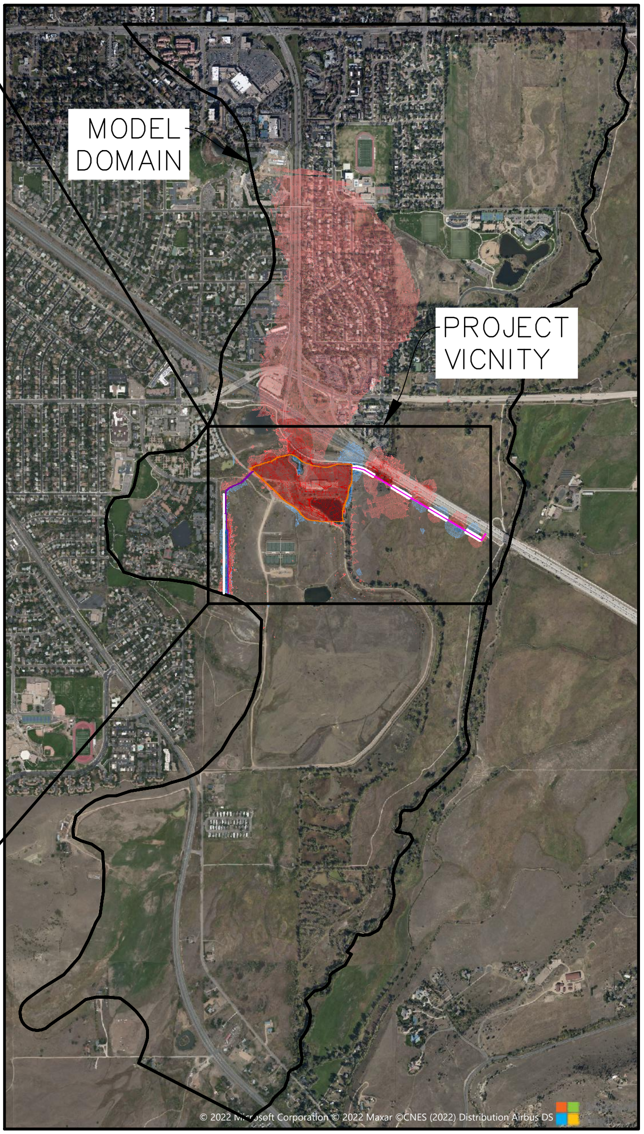

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Elevations Table

Number	Minimum Elevation	Maximum 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:

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REGIONAL DETENTION
PRELIMINARY DESIGN REPORT

PROPOSED CONDITIONS SCENARIO HEAD RESULTS - JUNE

PROJECT NO. 16134

July 2022

Figure 12.3

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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 inlet will be set at El. 5343.0, which is the bottom of the detention excavation. The outlet will be set at El. 5340.0, which is the invert elevation of Viele Channel north of US36. Tunneling will be required for the portion of the outlet works conduit below US36.

RJH performed hydraulic modeling to identify a preferred size and configuration for the outlet works (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 350 cfs through the outlet works during the 100-year flood event. The portion of the reservoir that does not freely drain to SBC will drain through the outlet works in approximately 10 hours, which meets both SEO and water rights criteria.

13.2 Intake Structure

The intake structure will consist of a 14.5-foot-high, reinforced concrete 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, sides, and top of the structure will include openings covered by trashracks. The intake structure will have interior dimensions of 7-feet by 10.5-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 and sides of the structure will be vertical. The trashrack along the top of the structure will be sloping. 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.

The intake structure will generally be exposed because it will be located at the upstream toe. This may increase the likelihood of vandalism and safety risks from pedestrians climbing the structure. Another option will be to embed the intake structure in the detention excavation slope and replace the vertical trashracks with a sloping trashrack. This will significantly reduce the visibility and exposure of the structure.

13.3 Conduit

The outlet works conduit will consist of a 730-foot long 60-inch diameter pipe. Approximately 480 feet of pipe will be installed in an open excavation and encased in reinforced concrete and tunneling will be required to install approximately 250 linear feet of pipe below US36.

Lithos performed a tunnel feasibility evaluation. A tunnel feasibility technical memorandum was developed and is presented in Appendix F. In general, Lithos did not identify any fatal flaws that will preclude construction of a tunnel. Lithos evaluated the feasibility, advantages, and disadvantages of several tunneling methods including pipe ramming, microtunnel boring machine (MTBM), and shielded tunneling:

- Pipe Ramming: This technique involves ramming a steel casing across the alignment prior to excavating material from within the casing. This method has the lowest risk of overexcavation and settlement than other feasible methods and is typically favored by other stakeholders. This method is also better able to extend through cobbles and boulders than other methods, and the risk for abandonment is lower. The practical maximum length for this technique is about 300 to 320 feet. This method may cause vibration concerns, and specific vibration monitoring and instrumentation will be required to manage or prevent claims.
- MTBM: This technique uses a pressurized rotating cutting head to excavate the ground. Ground support is provided by a jacked pipe or with erected supports and with a pressurized bentonite-slurry at the excavation face to counter earth pressures. MTMB has the ability to displace or break apart larger boulders at the cutting head; however, nested boulders and cobbles can be challenging. If the MTMB fails to cut through boulders or cobbles, a rescue shaft or dewatered tunnel may be required to salvage the machine. This technique is often significantly more expensive and schedule intensive and requires a much larger staging area than other techniques.
- Shielded Tunneling: This technique involves excavation with hand-tools and/or a hydraulic excavator arm at the head of the tunnel. Ground support is provided by a jacked pipe. Full alignment dewatering may be difficult to achieve with this technique, which will limit the ability to have a stable excavation face for this method. Even closely spaced well-points and deep wells within bedrock may only prove marginally effective at lowering the groundwater table sufficiently to have a stable excavation face.

Lithos identified that the pipe ramming technique is the preferred tunneling method with the least overall project risk. Pipe ramming will be used to install a 96-inch diameter steel casing pipe beneath US36. 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, and will be about 250-feet long, which is an appropriate distance for pipe ramming. A 60-inch steel carrier pipe will be installed within the casing pipe, and the annulus between the carrier pipe and casing pipe will be grouted. The portion of conduit upstream of US36 will consist of a concrete encased 60-inch steel pipe that is installed with an open excavation rather than a tunnel to accommodate the pipe ramming length limitations. A 30-degree bend will be required in the conduit between the portion of the conduit installed in the open excavation and the portion that is tunneled.

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 OSMP property to the north of DCD2. The receiving shaft will also be used to provide a supported excavation for construction of the outlet structure.

13.4 Outlet Structure and Discharge Channel

The outlet structure will consist of a reinforced concrete, baffled outlet structure at the downstream end of the conduit. The outlet structure was sized using the Reclamation Engineering Monograph No. 25 Hydraulic Design of Stilling Basins and Energy Dissipators (Reclamation, 1984). The required basin width for a 60-inch diameter pipe is 17 feet. These dimensions are based on the proportional relationships from model studies performed by Reclamation.

Riprap will be installed in Viele Channel in the vicinity of the outlet structure for erosion protection. The hydraulic modeling described in Section 9 was performed assuming Viele Channel downstream of US36 consists of significant shrub and tree growth. Some of this vegetation will need to be removed to install the outlet structure and riprap. As the Project advances, we will evaluate whether additional vegetation removal and maintenance in Viele Channel between US36 and South Boulder Road will provide hydraulic benefits to the Project. Removing existing vegetation from the channel would increase the hydraulic capacity, and this may provide some downstream flood benefits.

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

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 *Hydrologic Basin Response Parameter Estimation Guidelines* (SEO, 2008) 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 6,033 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 5.0.7. 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 6,030 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 130-foot-long segment of the downstream slope where the velocities will be between about 2 to 4 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.

14.3 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 that facilitate flood irrigation of OSMP property south of US36.

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

The capacity of the ditch varies significantly by location and ranges from 20 to 375 cfs. Based on information from the City of Boulder Water Resources Department, the decreed water right in DCD2 at the headgate is 44 cfs, which will be verified with the DCD2 Company during future design phases.

The spillway alignment intersects DCD2 approximately 75 feet upstream of US36. The Project will need to be designed to facilitate conveyance of the decreed flow rate in the ditch and maintain the ability to flood irrigate OSMP property. The portion of DCD2 upstream of US36 is currently, and will continue to be, inundated during large flood events in SBC. However, the depth and duration of inundation will be higher than existing conditions.

For the 30-percent design, modification to DCD2 will consist of extending the upstream face of the culvert through the spillway wall. This will accommodate future operations in DCD2 without obstruction from the Project. Based on hydraulic modeling performed by the Project team (see Section 9), the Project will increase the 100-year water surface elevation at the US36 culvert by approximately 2.5 feet (1.1 psi) and will increase the 100-year flow through the culvert by 70 cfs (from about 290 to 360 cfs). These increases in pressure and flow through the culvert should be acceptable for an RCBC, but this should be confirmed with additional analyses.

Additional information regarding DCD2 is presented in a technical memorandum in Appendix G.3. Additional coordination with DCD2 Company will be performed in the next stage of design to discuss modifications to their facilities.

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 to the 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 75 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 increase the 100-year flow through the wildlife crossing by 55 cfs (from about 810 to 865 cfs). These increases in flow through the culvert should be acceptable for an RCBC, but this should be confirmed with additional analyses.

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 65 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. One option to address this issue will be to extend the culverts to the face of the spillway wall. However, this will not accommodate drainage of the area between the US36 embankment and the spillway wall. To maintain drainage of this area and to convey routine irrigation flows, we propose to install small openings in the spillway wall directly upstream of each culvert except for the wildlife crossing and DCD2, which will be extended through the spillway wall. The small openings will be sized to convey routine irrigation flows and flows from routine precipitation and snowmelt events but will not convey an excessive amount of water during a flood event on SBC that will result in overtopping of US36 and subsequent flooding of the West Valley. Additional hydraulic modeling will be performed in the next stage of design to size these openings.

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 the detention excavation 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

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 needed in the next stage of design to reduce these impacts.

Since the excavation will be below existing groundwater elevations, a barrier wall is needed to keep the excavation from collecting groundwater, which will render it ineffective for detention storage. 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. The barrier will consist of a 3-foot-wide soil-bentonite barrier wall that will extend 5 feet into the underlying Pierre Shale bedrock.

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. We will obtain a waiver from the SEO for this variance.

As described in Section 9.3, the detention excavation grading plan presented in the 30-percent design results in localized increases (up to 0.1 foot) in the 100-year water surface elevation downstream of the site. The current detention excavation grading will be modified in the next phase of design to meet the design criteria. Currently the lowest elevation in the detention area is about 1 foot above the invert elevation of the outlet works inlet structure. We expect that the additional needed storage can be attained by lowering the bottom of the detention excavation area and by adjusting the slopes within the excavation.

15.2.2 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).

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 will 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 20H:1V 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.

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 H.

USACE will also require the development of a Biological Assessment and Cultural Resources Class III Report. The extent of ecological restoration will need to be identified prior to the completion of these reports, so this area can be included in the biological and cultural evaluations.

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. 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

Wetland mitigation will be required to comply with the City's Stream, Wetland, and Water Body Regulations, which requires mitigation at a ratio between 2:1 and 2.5:1 for wetland impacts, depending on the quality of the wetland. Wetland mitigation will also be required for the CWA Section 404 permit if impacted wetland areas exceed 1/10 acre. Mitigation of ULTO and PMJM habitat will also likely be required by USACE. Consultation between USACE and USFWS will be performed to discuss mitigation strategies.

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:

- Removal of the existing levee embankment to reconnect the OS-O area to the SBC floodplain and riparian corridor.
- Development of new wetlands while maintaining current wetlands in the OS-O area.
- Development of new T&E habitat while sustaining current T&E habitat conditions in the OS-O area.

The environmental mitigation and ecological restoration concepts will be identified in the next phase of design.

SECTION 17 - OPINION OF PROBABLE CONSTRUCTION COSTS

17.1 General

The RJH Team developed an OPPC based on the preliminary design concepts presented in this report. This OPPC is considered a Class 3 estimate as defined by the ASTM E2516-11. This class designation is used when the design is between 10 percent and 40 percent complete. The reliability of a Class 3 estimate according to ASTM is between minus 15 to plus 20 percent. Costs are presented in April 2022 dollars.

Cost opinions were developed by estimating quantities of elements of the work based on the preliminary-level design drawings and unit costs were developed from the following sources:

- Published and non-published bid price data for similar work.
- Manufacturer’s, suppliers’, and contractor’s budgetary price quotes.
- Our internal database, previous experience, and judgement.
- R.S. Means Heavy Construction Cost Data for 2021.

The “Base Construction Subtotal” (BCS) is the sum of costs of the work items currently defined. The “Direct Construction Subtotal” (DCS) is the BCS plus construction contingencies. For Preliminary Design a contingency allowance of 25 percent of the BCS was used to account for unit price and quantity variations, variable market conditions, and uncertainty at this phase of design. This percentage will likely decrease as the Project is better defined in subsequent stages of design. Other Project costs that are required to implement the Project are included as a percent of the BCS as follows:

- Design Engineering: 9 percent of the BCS.
- Construction Engineering and Management: 12 percent of the BCS.
- CLOMR/LOMR Engineering and Fees: 2 Percent of BCS.
- Environmental Permitting: 1 Percent of BCS.

A summary of the OPPC is presented in Table 17.1 and supporting information is presented in Appendix I.

TABLE 17.1
OPPC SUMMARY

Category	Cost
General Items	$9,606,000
General Earthwork	$4,730,000
Embankment Dam	$2,424,000
Spillway	$11,213,000
Instrumentation	$178,000
Barrier Wall	$1,404,000
Outlet Works	$3,935,000
Site Drainage	$375,000
US 36 Multi-Use Trail	$181,000
Bonds and Insurance	$510,000
BCS	**$34,556,000**
Contingencies (25 percent)	$8,511,000
DCS	**$43,067,000**
Other Costs	$7,948,000
OPPC	**$51,015,000**

The OPPC is based on professional opinions and may change as more design details are developed. Actual costs will be affected by several factors beyond current control, such as supply and demand for the types of construction required at the time of bidding, the Project vicinity, change in material supplier costs, changes in labor rates, competitiveness of contractors and suppliers, availability of qualified bidding contractors, changes in applicable regulatory requirements, change in economic conditions, and changes in design standards. Conditions and factors arising as the Project proceeds from development through bidding and construction may result in construction costs that differ significantly from the estimate provided in this Report.

17.2 Basis of Cost Opinion

Design concepts and considerations are discussed in Sections 4 through 16. Additional considerations used to develop the OPPC are as follows:

- Stripping and stockpiling topsoil will consist of removing the top six inches of existing topsoil.
- Demolition of existing CU Boulder South facilities will include the demolition of fencing, concrete pavement, a maintenance building, and tennis courts. Some items

associated with the demolition of the maintenance building are unknown, such as if the building contains asbestos, and costs associated with unknown items were not included.

- Erosion and sediment control will consist of a silt fence extending along the limits of site disturbance.
- Ecological restoration will include mitigating impacts to wetlands at a ratio of 2.5:1. For each acre of wetlands impacted by the Project, 2.5 acres of ecological restoration will occur. Costs for additional ecological restoration beyond what is required for wetland mitigation are not included. Some of the calculated wetland impacts are located in areas of temporary disturbance and it is possible that impacts to these wetlands could be reduced.
- The cost for imported fill includes cost of placing fill from off-site excavations and the material, loading, and hauling costs. An off-site fill source has not yet been identified, so we considered a 10-mile haul cycle for the imported earthfill materials.
- Temporary signage and traffic control for the US36 multi-use path consists of detour signage placed every 500 feet along the detour as well as at each intersection. Two barricades will also be placed at each end of the detour. The construction period is assumed to be 18 months and temporary signage will be inspected daily by traffic control personnel during the construction period.

SECTION 18 - CONSTRUCTABILITY CONSIDERATIONS

18.1 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.

18.2 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 preliminary 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 future stages of design to evaluate whether the PUB land use area can be used for contractor staging.

18.3 Earthwork Balance

Primary onsite borrow sources for the Project include the detention excavation, CU Boulder levee, and CU Boulder west berm. Required excavations in these areas are estimated to produce about 230,000 cubic yards (CY) of borrow material. Primary fill areas include the dam embankment and earthen roadway ramp and are estimated to require about 300,000 CY of fill. 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 significant quantity (at least 70,000 CY) of earthfill will need to be imported to the Site to construct Project facilities. This quantity could increase depending on the volume of excavated material that cannot be used for fill because of excessive boulders or other undesirable materials. At this time, an offsite borrow source has not been established. Potential offsite locations should be identified in the next phase of design so they can be secured, and appropriate permits obtained in advance of bidding the Project. It is our opinion that identification and procurement of this large quantity of import may be very difficult during a bidding process. Earthwork balance will also 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 future phases 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.

For this level of design, 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 practicality of processing onsite soils to produce these products should be evaluated in future phases of design.

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.

18.4 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. Additional test pits should be performed in the next phase of investigation to better characterize the oversized materials onsite. 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.

18.5 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 from groundwater that is dewatered from other Project alignments should also be evaluated. Key issues will be water rights, water quality, and water quantity.

18.6 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. For the 30-percent design, we have assumed a 90-foot-wide construction corridor directly south of the CDOT ROW boundary based on discussions with City and OSMP staff. This will accommodate the excavation of a 56-foot-wide working platform with 1.5H:1V side slopes. An excavated working platform slightly above the groundwater level is required to install the groundwater conveyance system collection and distribution trenches within standard trench boxes instead of in excessively deep shored excavations. The 56-foot-wide platform will accommodate two lanes of equipment traffic, each with sufficient width for an excavator and dump truck. However, the 56-foot-wide platform will not provide sufficient space for stockpiling excavated materials along the spillway alignment.

There are two general options to manage the large volume of excavated material along the spillway alignment, which consist of the following:

1. Large-scale stockpiling of all excavated material could occur within the contractor staging area. The material will then be hauled back to the spillway alignment for backfilling the completed structure. This additional movement of material will add cost and duration to the Project.
2. Stockpiling only enough material within the contractor staging area to open an initial work area along the spillway alignment. Subsequent material excavated along the spillway will be placed directly to backfill the adjacent completed work as construction progressed along the spillway. This option will reduce the overall handling of earthwork; however, it could significantly limit the productivity and increase the duration of specialty construction (i.e., secant pile installations).

A wider working platform and construction corridor will be beneficial to simplify and facilitate construction; however, this will encroach further into OSMP property.

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-ft by 20-ft 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.

18.7 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.

18.8 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.

During future phases 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. Additional coordination with the DCD2 Company will be required as the design advances.

18.9 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). If construction of the spillway occurs during the irrigation season (i.e., generally April through October), then 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 near the work areas.

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.

18.10 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 will be to allow the contractor to select the level of flood protection. This will transfer the risk to the contractor, and the cost for assuming this risk will be incorporated into the overall Project cost.

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.

18.11 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 it 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 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 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 be several feet above groundwater to maintain stability of the excavation.

18.12 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.

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

18.13 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 for 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 along the south side of US36 will be closed to the public during construction. A temporary detour for the multi-use trail will be established. The detour will extend along South Boulder Road and South Cherryvale Road. The contractor will be required to install and maintain appropriate traffic control and signage for the detour.

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

18.14 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.

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APPENDIX A

JURISDICTIONAL SIZE AND HAZARD CLASSIFICATION EVALUATION

TECHNICAL MEMORANDUM

Project 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Robert Huzjak, P.E. - RJH Consultants, Inc.**

DATE: **August 26, 2021**

REVISED: **March 1, 2022**

RE: **South Boulder Creek Regional Detention Project**
Jurisdictional Size and Hazard Classification Evaluation

1.0 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present the methodology and results of the jurisdictional size and hazard classification evaluation for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City).

2.0 Background

The Project will consist of installing an embankment dam, spillway, outlet works, detention excavation, and other ancillary facilities. These components are collectively referred to as the dam. The Colorado Office of the State Engineer (SEO) has established criteria for a jurisdictional size dam. Jurisdictional dams in Colorado are regulated and subjected to the authority of the SEO.

The SEO has also developed 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.

3.0 Regulatory Criteria

3.1 Jurisdictional Size

In accordance with Rule 4.6.1 of the *Rules and Regulations for Dam Safety and Dam Construction* (SEO Rules) (SEO, 2020), a jurisdictional size dam must meet one of the following criteria:

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

3.2 Hazard Classification

In accordance with Rule 4.13 of the SEO Rules, the hazard classification of a dam is identified by analyzing the potential consequences from a sunny-day failure of the dam. A high-hazard dam is a dam for which loss of human life is expected to result from a dam failure.

4.0 Jurisdictional Determination

RJH developed elevation-capacity data for the proposed reservoir. Reservoir storage will be provided by a combination of below- and above-grade storage. Elevation-capacity data used for this evaluation presented in Table 4.1. Elevation-capacity data will be updated as the Project grading is refined in future stages of design.

TABLE 4.1
ELEVATION-AREA-CAPACITY

Elevation	Surface Area (acre)	Capacity (acre-feet)
5343	2.1	0
5344	2.3	2.2
5345	2.7	4.7
5346	3.1	7.6
5347	3.5	10.9
5348	4.0	14.7
5349	4.5	18.9
5350	6.0	24.1
5351	7.7	30.9
5352	9.4	39.4
5353	11.7	50.0
5354	13.6	62.6
5355	20.0	79.3
5356	22.0	100.2
5357	30.4	126.3
5358	34.3	158.6
5359	41.0	196.2
5360	46.9	240.1
5361	54.0	290.5
5362	61.2	348.1
5363	67.5	412.4
5364	73.2	482.8

The reservoir will typically be dry except during and immediately after a storm event. The maximum normal reservoir pool will be controlled by the spillway with a crest elevation of El. 5363.8. Reservoir storage above El. 5359.5 that is not conveyed through the spillway will drain to SBC as the stormwater flow in SBC decreases. Reservoir storage below El. 5359.5 will not drain to SBC and will be discharged through the outlet works. A plan of Project components is presented on Figure 1. A profile and sections of the embankment and spillway are presented on Figures 2 to 6.

The maximum normal pool (i.e., El. 5363.8) is typically used to evaluate whether the dam is jurisdictional. However, this Project is unique because a portion of the reservoir pool freely drains to SBC. It is more appropriate to consider only the portion of the reservoir that does not drain to SBC (i.e., below El. 5359.5). Reservoir capacity and surface area for these elevations are presented in Table 4.2.

TABLE 4.2
RESERVOIR CAPACITY AND SURFACE AREA

Maximum Normal Pool	Reservoir Capacity (ac-ft)	Reservoir Surface Area (ac)
El. 5363.8	469	72
El. 5359.5	218	44

The reservoir capacity and surface area would meet requirements for a jurisdictional size dam for both of the reservoir pool elevations.

The jurisdictional height is defined as the vertical dimension from the lowest point of the natural ground surface or invert of the outlet pipe, whichever is lower, to the spillway crest. The jurisdictional height of the dam is 20.8 feet based on a spillway crest of El. 5363.8 and an outlet works invert elevation of El. 5343.0. The jurisdictional height of the dam would be 10.8 feet if it is based on the lowest natural ground surface elevation along the dam alignment (~El. 5352). In either case, the jurisdictional height exceeds the 10-foot criteria.

In RJH's opinion, the dam should be classified as a jurisdictional size dam by the SEO. The dam meets all three criteria presented by the SEO for a jurisdictional dam.

5.0 Hazard Classification

The hazard classification is identified based on potential consequences associated with a sunny-day failure of the dam. For this evaluation, a sunny-day condition was considered to be the volume of water stored after the flood event, which is consistent with other flood control dams we have evaluated. RJH performed simplified dam breach analyses in accordance with procedures presented in the SEO *Guidelines for Dam Breach Analysis* (SEO, 2020). The simplified breach analysis consisted of the following components:

Breach Parameter Estimate:	Empirical methods
Breach Hydrograph Development:	Parametric hydrologic model (HEC-HMS)
Breach Hydrograph Routing:	Two-dimensional hydraulic model (HEC-RAS)
Hydraulics at Critical Locations:	Two-dimensional hydraulic model (HEC-RAS)

Dam breach parameters were developed using the Froehlich equations in general accordance with the SEO Guidelines based on storage intensity. Dam breach analyses were performed for reservoir pools at El. 5363.8 and 5359.5. Breach parameters are summarized in Table 5.1.

TABLE 5.1
SUNNY-DAY BREACH PARAMETERS

Parameter	Reservoir Pool El. 5363.8	Reservoir Pool El. 5359.5
Bottom Breach Width, B_b	55 feet	40 feet
Breach Formation Time, t_f	0.75 hour	0.5 hour
Breach Side Slopes, z (ZH:1V)	0.7	0.7

The simulated dam breach hydrographs were developed using the U.S. Army Corps of Engineers (USACE) HEC-HMS model. The dam breach parameters shown in Table 5.1 were used in the HEC-HMS program to model the temporal development of the breach and resulting outflow.

TABLE 5.2
BREACH OUTFLOWS

Parameter	Reservoir Pool El. 5363.8	Reservoir Pool El. 5359.5
Peak Outflow	7,755 cfs	4,240 cfs
Breach Volume	450 ac-ft	210 ac-ft

RJH performed downstream flood routing using an unsteady, two-dimensional USACE HEC-RAS (HEC-RAS 2D) model. The model terrain was developed from LiDAR data acquired in 2013. We assumed the dam breach occurred at the maximum section of the dam. Hydraulic model results are presented on Figures 7 to 10 for reservoir pools at El. 5363.8 and 5359.5, respectively.

The SEO *Guidelines for Hazard Classification* (SEO, 2020) state that loss of life is expected when the product of flow depth and velocity is greater than seven. For both reservoir pool elevations, the product of flow depth and velocity significantly exceeds seven at major roadways downstream of the dam.

Based on this analysis, it is our opinion that the dam would be classified as a high-hazard dam by the SEO because loss of life is anticipated during a potential sunny-day breach.

6.0 March 2022 Update

The spillway elevation and detention excavation grading plan were revised during development of the 30-percent design. The spillway elevation was raised by one foot to El. 5364.8 for freeboard, and the detention excavation volume was increased to provide additional flood storage. These modifications do not change RJH's opinion regarding the jurisdictional determination and hazard classification. The increased maximum normal reservoir pool and increased storage would only increase downstream flood impacts, which would not change the hazard classification because it was previously identified as high hazard.

7.0 Conclusions

Based on discussions with the SEO and analyses presented herein, the proposed dam would be classified as a jurisdictional, high-hazard dam.

8.0 References

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Colorado Office of the State Engineer (2020). *Guidelines for Dam Breach Analysis*.

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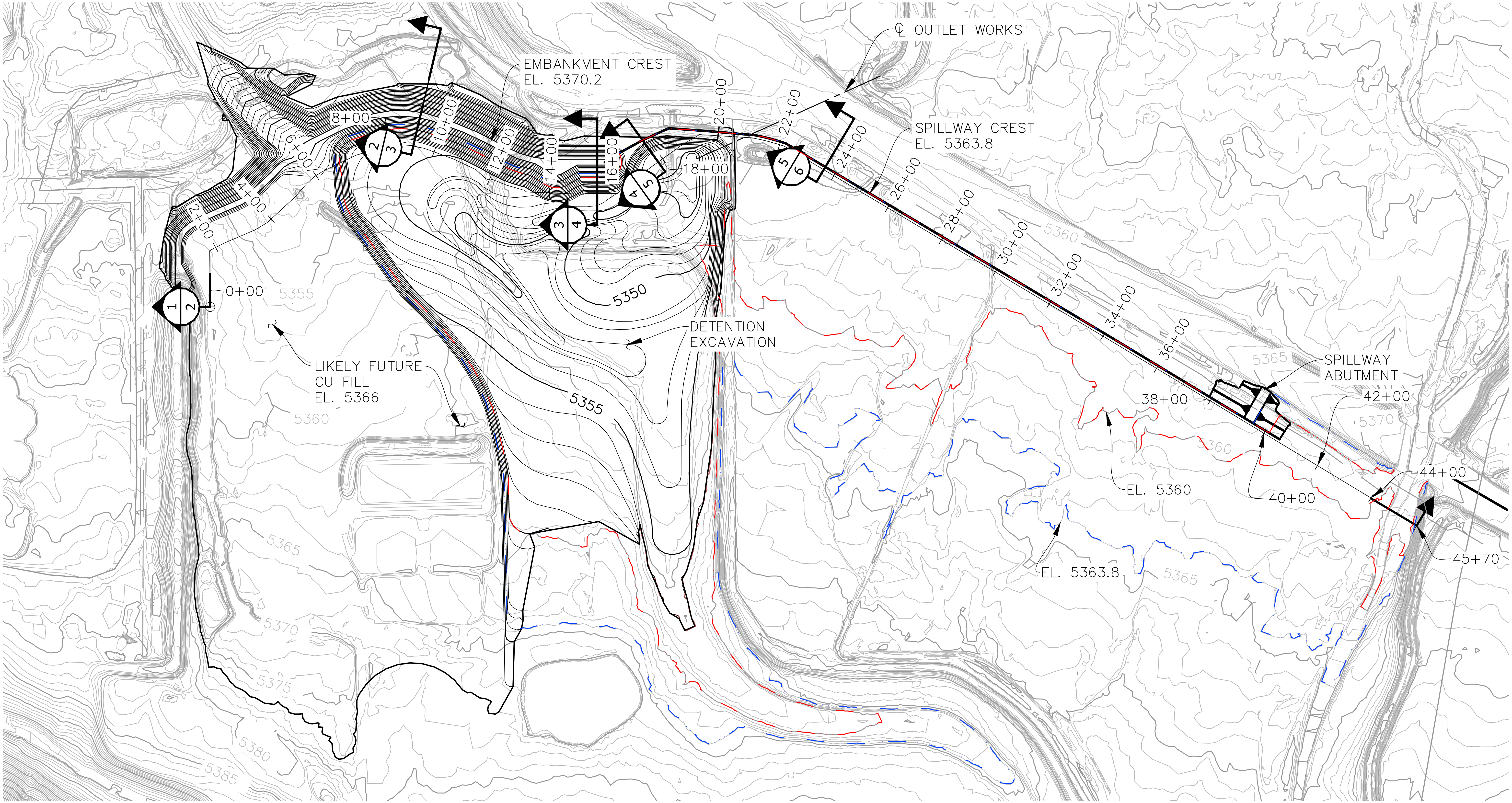

RJH
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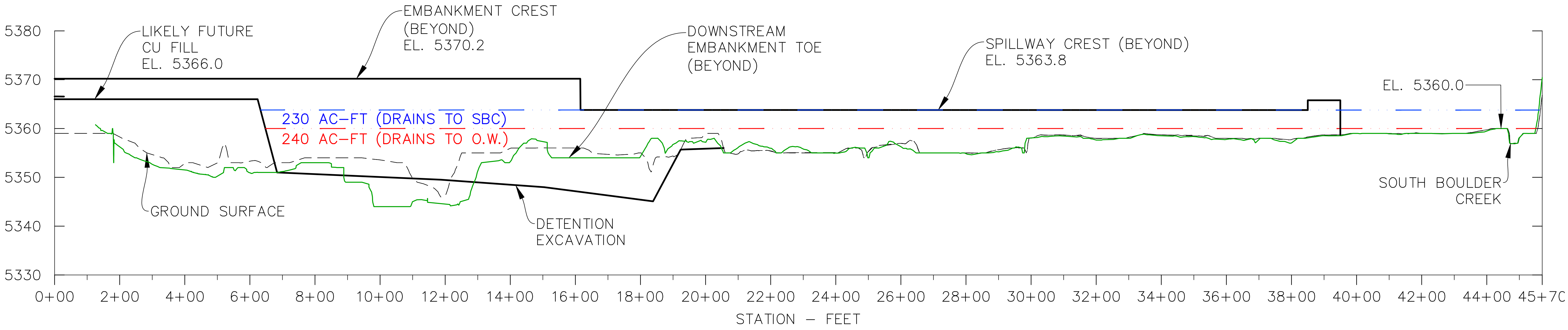
SOUTH BOULDER CREEK
REGIONAL DETENTION PROJECT

PLAN

PROJECT NO. 16134

April 2021

Figure 1



SECTION 1/1

0 30 60 120
VERTICAL SCALE IN FEET

0 150 300 600
HORIZONTAL SCALE IN FEET

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SOUTH BOULDER CREEK
REGIONAL DETENTION
PROJECT

EMBANKMENT AND SPILLWAY
PROFILE

PROJECT NO. 16134

April 2021

Figure 2

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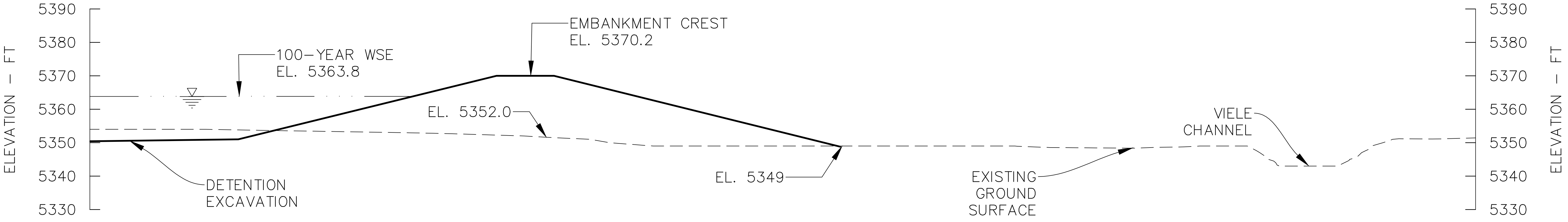

SECTION 2/1

0 15 30 60
SCALE IN FEET

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SOUTH BOULDER CREEK
REGIONAL DETENTION PROJECT

SECTION 2

PROJECT NO. 16134

April 2021

Figure 3

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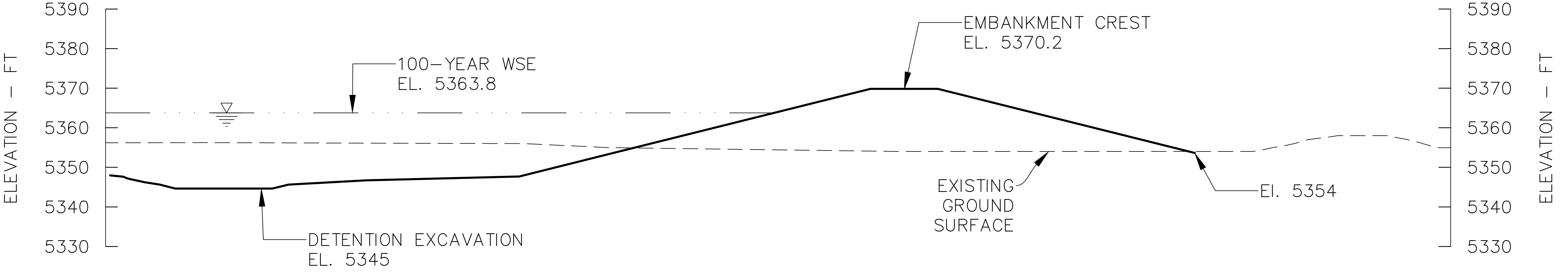

SECTION 3/1

0 15 30 60
SCALE IN FEET

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION PROJECT

PROJECT NO. 16134

SECTION 3

April 2021

Figure 4

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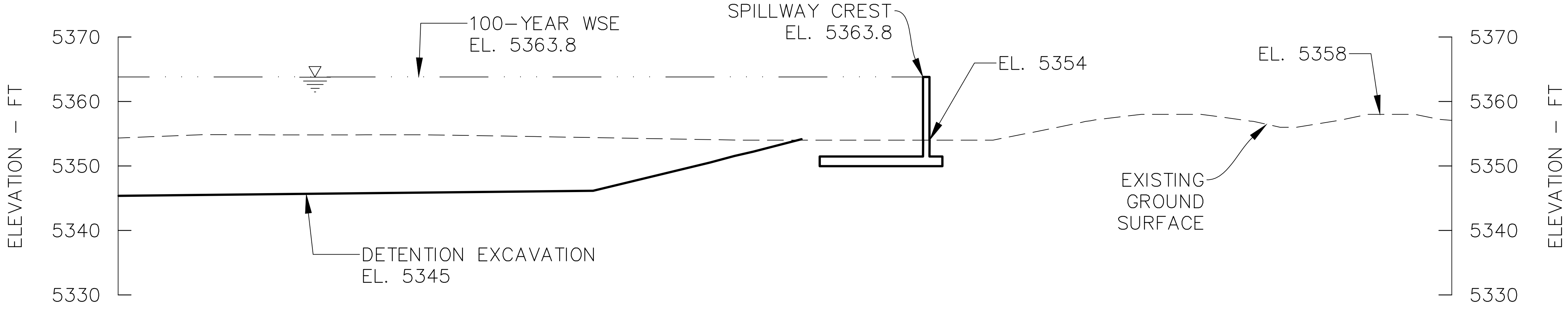

SECTION 4/1

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SOUTH BOULDER CREEK
REGIONAL DETENTION PROJECT

SECTION 4

PROJECT NO. 16134

April 2021

Figure 5

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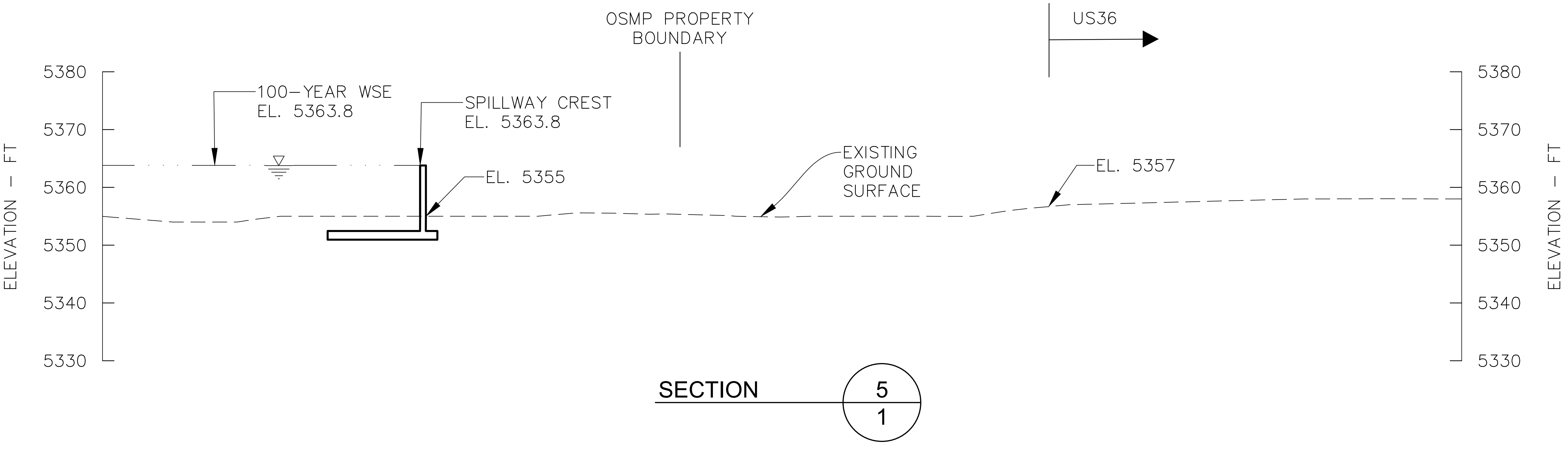

SOUTH BOULDER CREEK
REGIONAL DETENTION PROJECT

SECTION 5

PROJECT NO. 16134

April 2021

Figure 6

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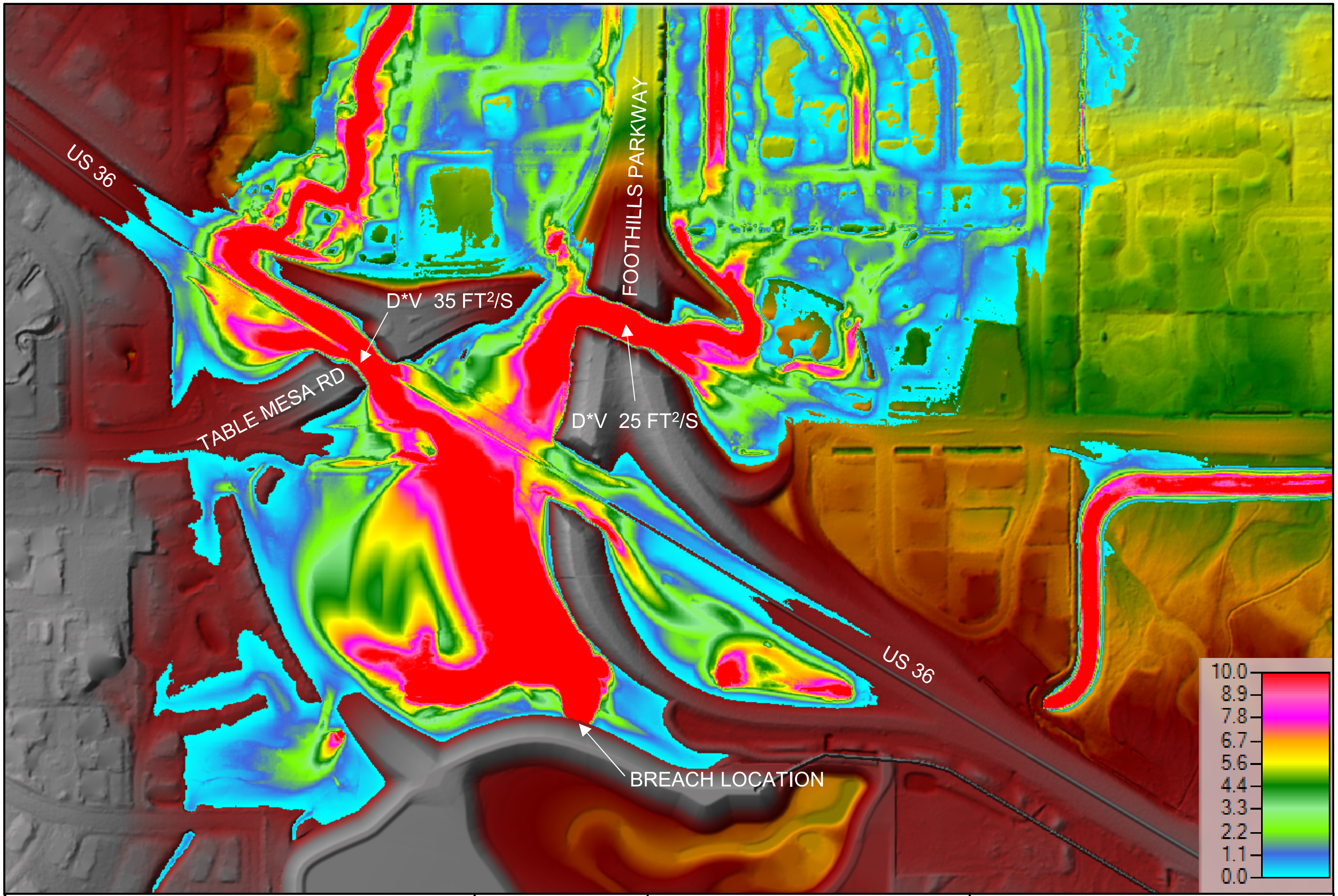
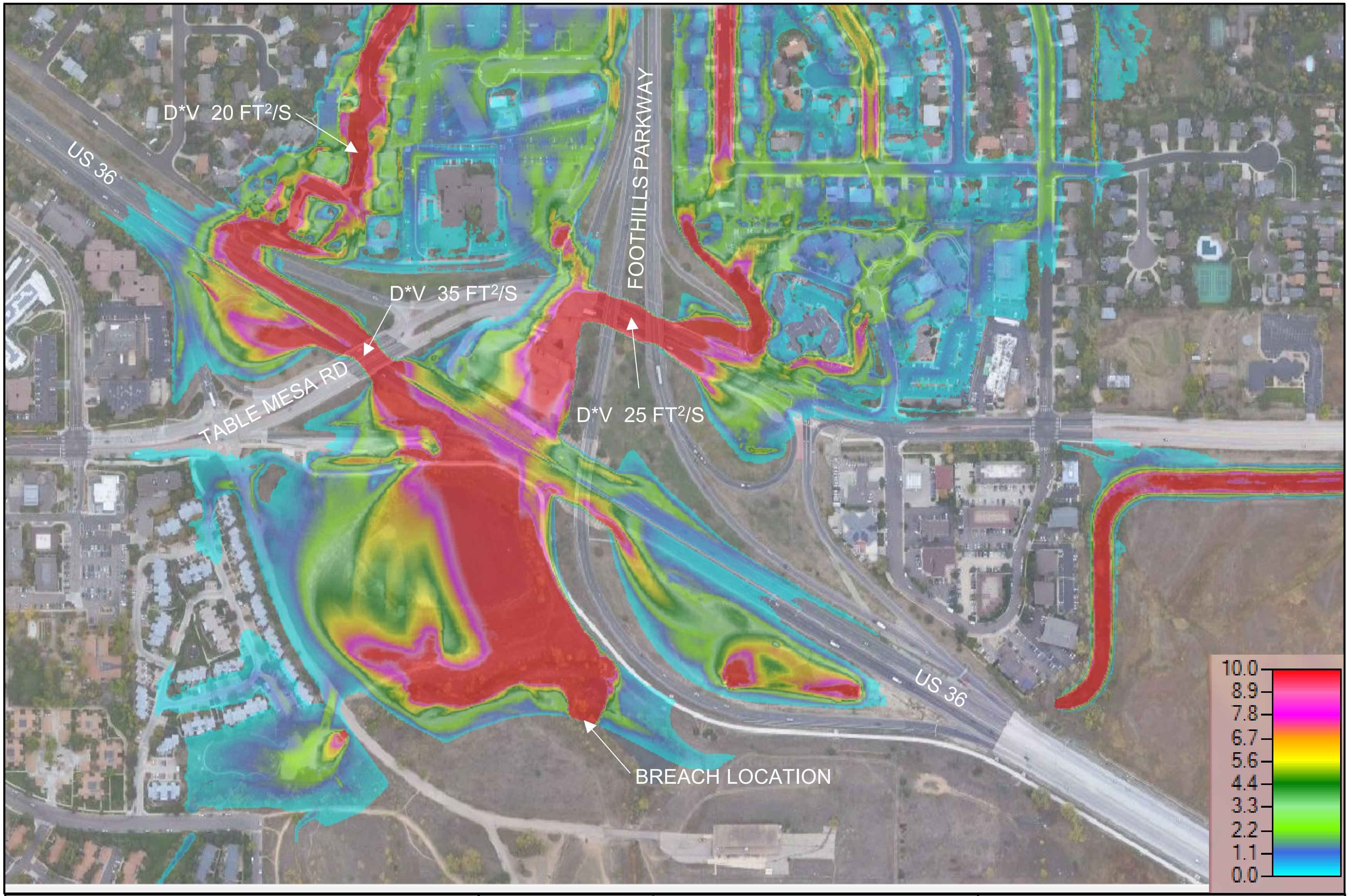


SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT	DEPTH-VELOCITY RESULTS – RESERVOIR POOL AT EL. 5363.8	
PROJECT NO. 16134	April 2021	**Figure 7**

RJH CONSULTANTS, INC.



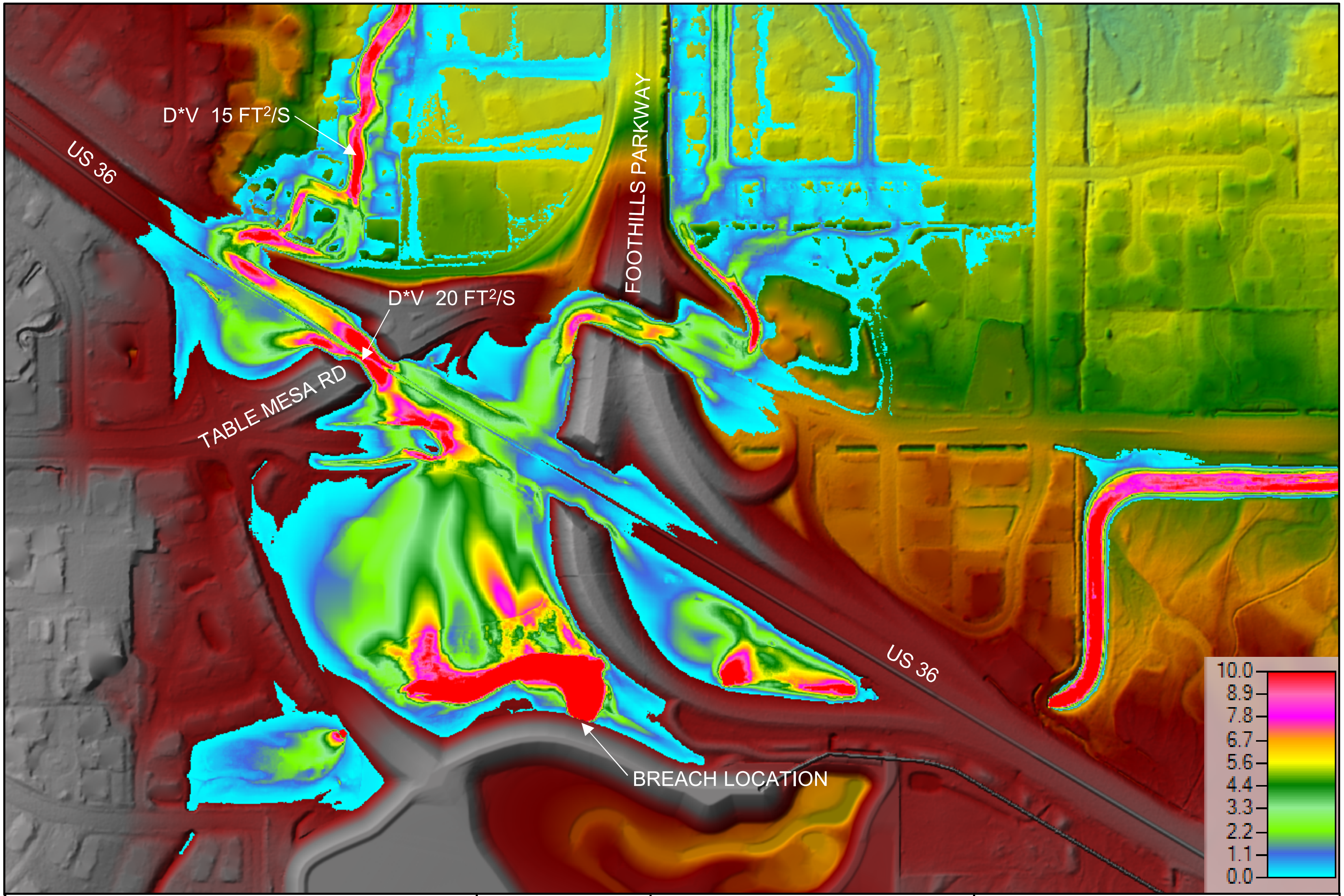
SOUTH BOULDER CREEK
REGIONAL DETENTION
PROJECT

DEPTH-VELOCITY RESULTS
– RESERVOIR POOL AT
EL. 5363.8

PROJECT NO. 16134

April 2021

Figure 8



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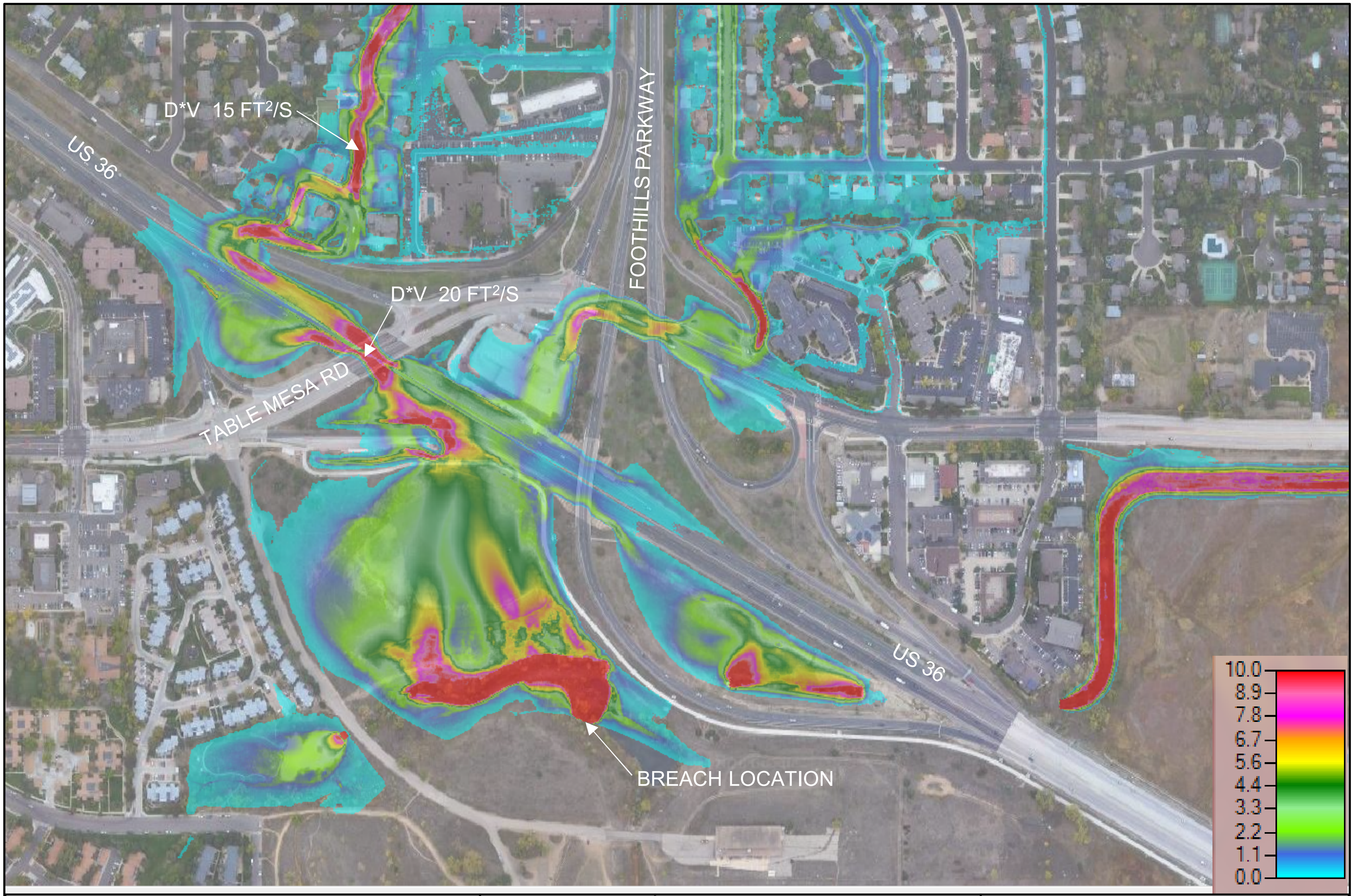
SOUTH BOULDER CREEK
REGIONAL DETENTION
PROJECT

PROJECT NO. 16134

DEPTH-VELOCITY RESULTS
– RESERVOIR POOL AT
EL. 5360.0

April 2021

Figure 9



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SOUTH BOULDER CREEK REGIONAL DETENTION PROJECT

PROJECT NO. 16134

DEPTH-VELOCITY RESULTS – RESERVOIR POOL AT EL. 5360.0

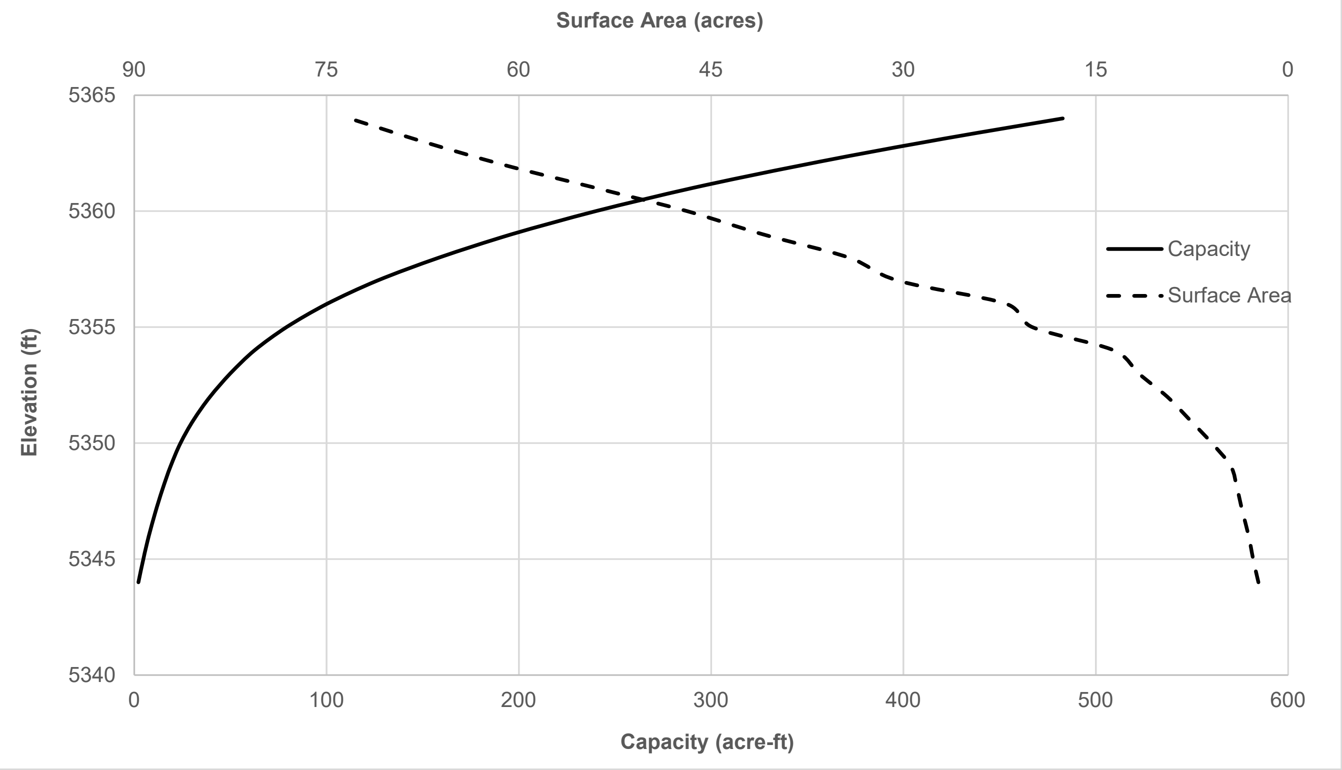
April 2021

Figure 10

16134 - South Boulder Creek
Elevation Capacity Data

Elevation	Surface Area (SF)	Surface Area (Acre)	Incremental Volume (Acre-feet)	Total Volume (Acre-ft)
5343	92,410	2.1	--	--
5344	100,772	2.3	2.2	2.2
5345	118,833	2.7	2.5	4.7
5346	134,041	3.1	2.9	7.6
5347	154,172	3.5	3.3	10.9
5348	172,644	4.0	3.7	14.7
5349	194,690	4.5	4.2	18.9
5350	259,400	6.0	5.2	24.1
5351	334,092	7.7	6.8	30.9
5352	410,987	9.4	8.5	39.4
5353	507,702	11.7	10.5	50.0
5354	592,717	13.6	12.6	62.6
5355	869,524	20.0	16.7	79.3
5356	957,441	22.0	21.0	100.2
5357	1,324,803	30.4	26.1	126.3
5358	1,493,707	34.3	32.3	158.6
5359	1,785,212	41.0	37.6	196.2
5360	2,042,917	46.9	43.9	240.1
5361	2,352,996	54.0	50.4	290.5
5362	2,665,859	61.2	57.6	348.1
5363	2,939,780	67.5	64.3	412.4
5364	3,188,903	73.2	70.3	482.8

Water Surface El.:	**5,363.8**	**ft**
Total Capacity:	**468.7**	**ac-ft**



APPENDIX B

HYDRAULIC MODELING

B.1 PRELIMINARY PROPOSED CONDITIONS HYDRAULIC MODEL

B.2 ALTERNATE CORRECTED EFFECTIVE MODEL

APPENDIX B.1

PRELIMINARY PROPOSED CONDITIONS HYDRAULIC MODEL

MEMORANDUM

To:	Eric Hahn, PE (RJH Consultants, Inc.)
From:	Stephen Blake, PE and Ian Dubinski, PhD (DHI)
Date:	April 14, 2022
Subject:	Flood Modeling for Preliminary Design of South Boulder Creek Regional Detention Facility

---

This memorandum summarizes the supporting flood modeling performed for the preliminary design and selection process for the South Boulder Creek Regional Detention Facility at US Highway 36 (US36) using the project Corrected Effective Model (CEM).

1 Introduction

Hydraulic modeling was performed as part of the concept design phase to select a preferred alternative for the regional detention facility. Based on this work, the City of Boulder (City) selected Variant 1, Option 1 (100-year) concept presented in the South Boulder Creek Regional Detention Concept Design Report (RJH, 2020) as the preferred alternative to advance into preliminary design /19/.

The primary objective of the hydraulic modeling summarized in this memorandum is to compare the changes in hydrodynamics during design flood event for various configurations of the preferred alternative. This will inform selection of the preliminary design to advance into the 60% design phase.

Scenario flood models for preliminary design alternatives were built on the underlying project Corrected Effective Model of the MIKE FLOOD South Boulder Creek (SBC) Effective Model for the design flood events. For each scenario, a suite of results including hydrographs, water surface elevations (WSEs), and flood extents at selected locations were generated for performance and impact comparisons to inform concept selection.

2 Project South Boulder Creek Corrected Effective Flood Models

The current Federal Emergency Management Agency (FEMA) 100-year SBC Effective Model (EM100) covering the project site are the South Boulder Creek Effective Model series was built using DHI's MIKE FLOOD software package /3/. These were built as part of the Flood Mapping Study completed by HDR Engineering, Inc. (HDR) in 2008 as documented in the Hydraulic Modeling Report /3/. Digital copies of the EM100 model were obtained in person by DHI from the Urban Drainage and Flood Control District in October 2017 /4/.

As part of this project, a preliminary Corrected Effective Model (CEM) was developed for the 100-year design event, the same flood event in EM100 (CEM100 2021). The primary changes from EM100 are:

- Update in MIKE FLOOD Version 2021
- Updated channel bathymetry and floodplain topography
- Updates to structures
- Update to MIKE FLOOD Links between 1-D and 2-D models
- Addition of the US36 Wildlife Crossing.

These updates are briefly summarized in this memorandum. For further detail on the preliminary CEM please refer to /5/.

2.1 Changes to Effective Model

The EM100 and EM500 models obtained from the MHFD are in the 2009 SP1 version of the MIKE FLOOD software modeling package. DHI upgraded the EM100 and EM500 models from version 2009 SP1 /6/ to version 2021 Update 1 /7/ to incorporate software updates made since 2009. These include computational speed increases that allow for running multiple scenarios much more efficiently.

EM Update from MIKE Flood Version 2009 SP1 to Version 2021 Update 1

As part of model support for Concept Design of the South Boulder Creek Regional Detention Facility /2/, EM100 was updated from version 2009 SP1 /6/ to version 2017 SP2 /8/. Changes to EM100 as part of the software upgrade were a combination of naming conventions, additional results outputs, and modifications to the Mike11.ini file.

The Mike11.ini file provides for customization of the 1-D MIKE 11 model including computational approaches. The Mike11.ini file was updated to version 2017 SP1 format which includes new settings not available in version 2009 SP1. The following changes to the Mike11.ini file were made during conversion to MIKE FLOOD 2017 SP1 to ensure consistency in the computational approach to reduce changes in results between versions:

- HD-Variable no. 56 MF_semi__implicit = Off
- HD-Variable no. 56 MF_Semi_Implicit_V2011SP4_Correction = Off
- Other settings not previously available in version 2009 SP1 set to default settings

As part of the model support for Preliminary Design of the South Boulder Creek Regional Detention Facility, the EM100 in MIKE FLOOD version 2017 SP2 was upgraded to version 2021 Update 1. Changes to the model setup were only required for the setup file structure to comply with the setup file structure in version 2021 Update 1. No additional changes were required to run EM100 version 2017 SP1 in MIKE FLOOD version 2021 Update 1.

Updates to Channel Bathymetry and Floodplain Topography

The channel bathymetry and floodplain topography from the EM100 was fully updated with more recent surveys. The following surveys were merged in ascending order to form the bathymetry in CEM100 2021:

1. FEMA 2013 Post-Flood LiDAR Final: Post-2013 Flood LiDAR survey covering entire MIKE FLOOD model domain /9/
2. South Boulder Creek Restoration Project 2019 As-Built Channel Bathymetry: 2019 flood restoration project as-built bathymetry (cross sections for South Boulder Creek channel covering from approximately from CO93 downstream to South Boulder Road /10/
3. Project Site Survey 2021: 2021 ground-truth survey covering the proposed detention facility site and the floodplain from SBC to the detention facility along US36 /11/

For further detail on development of the bathymetry for CEM100 2021 please refer to /5/.

Updates to Structures

DHI evaluated the MIKE 11 structures that needed modifications based on changes in the model domain that have occurred since the completion of the FIS /12/. These include the expansion of US 36, other construction improvements to South Boulder Road and the South Boulder Creek channel, and recommendations affecting structures from a previous review by CH2M HILL. A total of 38 structures required modifications, including 26 culverts and 12 bridges. This includes the following updates:

- US36 bridge opening after US36 expansion
- Revised downstream invert elevation for culvert EJF_01 on Dry Creek Ditch No. 2

For further details on structure updates please refer to /5/.

Updating MIKE FLOOD Links

Updates to MIKE FLOOD links covered both lateral and standard links. The MIKE FLOOD links connect MIKE 11 structures and branches to the MIKE 21 2D model domain. The MIKE FLOOD links have been generally recreated so that the correct elevations are used to incorporate the updated 2D bathymetry, the revised cross sections, and the revised locations of channel markers, to accurately describe the exchange and storage of floodwater between the 1D and 2D model domains. For further details on link updates please refer to /5/.

Addition of the US36 Wildlife Crossing

The wildlife crossing passing under U36 near the US36 bridge on SBC was added to the MIKE FLOOD model as part of the CEM100 2021. No debris blockage is currently included. The wildlife crossing is simulated as a culvert with the following dimensions:

-	Length	175 ft.
-	Number of culverts	2
-	Shape	Rectangular
-	Size of each culvert	9.84 ft wide by 4 ft tall (3 m by 1.2192 m)
-	Upstream invert	5356.7 ft
-	Downstream invert	5355.9 ft

For further details please refer to /5/.

Flood maps showing a comparison between EM100 2009 and CEM100 2021 are presented on Figure 1.

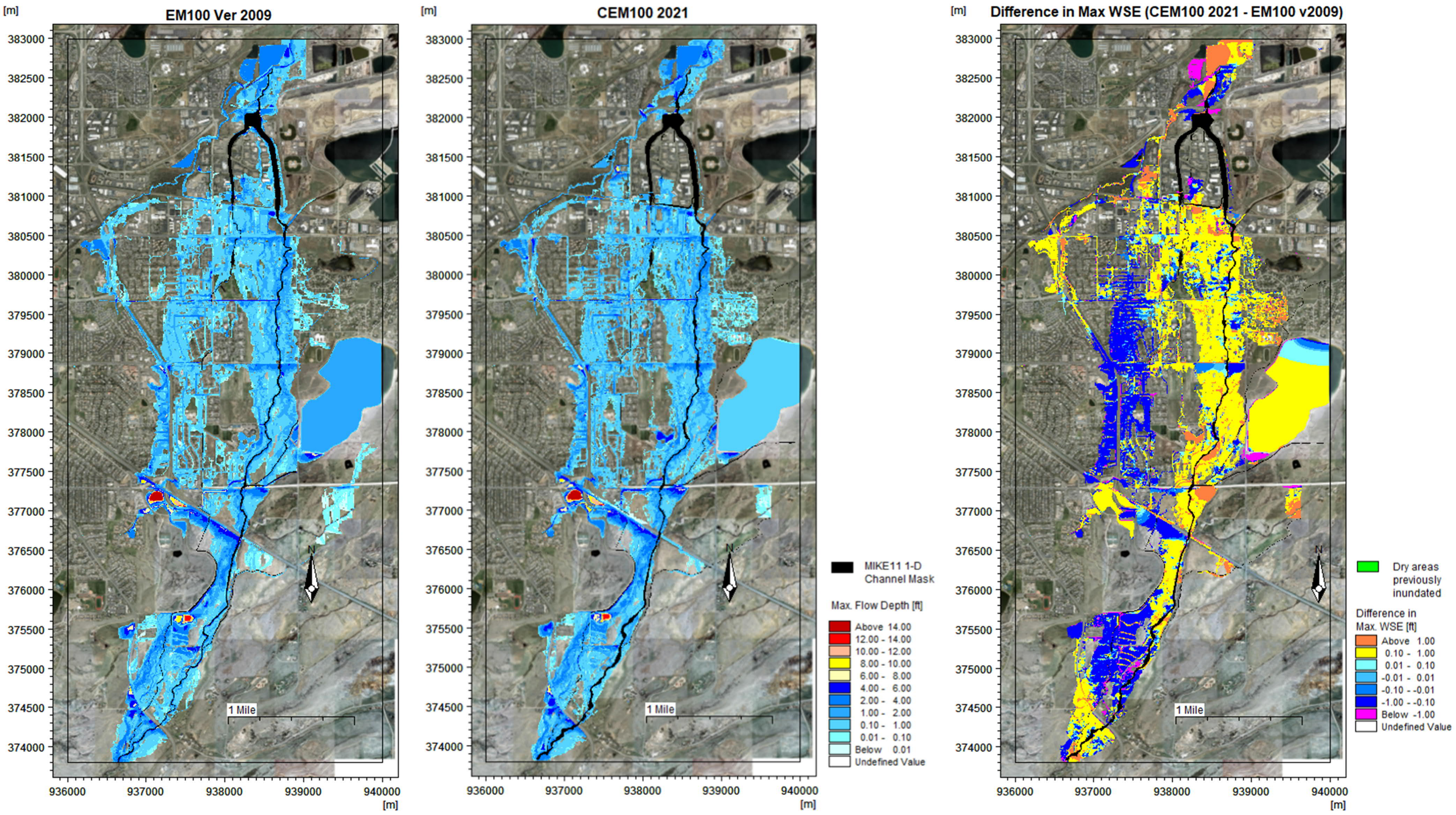


Figure 1 100-year Thunderstorm Event Flood Map - Comparison Between EM100 v2009 and project CEM100 2021

2.2 CONORT Projection

The MIKE FLOOD effective models obtained from MHFD utilize a customized geographic projection named “CONORT” /3/. This projection is used for the preliminary CEM.

The CONORT projection is defined as:

Name	CONORT
Ellipsoid	WGS 1984
Scale factor	0.999966637
Central meridian	-105.5
Latitude origin	0
False easting	914401.8289
False northing	-4050560.139453
Linear unit	Meter

3 Preliminary Design Model Scenarios

Nine (9) preliminary design scenarios were evaluated that varied the following design elements:

- CU Levee
- Outlet Works
- Blockage of US36 Wildlife Crossing
- Detention Facility Grading

The nine scenarios are listed below in Table 1. For each of these scenarios, the 100-year Thunderstorm design flood event was simulated. The 100-year Thunderstorm is the primary design flood events selected for evaluating the flood mitigation performance of the preliminary design scenarios.

Table 1 Preliminary Design Model Scenarios

Scenario	CU Levee Removed?	Outlet Works	US36 Wildlife Crossing Opening Blockage %	Detention Facility Grading	CU Levee Grading
Scenario 1	No	2 x 60" Circular Pipe	None	Base	2013 LiDAR
Scenario 2	No	2 x 60" Circular Pipe	None	Expanded	2013 LiDAR
Scenario 3	Yes	2 x 60" Circular Pipe	None	Expanded	Levee Removed
Scenario 4	Yes	2 x 60" Circular Pipe	50%	Expanded	Levee Removed
Scenario 5	Yes	3 x 60" Circular Pipe	None	Expanded	Levee Removed
Scenario 6	Yes	4 x 60" Circular Pipe	None	Expanded	Levee Removed
Scenario 7	Yes	1 x 60" Circular Pipe	50%	Expanded	Levee Removed
Scenario 8	Yes	1 x 60" Circular Pipe	None	Expanded	Levee Removed
Scenario 9	Yes	1 x 60" Circular Pipe	None	Revised Expanded	Levee Removed

3.1 Design Flood Events

The Climatology/Hydrology Report /13/ completed by HDR in 2007 as part of the Flood Mapping Study provides the base hydrology for developing design flood events. A 1-D MIKE 11 rainfall-runoff model was used to simulate the hydrographs in SBC and other basin inputs to the MIKE FLOOD model for SBC.

Flows from local stormwater runoff and other smaller drainages are also included in the MIKE FLOOD model for SBC as documented in /3/. The basis for the flow is documented in the Hydraulic Modeling Report /3/ and Climate and Hydrology Report /13/.

100-year Thunderstorm

The current 100-year design flood event for South Boulder Creek through the City of Boulder are from the Flood Mapping Study as documented in the Climatology/Hydrology Report completed in 2007 /13/. The 100-year design flood event is generated by a short duration, high intensity thunderstorm (100-year Thunderstorm).

The simulated peak flow for the 100-year design flood events when (1) approaching the project site and (2) passing under the US36 bridge crossing SBC for CEM100 is in Table 2.

Table 2 Peak Flows at US36 for Simulated Design Flood Events

Design Flood Event	Simulated Peak Flow Approaching US36 [cfs][1]	Simulated Peak Flow Passing Under US36 Bridge [cfs][2]
100-year Thunderstorm	7,112	3,682

[1] This is the combined flow in the SBC channel and floodplain including nearby Dry Creek Ditch No. 2

[2] Flow split upstream at US36 Bridge diverts a portion of approaching flow to west where it overtops US36

3.2 Scenario Model Variations

Detention Facility Embankment, Spillway, and Grading

Muller Engineering Inc. and RJH provided drawings for the following:

- Earthen embankment borders US36 west of Viele Channel then extends west and south to terminate on the west side of CU South Campus /14/
- Spillway tie-in to US36 /15/
- Three (3) alternative gradings for the combination of the detention excavation of the interior of the storage footprint and fill on the CU South Campus /14//16//17/
- The embankment and spillway tie-in to US36 is represented in all scenarios by the preliminary design grading provided in /14/ and /15/ respectively.

The three (3) alternative gradings for the combination of the detention excavation of the interior of the storage footprint and fill on the CU South Campus are the following:

- **Base Grading**: Baseline grading plan for preliminary design /14/

- **Expanded Grading**: Alternative preliminary design with more storage than Base Grading /16/
- **Revised Expanded Grading**: Second alternative preliminary design with more storage than Base Grading but less than Expanded Grading /17/

The provided surfaces representing the embankment, spillway tie-in, fill on CU South Campus and detention facility excavation was interpolated onto the MIKE FLOOD bathymetry. This new bathymetry replaces the existing bathymetry in the project CEM100 for that footprint in the scenario. Elevations outside of the footprint were not changed from the scenario CEM.

CU Levee

Two CU Levee scenarios were considered:

- **With Levee:** The CU Levee is represented in the MIKE 21 bathymetry of the preliminary CEM100 2021 on which the scenario is based (see /5/).
- **Without Levee:** RJH provided drawings for the ground surface representing removal of the CU Levee with grading from the upstream to downstream toes of the levee for the Preliminary Design /18/. This surface was interpolated onto the MIKE FLOOD bathymetry, replacing the existing preliminary CEM bathymetry for the CU Levee footprint represented in /18/.

Outlet Works

The outlet works for the detention facility is represented by a culvert structure in the MIKE 11 model connecting the interior of the storage facility footprint to an outlet near the Viele Channel culvert outlet downstream of US36 (CV15).

The culvert structure has the following dimensions:

Length	730.6 ft.
Number of culverts	Varies by scenario, see Table 1
Shape	Circular
Diameter of each culvert	60 in. (1.524 m)
Upstream invert	5344.1 ft
Downstream invert	5338.2 ft

- Only Positive Flow Allowed (i.e., only flows out of detention facility)

4 Scenario Results

4.1 100-year Thunderstorm

The scenarios results for the 100-year Thunderstorm are summarized in Appendix A by scenario. The maximum WSEs and peak flows at selected key locations for design performance are presented in Table 3.

Table 3 Maximum Flood Flow and WSE at Select Locations by Scenario

Scenario	Max WSE at US36 Bridge (ft)	Max WSE in Detention Facility (ft)	Peak flow under US36 Bridge (cfs)	Peak flow at South Boulder Road in SBC corridor (cfs)	Peak flow through Outlet Works (cfs)	Peak flow through US36 Wildlife Crossing (cfs)
CEM100 2021	5361.5	5356.4	3682	5477	N/A	810
Scenario 1	5362.0	5360.9	4128	6134	705	899
Scenario 2	5362.0	5359.5	4124	6061	677	899
Scenario 3	5361.3	5362.3	3378	5294	734	850
Scenario 4	5361.4	5362.9	3571	5088	746	421
Scenario 5	5361.3	5361.8	3376	5431	979	850
Scenario 6	5361.3	5360.9	3378	5609	1246	850
Scenario 7	5361.6	5363.4	3852	5135	351	444
Scenario 8	5361.2	5363.0	3374	5045	348	862
Scenario 9	5361.4	5363.3	3573	5293	350	865

Plan Flood Maps

The difference in simulated maximum water depths from the scenario CEM are shown on Figure 3 to Figure 11. The light green areas on each figure show previously inundated areas in the project CEM100 that are dry in the scenario simulation.

Flow Results

Model results are shown at select key locations identified on Figure 2, as described below.

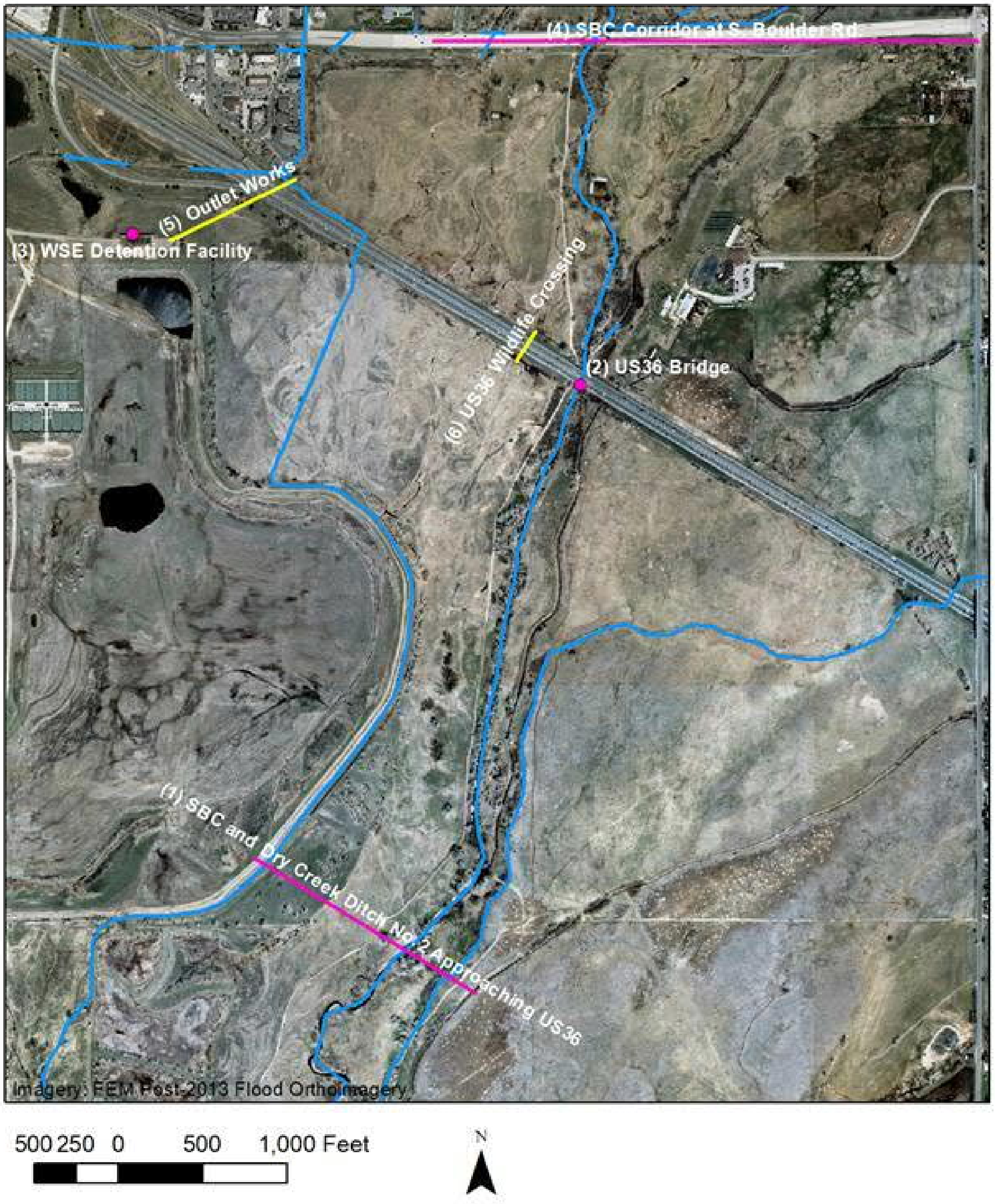


Figure 2 Simulation Results Location Map

Flow Under US36 Bridge

The simulated flow under the US36 Bridge during the flood are summarized in the following:

- Simulated flood hydrograph passing under the US36 Bridge for scenarios 1 to 7 are shown on Figure 12 and scenarios 8 and 9 on Figure 14.

- The peak flow and maximum WSE at the upstream entrance to the US36 Bridge for all scenarios are shown earlier in Table 3.

Flow At South Boulder Road in SBC Corridor

This is a simulated hydrograph of the flow passing S. Boulder Rd. in the SBC flood corridor. It includes the flow in Viele Channel, which junctions with SBC downstream. The flood hydrograph is a composite of the simulated flows at the following locations:

- Bridge passing SBC under S. Boulder Rd. (Bridge B35_SBC@SBoulderRd)
- Floodplain flow over S. Boulder Rd.in SBC corridor
- Culvert passing flow in Viele Channel under S. Boulder Rd. (Culvert CV2021)

The simulated hydrographs for scenarios 1 to 7 are shown on Figure 13 and scenarios 8 and 9 on Figure 15. The peak flow for all scenarios is shown earlier in Table 3.

Flow Through Outlet Works

This is the simulated drainage of the detention facility through the outlet works (OutletPipe). The simulated flood hydrographs for scenarios 8 and 9 are shown on Figure 16. The peak flow for all scenarios is shown earlier in Table 3.

Flow Through US36 Wildlife Crossing

This is the simulated flow passing through the US36 Wildlife Crossing (WildlifeCrossing_US36). The simulated flood hydrographs for scenarios 8 and 9 are shown on Figure 17. The peak flow for all scenarios is shown earlier in Table 3.

WSE Profile from SBC to Detention Facility

This is the simulated maximum WSE profile along a section running streamwise along the flow path from SBC to the Detention Facility. The profile location is shown on Figure 18. The simulated maximum WSEs is shown on Figure 19.

5 Conclusions

The preliminary design scenarios were evaluated using multiple criteria. Key performance criteria considered for the 100-year design flood event are:

- Prevent flood waters from SBC from overtopping US36 in the West Valley.
- Maintain or decrease the maximum WSE and peak flow in SBC at the US36 Bridge compared to CEM100 2021.
- Maintain or decrease the extent and maximum WSE of flood inundation downstream of US36
- Maintain or decrease the peak flow in the SBC flood corridor passing South Boulder Road compared to CEM100 2021.

After review of the hydraulic modelling results for the preliminary design scenarios, DHI offers the following conclusions:

- All scenarios prevented flood waters from SBC from overtopping US36 in the West Valley
- Scenario without the CU Levee removed (Scenarios 1,2) performed poorly with increases in maximum WSE at US36 Bridge, increases in peak flow at the US36 Bridge and South Boulder Road, and increases in both flood extent and maximum WSE downstream of US36. Attenuation of the flood waters that would have overflowed US36 into the West Valley was not sufficient to prevent rerouting into SBC downstream of U36 from worsening existing flood conditions.
- Scenario 8 with Expanded Grading for the Detention Facility, CU Levee removed, and outlet works with single 60 in. outlet pipe shows the best performance.
  - Decreases maximum WSE and peak flow under US36 Bridge
  - Decreases peak flow across SBC corridor at South Boulder Road
- Scenario 9 with Revised Expanded Grading for the Detention Facility also shows good performance.
  - Decreases maximum WSE and peak flow under US36 Bridge
  - Decreases peak flow across SBC corridor at South Boulder Road
  - Minor increases in maximum WSE in some areas downstream of US36.

6 References

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/11/ RJH Engineering, Inc. 2021. Project Site Survey 2021. 70430-Combined Topo 2021 12 C21_TRIMMED TO OSMP SOUTH.dwg. Received by DHI from RJH on December 8, 2021.

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/15/ Muller Engineering. 2021. Base Detention Facility Grading Floodwall Tie-in to US36. 2021-07-27_DHI_etransmit.dwg. July 27, 2021. Received by DHI from RJH on July 27, 2021.

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A Scenario Results

A.1 Plan Flood Maps

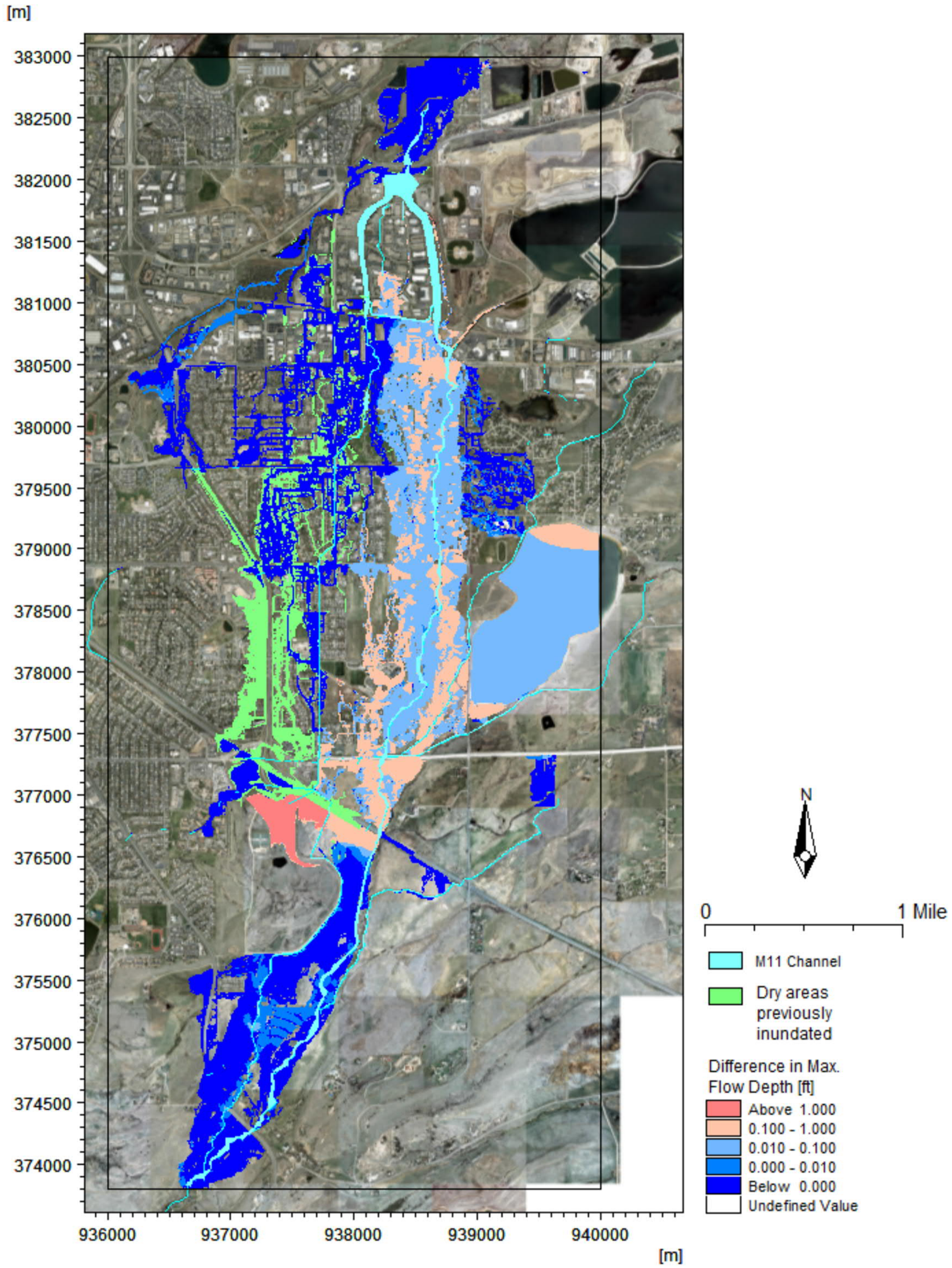

Figure 3 Difference in Maximum Water Depth from CEM100 – Scenario 1

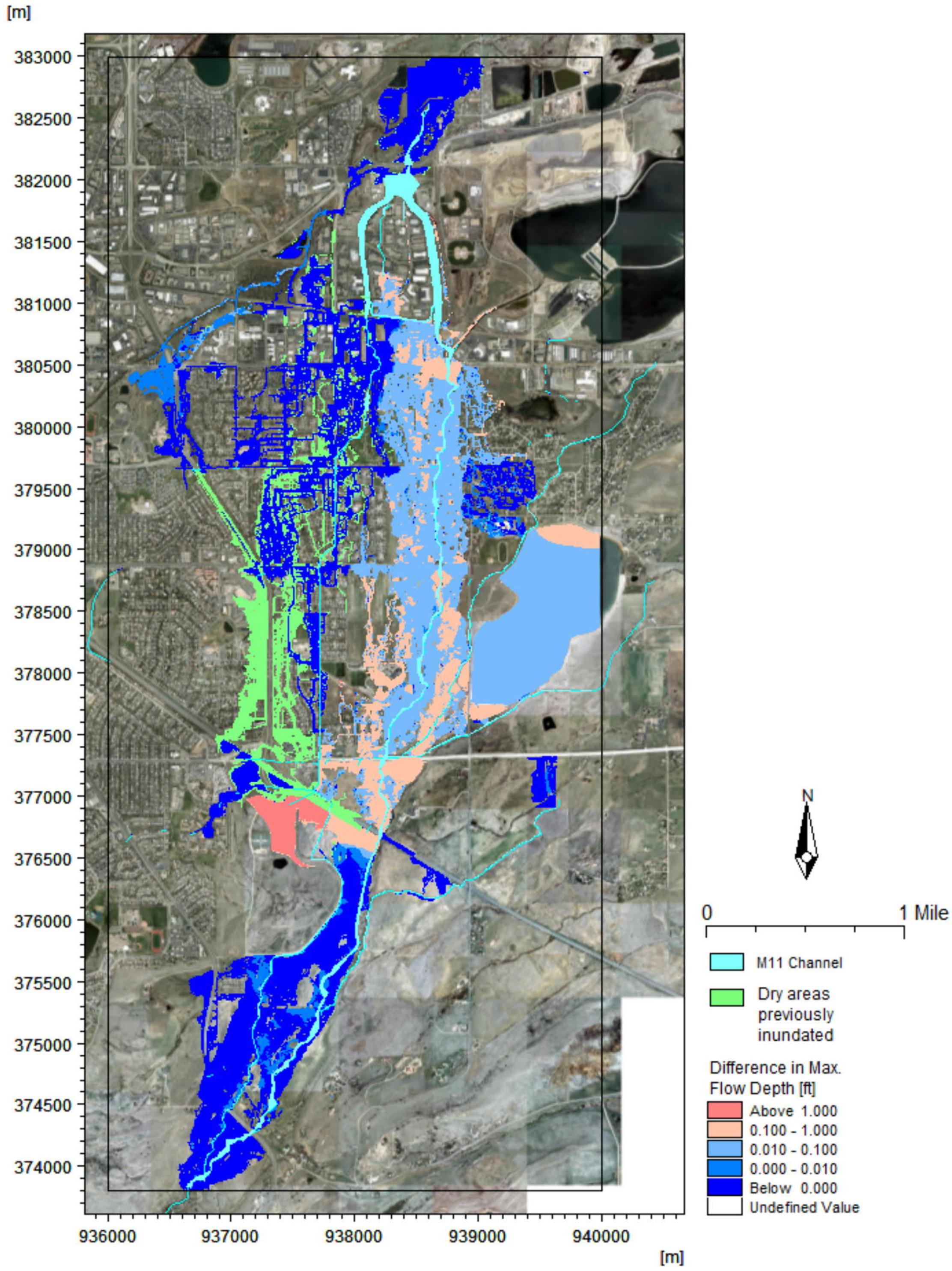

Figure 4 Difference in Maximum Water Depth from EM100 – Scenario 2

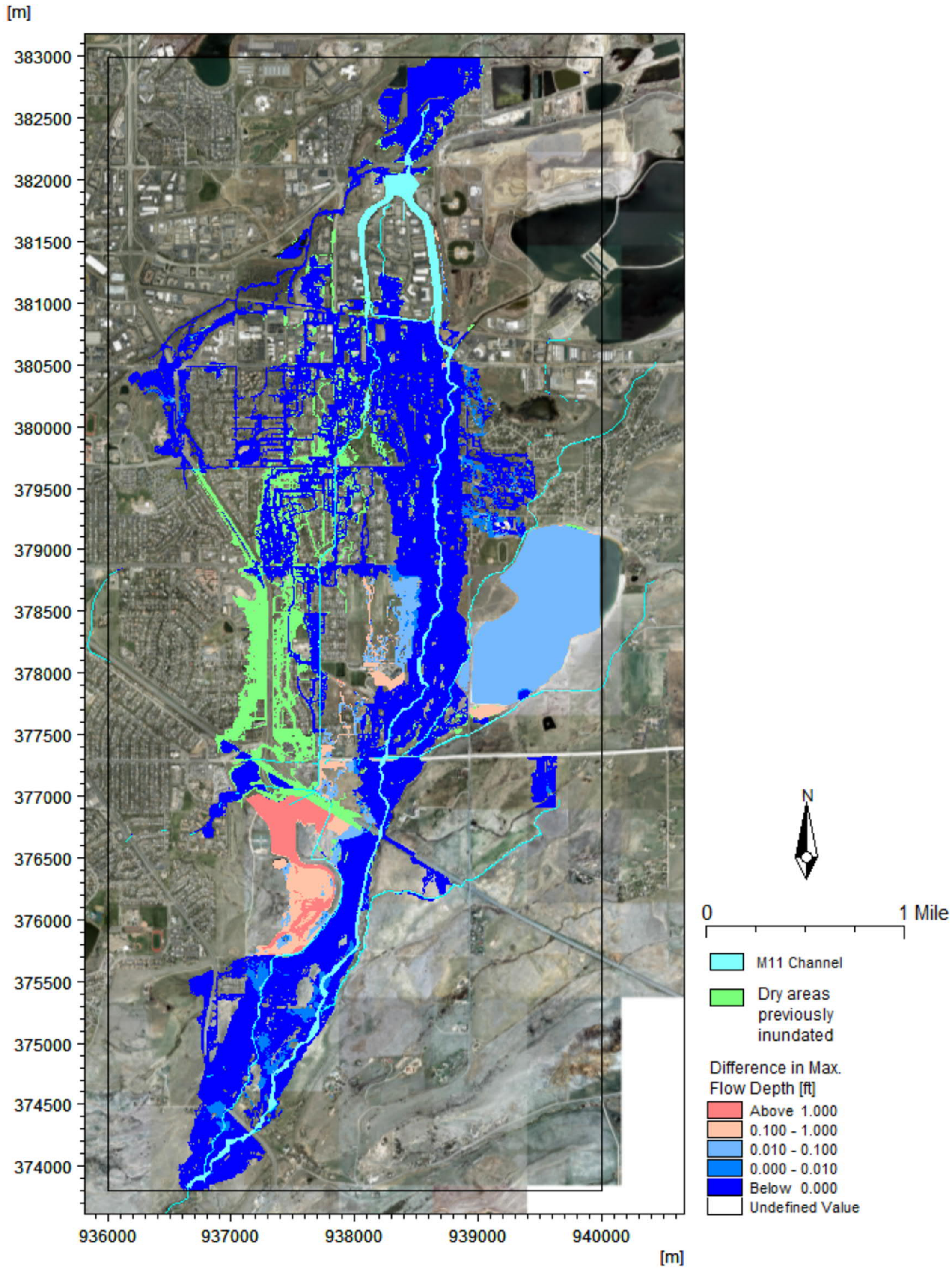

Figure 5 Difference in Maximum Water Depth from EM100 – Scenario 3

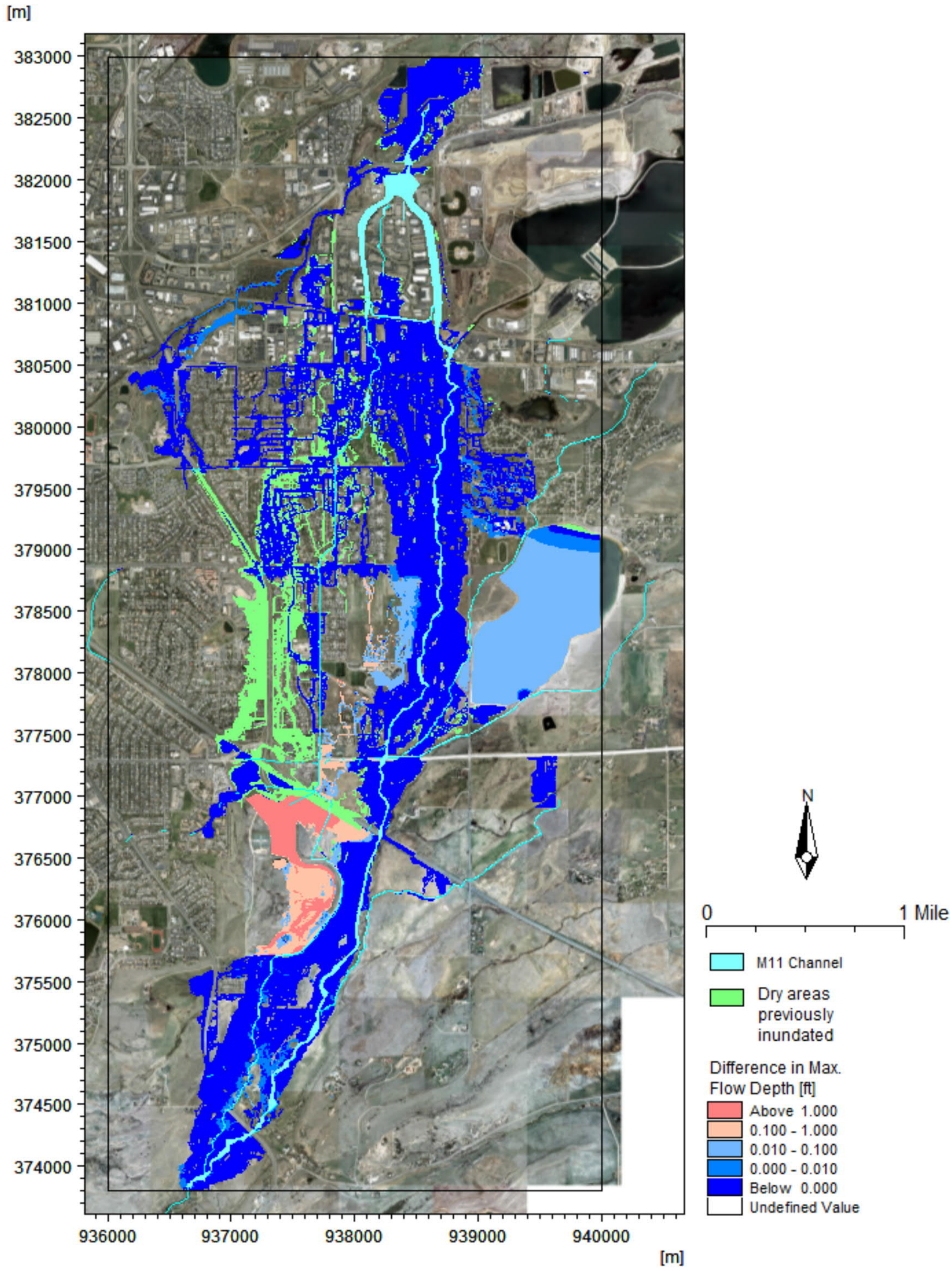

Figure 6 Difference in Maximum Water Depth from EM100 – Scenario 4

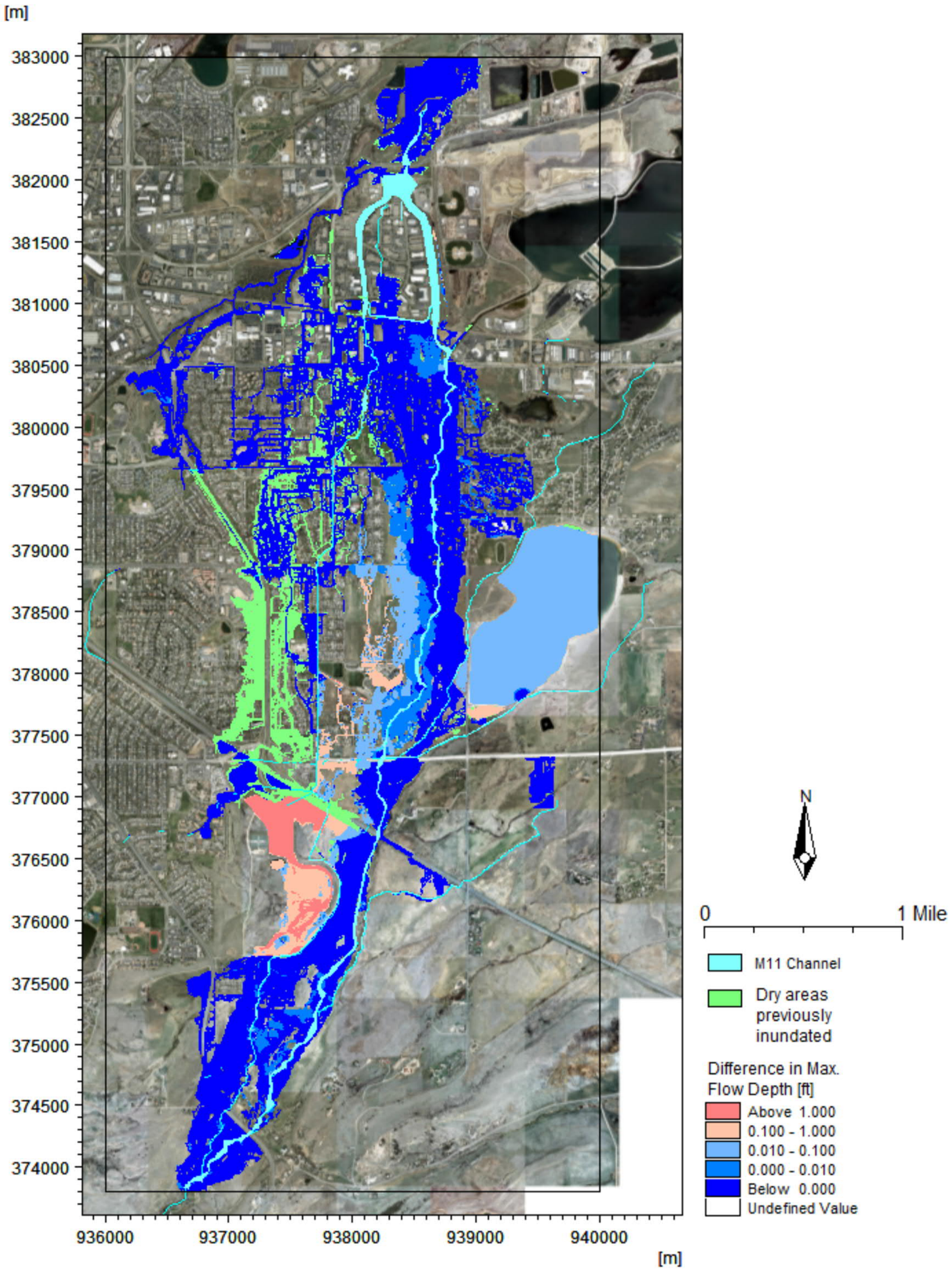

Figure 7 Difference in Maximum Water Depth from EM100 – Scenario 5

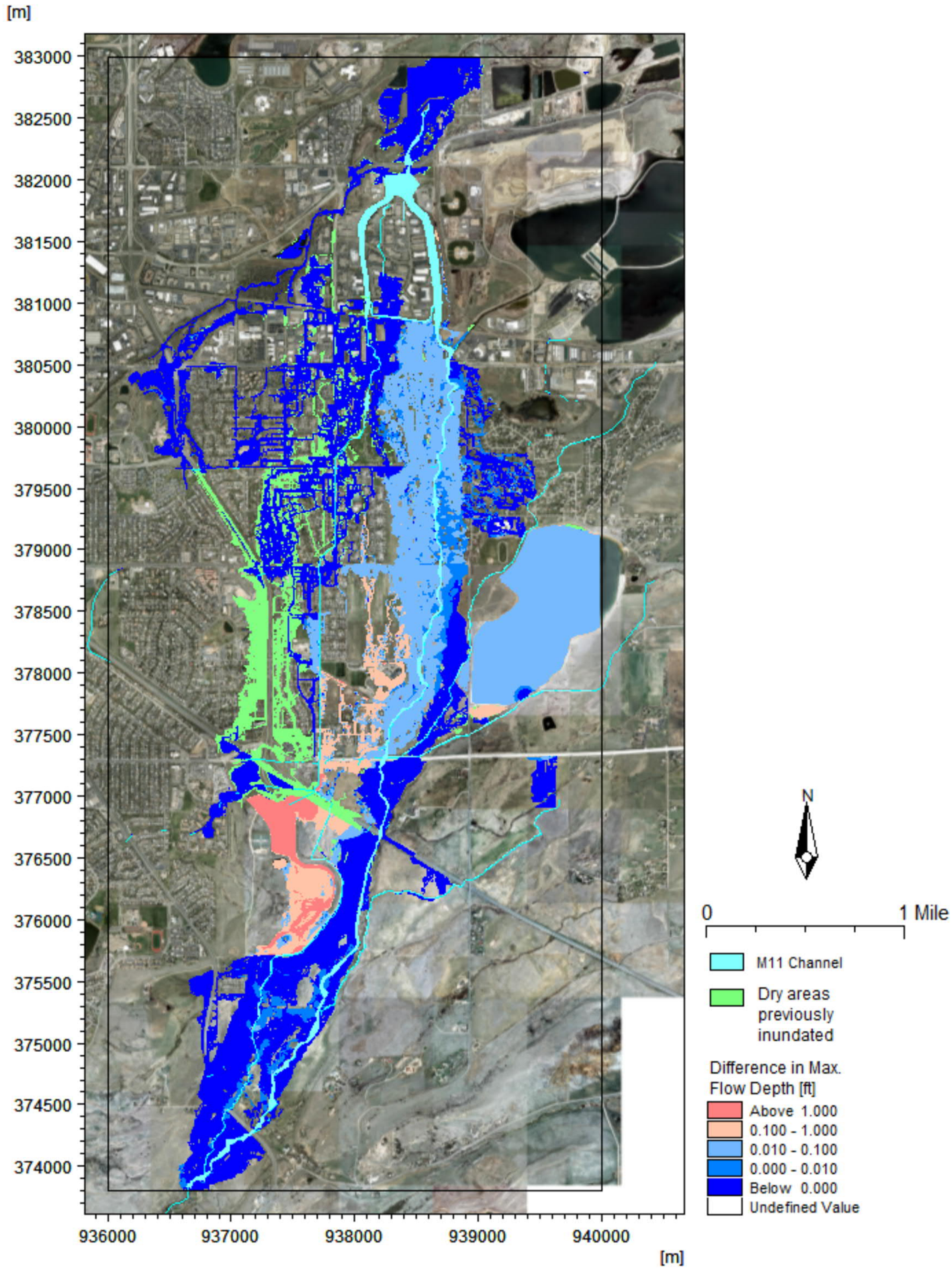

Figure 8 Difference in Maximum Water Depth from EM100 – Scenario 6

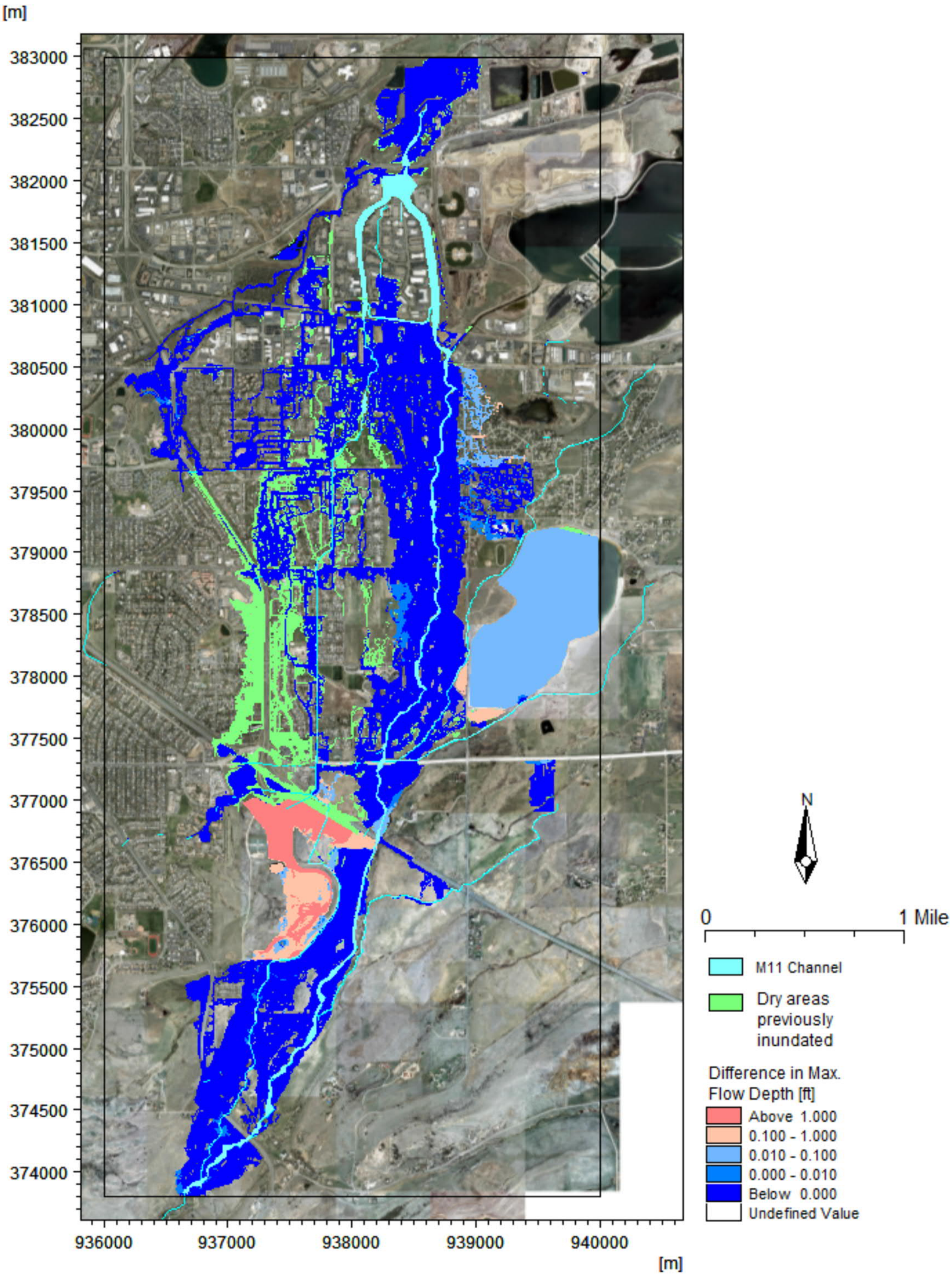

Figure 9 Difference in Maximum Water Depth from EM100 – Scenario 7

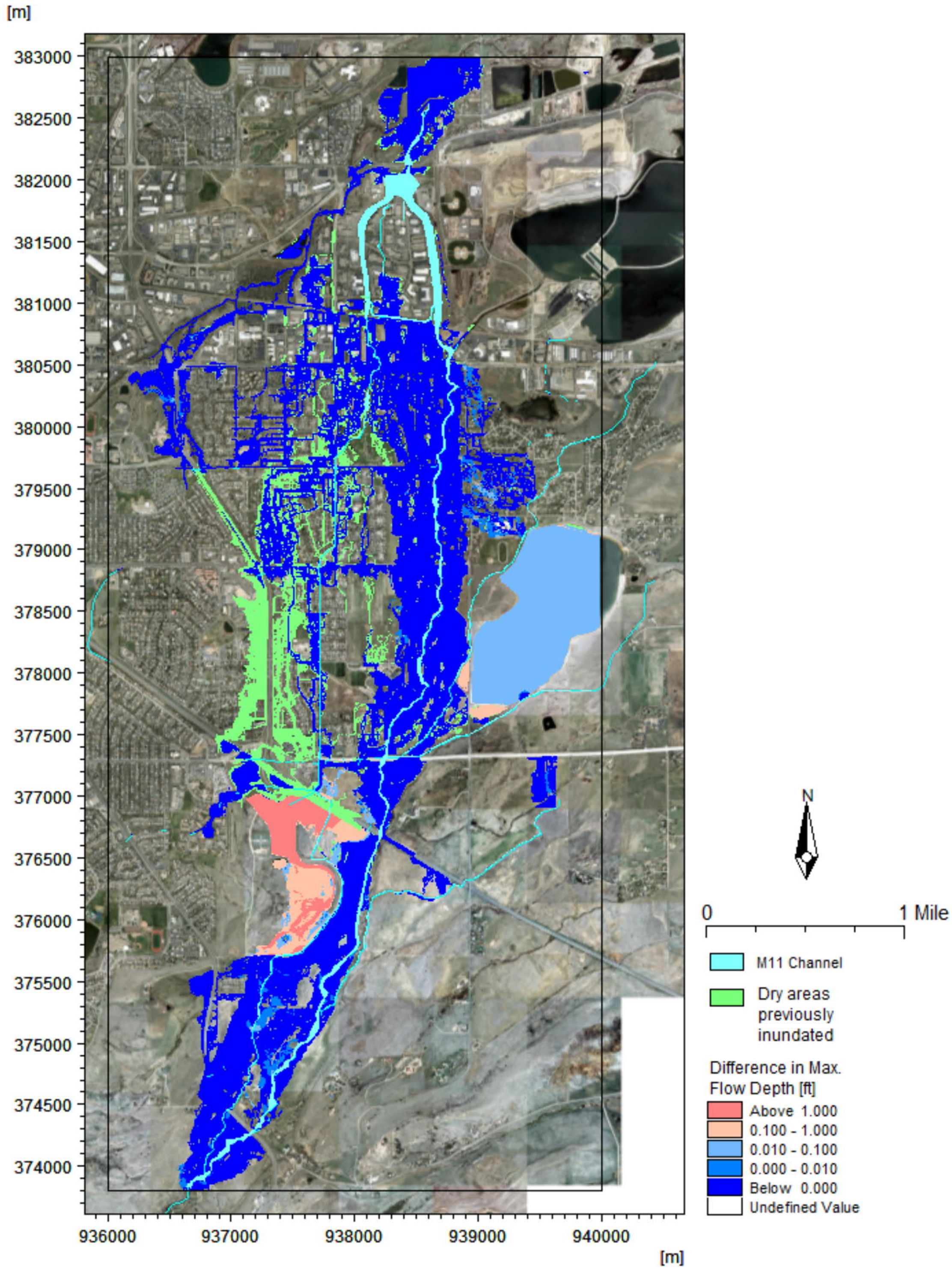

Figure 10 Difference in Maximum Water Depth from EM100 – Scenario 8

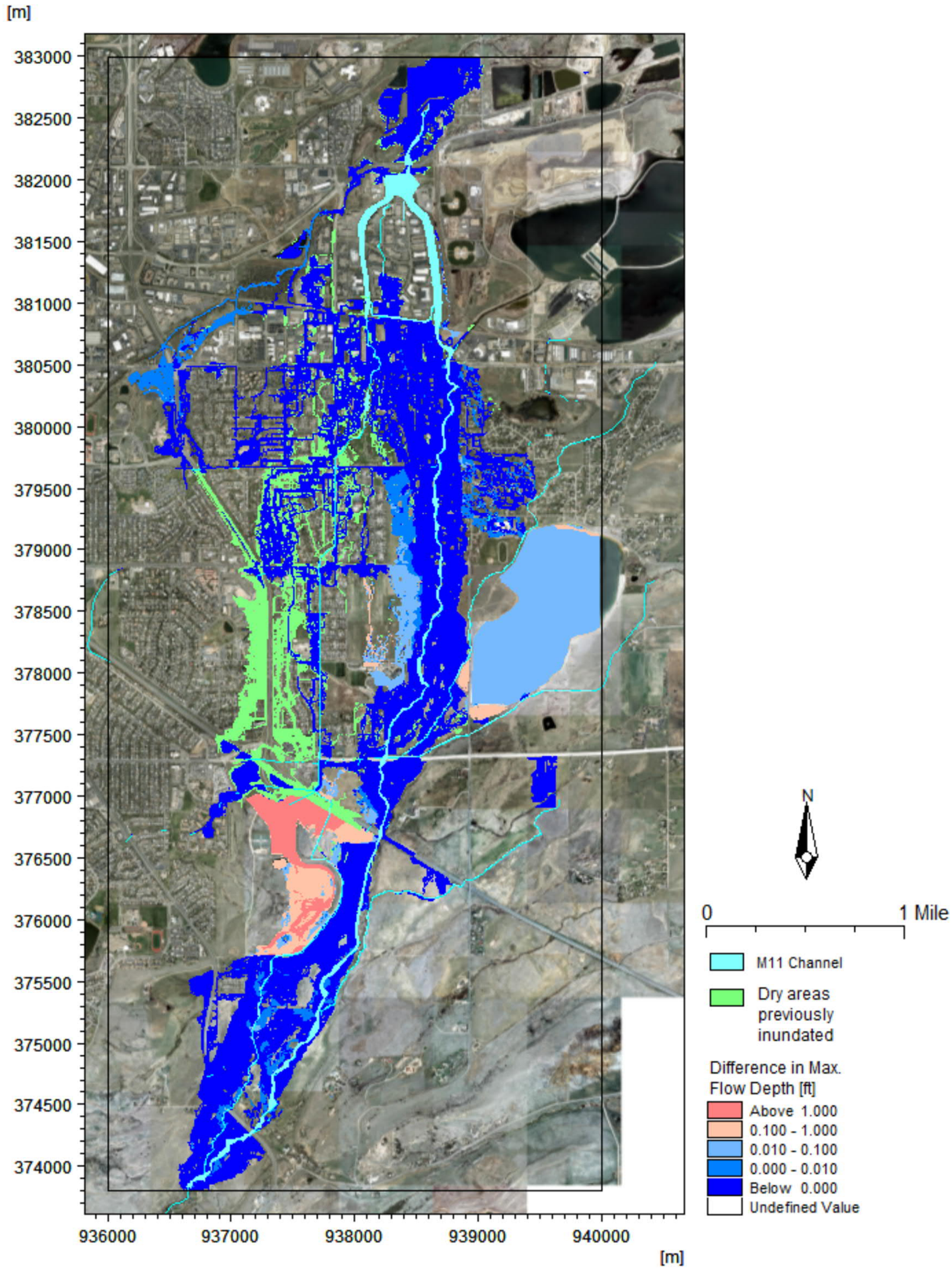

Figure 11 Difference in Maximum Water Depth from EM100 – Scenario 9

A.2 Flood Hydrographs

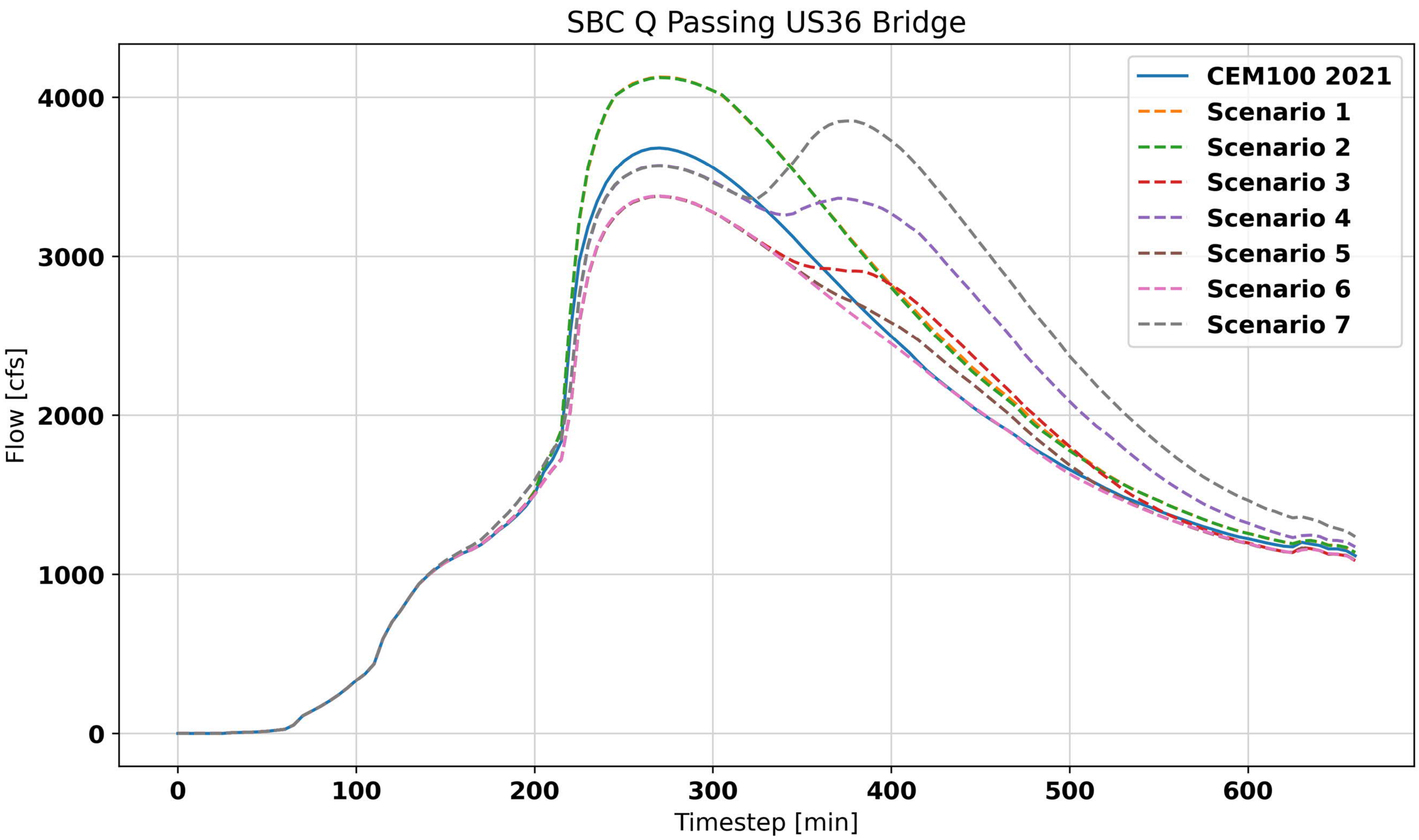

Figure 12 Flow Under US36 Bridge - 100-year Thunderstorm Event – Scenarios 1 to 7

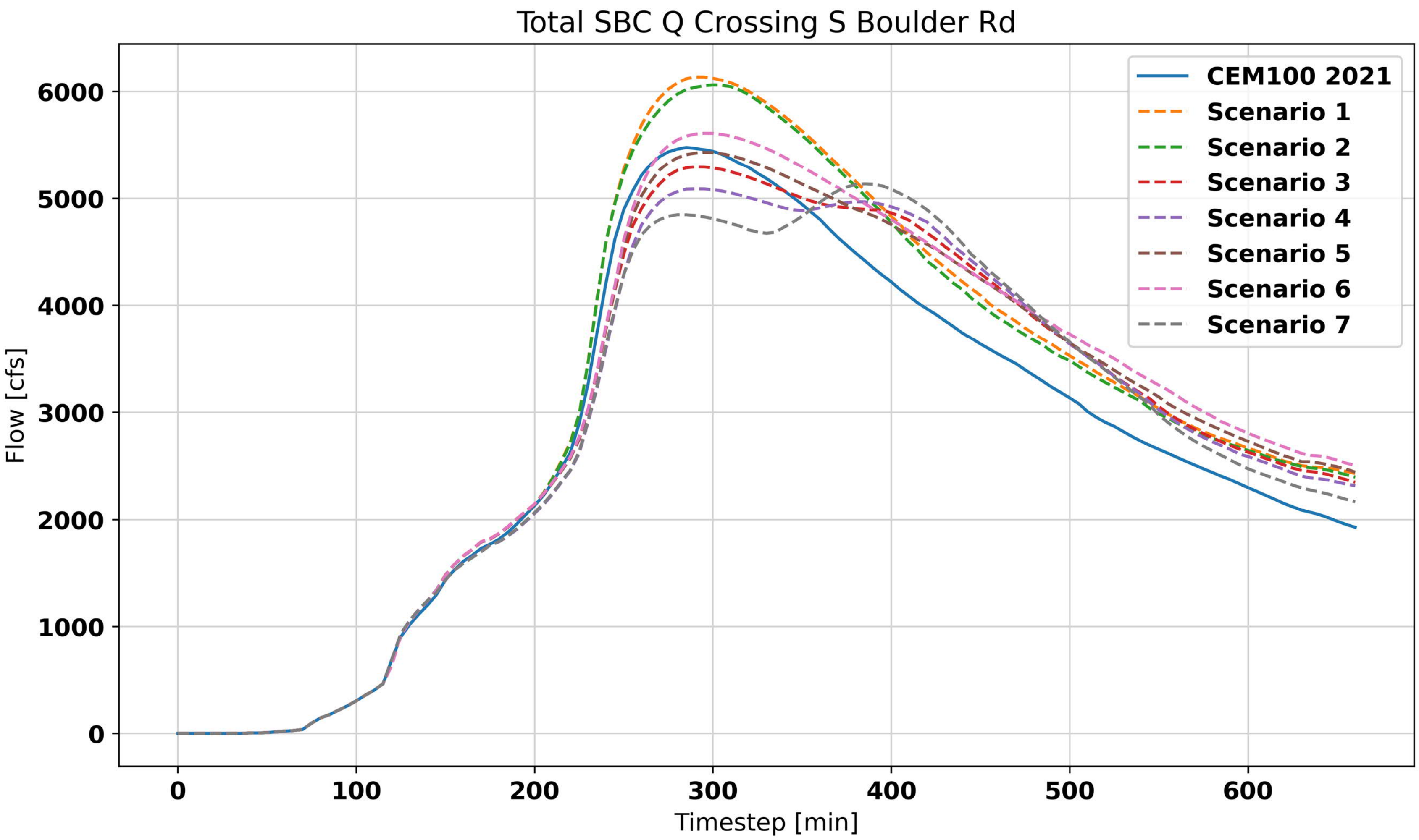

Figure 13 Flow at South Boulder Road in SBC corridor - 100-year Thunderstorm Event – Scenarios 1 to 7

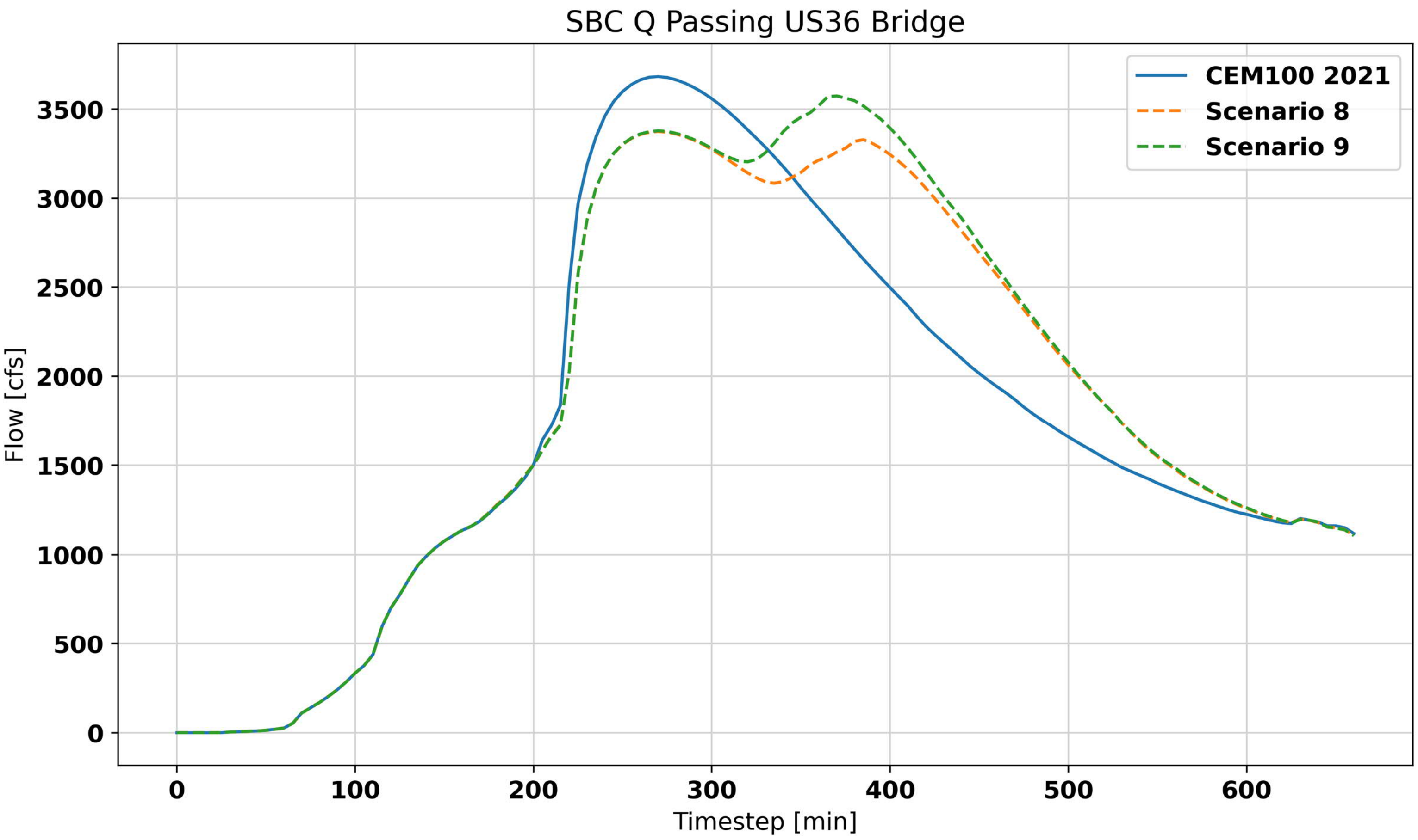

Figure 14 Flow Under US36 Bridge - 100-year Thunderstorm Event – Scenarios 8 and 9

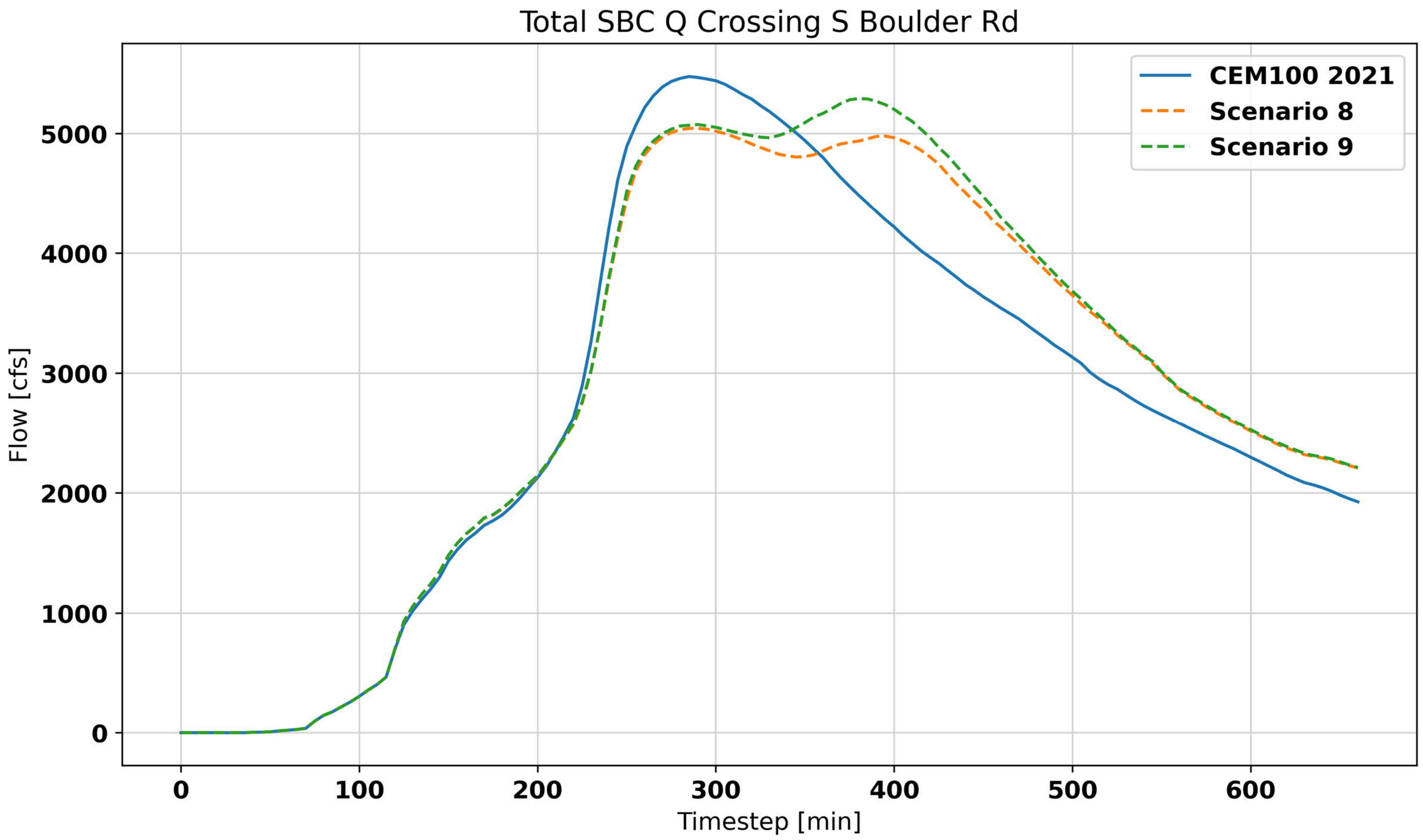

Figure 15 Flow at South Boulder Road in SBC corridor - 100-year Thunderstorm Event – Scenarios 8 and 9

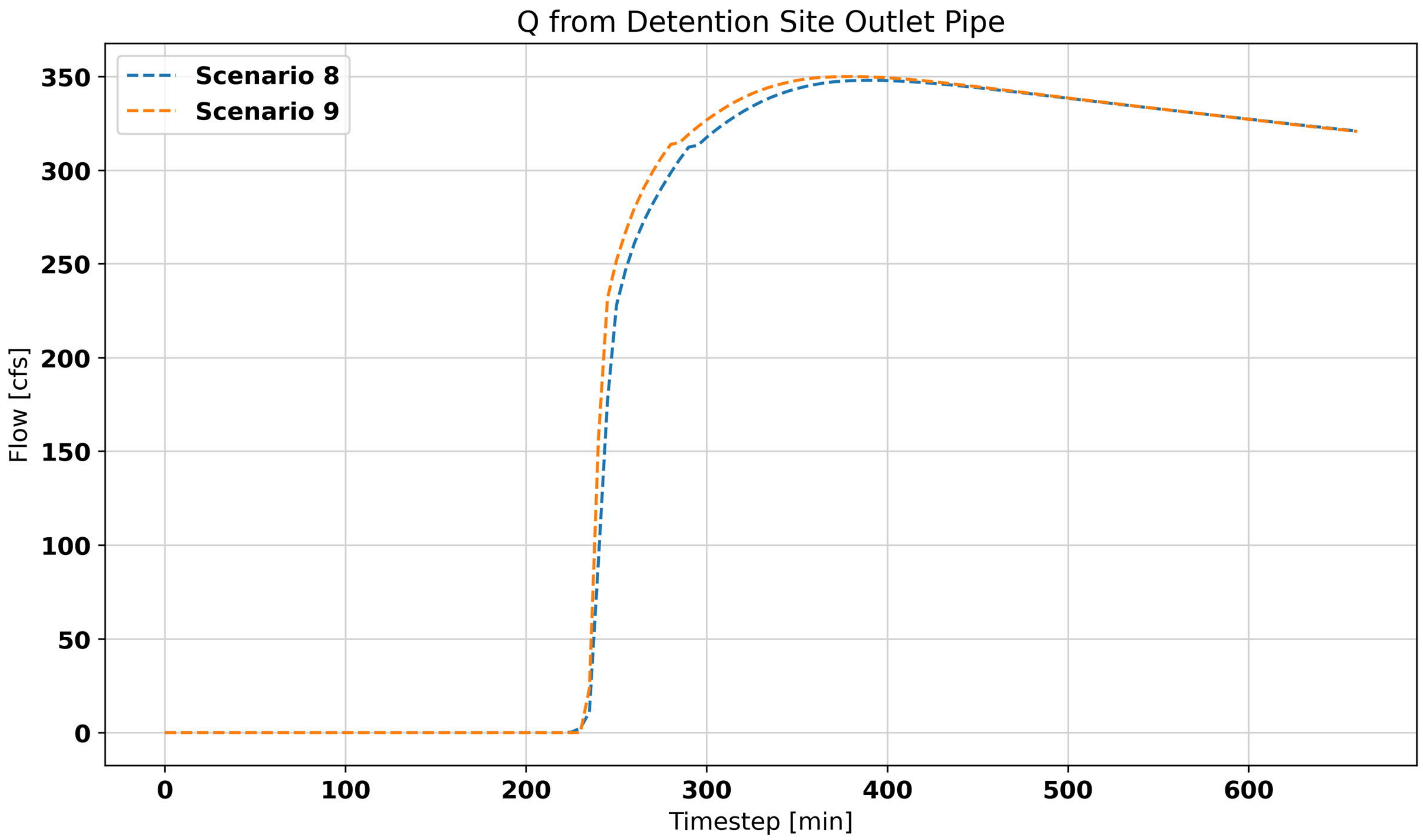

Figure 16 Flow through Outlet Works - 100-year Thunderstorm Event – Scenarios 8 and 9

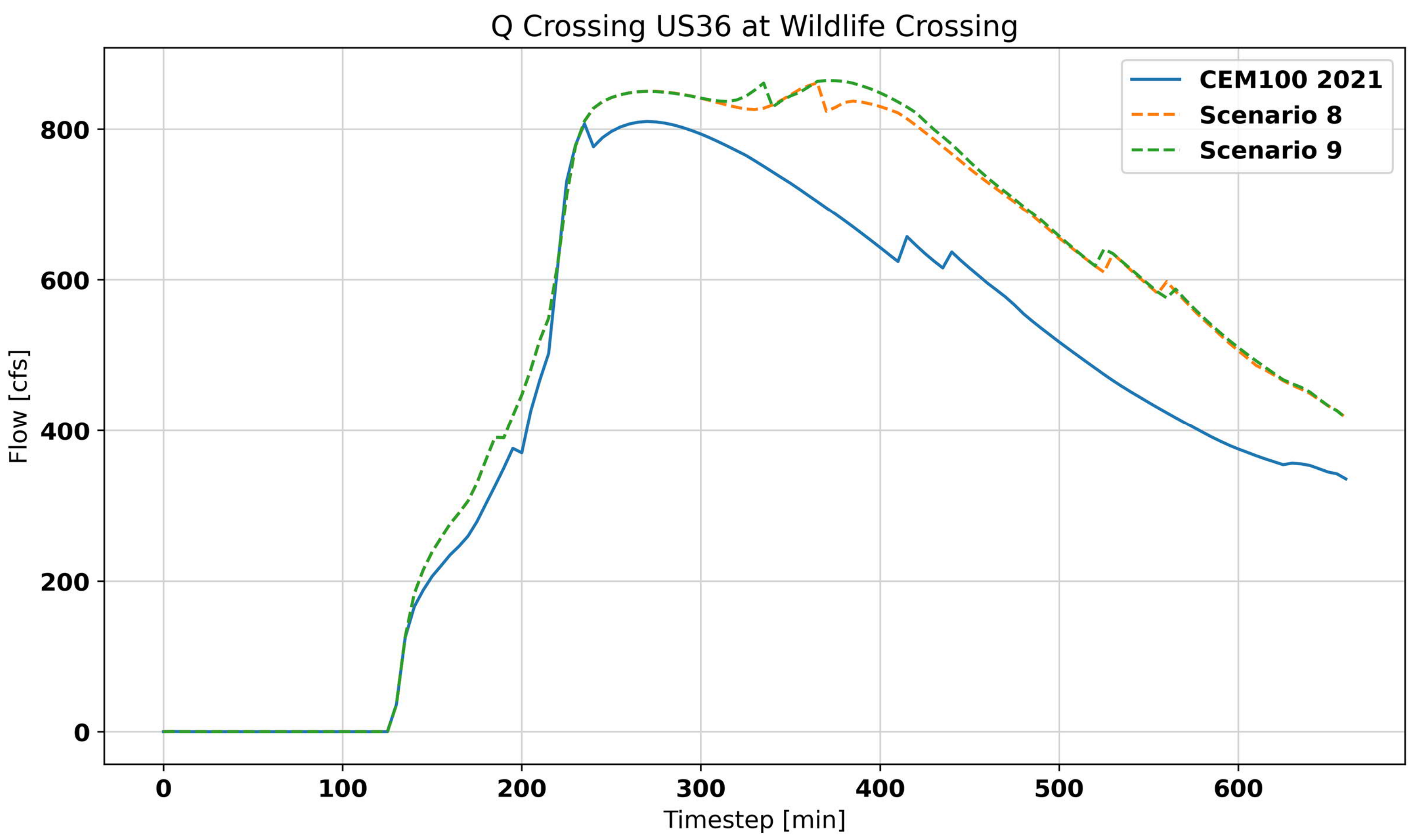

Figure 17 Flow through Wildlife Crossing - 100-year Thunderstorm Event – Scenarios 8 and 9

A.3 WSE Profiles

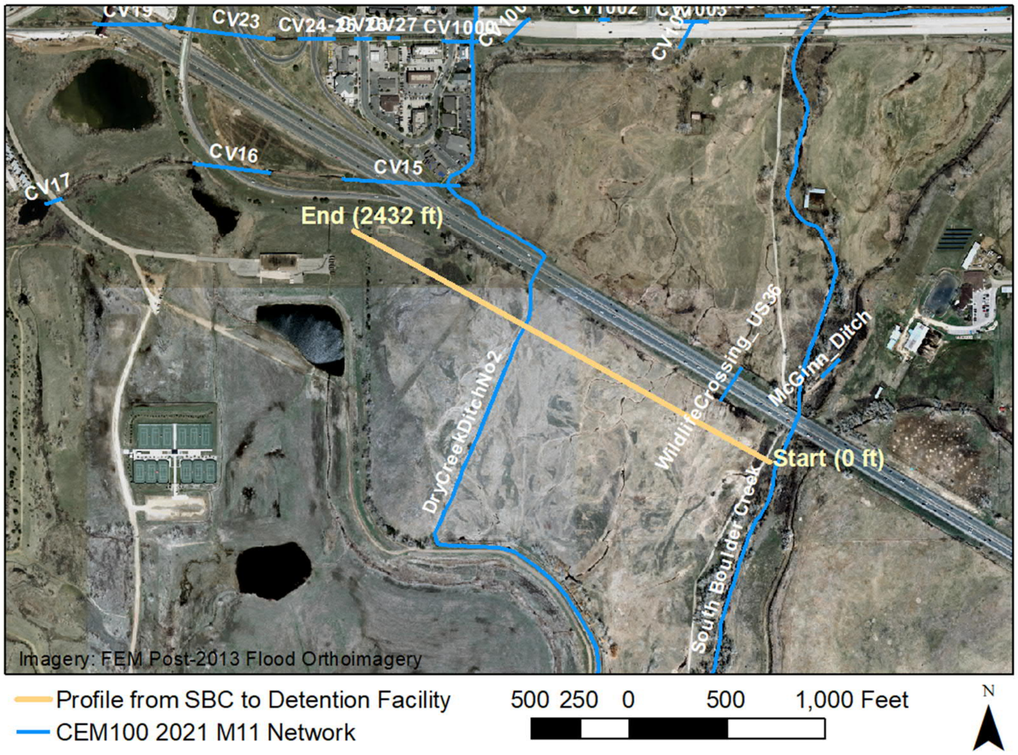

Figure 18 Location of Profile from SBC to Detention Facility

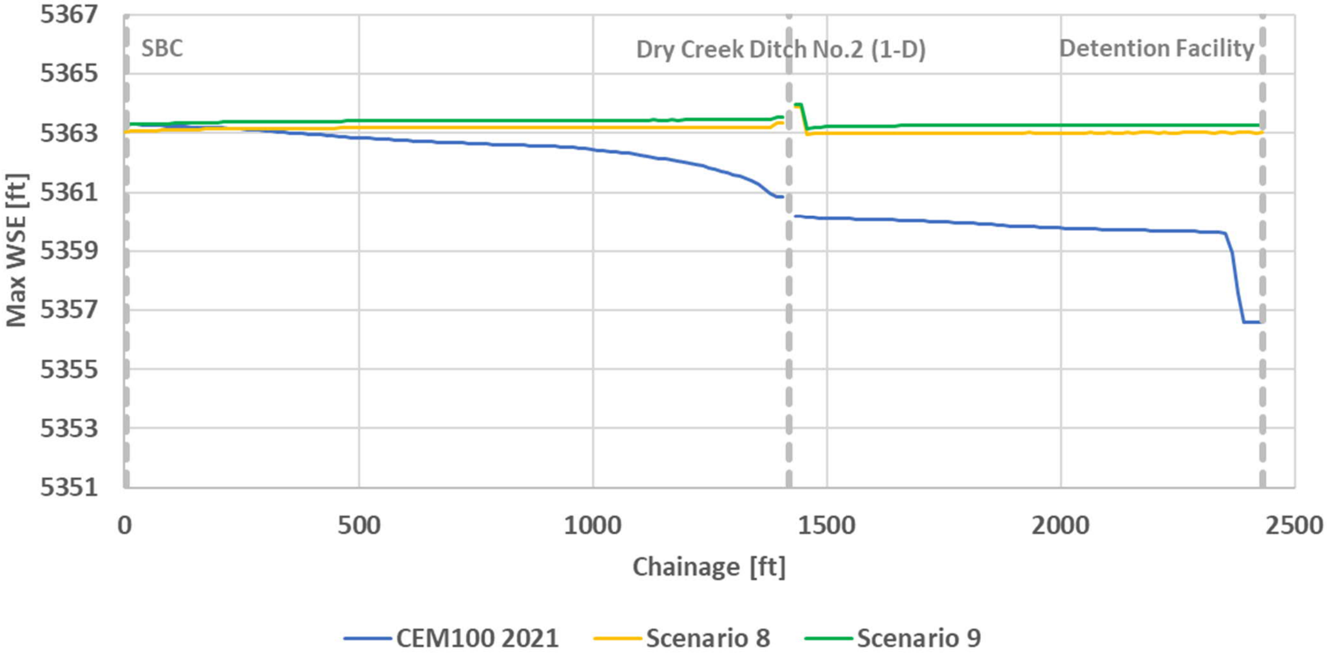

Figure 19 Maximum WSE Profile from SBC to Detention Facility - 100-year Thunderstorm Event – Scenarios 8 and 9

APPENDIX B.2

ALTERNATE CORRECTED EFFECTIVE MODEL

SOUTH BOULDER CREEK 2D HEC-RAS MODELING

TECHNICAL MEMORANDUM

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PREPARED FOR
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and City of Boulder, Colorado

MARCH 2022

TABLE OF CONTENTS

INTRODUCTION

The current Federal Emergency Management Agency (FEMA) effective model for South Boulder Creek was prepared using MIKEFLOOD hydraulic modeling software developed by DHI. The MIKEFLOOD model is a combination 1D/2D model and was used to prepare the South Boulder Creek floodplain delineation shown on the effective Flood Insurance Rate Map (FIRM) panels for the City of Boulder and unincorporated Boulder County.

Currently RJH, under contract to the City of Boulder is preparing a preliminary design for the South Boulder Creek Regional Detention Facility located just upstream of US Highway 36 (US 36). The facility is located within the floodplain created by the South Boulder Creek 1D/2D model. DHI as a subconsultant to RJH has created several hydraulic models in support of the preliminary design of the detention facility. One of the models they are creating is an Existing Conditions Model (ECM) using MIKEFLOOD. The MIKEFLOOD ECM is intended to update the effective model to capture several changes in the floodplain, updated model versioning, and current LiDAR data. Additionally, the ECM is a fully 2D model and eliminated the previous 1D sections found in the effective model.

The City of Boulder is interested in exploring the possibility of using HEC-RAS 2D as the modeling software for future FEMA floodplain updates for South Boulder Creek. HEC-RAS 2D is a free-to-use hydraulic modeling platform developed by the US Army Corps of Engineers that has been widely adopted by consultants, developers, and floodplain managers for FEMA floodplain mapping studies. The purpose of this study is to compare input, methodology, and output for the two models - 2D HEC-RAS and MIKE FLOOD. The following sections present a comparison of the two models and our recommendations moving forward.

FEMA 2D MODELING GUIDANCE

For this project RESPEC reviewed the current FEMA guidance on 2D hydraulic modeling presented in "Guidance for Flood Risk Analysis Mapping. Hydraulics: Two-Dimensional Analysis", dated December 2020. The modeling efforts by RESPEC fall in accordance with these guidelines. The guidelines are not specific to modeling approaches or decisions made by the modeler but focus more on general best practices and ensuring that submittals include well documented information on how the models were developed. Key parts of the document include ensuring that the surface used in the model has sufficient resolution for the model domain, considering whether advantages to 2D modeling over 1D modeling are sufficient to warrant developing a 2D model, appropriate levels of calibration to validate the models, and proper georeferencing of 2D models for review.

DATA RECEIVED

The following data was used to create the 2D HEC-RAS model:

- ECM MIKE FLOOD input from DHI (March 3, 2021)
- Manning's n raster from DHI (March 26, 2021).
- Digital elevation model (DEM) for the model domain between Highway 93 and Baseline Rd. from DHI (March 26, 2021)
    - Note that to account for input hydrographs located upstream on Highway 93 RESPEC combined the provided DEM with a 1-meter DEM taken from the USGS.
    - An additional DEM was provided to RESPEC on January 20, 2022, for the area upstream of US 36, encompassing approximately 0.8 square miles.
- Inflow hydrographs and locations from DHI (March 26, 2021)
- Letter of Map Revision (LOMR) Case No. 17-08-1389P including all back-ground 1D HEC-RAS modeling and floodplain mapping.

1D VS. 2D MODELING

Floodplain modeling can be done using either a 1D, 2D or combined 1D/2D approach. While it is more common to see 1D models in FEMA floodplain analysis, 2D modeling has become more accessible and shows potential for more accurately representing split flows throughout the model domain given the ability to account for horizontal flow movement. While 1D modeling relies on cross sections of a river to interpolate water surface elevation as flow progresses down a channel, 2D modeling allows for flow to be controlled modeled based off storage volume and elevation differences represented in a digital elevation model (DEM). This allows for a more complete picture of the floodplain, as flows are not constrained to user defined cross sections. More discrete changes in elevation can be captured by the model, allowing for potential split flows and ponding to be more accurately represented at each point in the model domain. 2D modeling also provides an easier user interface for the modeler, as over banks, channel obstructions, distance between cross sections, and cross section specific Manning's n values do not need to be assigned to each cross section but are instead accounted for by the terrain surface, Manning's n raster, and the mesh grid assigned to the model domain.

1D hydraulic modeling relies on interpolating the water surface elevation between 1D cross sections cut from a DEM, these cross sections are used by the model to convey information about each cross section (roughness, over banks, channel obstructions, etc.) to the floodplain. 2D modeling allows the user to define a mesh grid over the model domain, each cell in the mesh grid is used by the model to convey flow, accounting for volume, slope, roughness, a velocity of flow at each point in the model domain in both the downstream direction as well as a horizontal direction to the slope of the channel. Culverts in a 2D model are still modeled in 1D, and flow is passed between 2D sections of the model through these 1D elements. 2D modeling can more accurately represent flow into and out of hydraulic structure with additional details; flows, velocities, hydraulic grade lines, and energy grade lines are present in 2D models and can provide “reality checks” as well as aiding in calibration of the model. In 1D models, bridges are modeled similar to culverts, for this project the US 36 bridge located southeast of US 36 and Table Mesa Drive was modeled in 2D, with information about the bridge piers being stamped into the model mesh to properly convey flow around the bridge piers. Information for the bridge piers (width), was taken from the 2018 US 36 LOMR (Case #17-08-1389P).

HEC-RAS 2D INPUT

The following subsections describe the input used for the South Boulder Creek 2D HEC-RAS model:

Grid

RESPEC employed a variable size mesh grid throughout the model, this allows for more refinement in areas where higher accuracy is required and less in areas where water surface elevations and velocities are less critical to the project. Cell sizes through the model domain vary from 150’x150’ in the upper watershed outside of the channel to 25’x25’ upstream of the crossing at US 36. Employing a variable size mesh grid decreases model run time significantly, from approximately 30 hours to 9 hours. The model mesh is further refined throughout the study area using break lines to ensure that model mesh associated with levees, roadways, and structures is properly aligned to the direction of flow. The current FEMA standards do not provide guidance on cell size; however, the current Boulder Creek model cell size is 100’x100’. Figure 1 below shows an example of the variable mesh constructed for the model region associated with the crossing at US 36.

The MIKEFLOOD local model uses an unstructured variable mesh of 4-meters (13.12’) with 1–2-meter (3.28’ – 6.56’) mesh grid around US 36.

Equations

The HEC-RAS model uses full momentum equations, with the time step controlled by the Courant number. This allows the model to vary the time step higher or lower based on calculated errors during modeling.

The MIKEFLOOD model uses full momentum equations, DHI also uses the Courant condition to vary the timestep during model simulations.

Manning’s n

Manning’s n is a roughness coefficient used in both 1D and 2D hydraulic modeling. In 2D modeling Manning’s n is represented as a raster of points representing unique values at each cell in the raster. DHI provided RESPEC with the Manning’s n raster used in this modeling effort. The data provided describes vegetation as 0.06, and buildings as 0.08.

Boundary Conditions

The downstream boundary condition modeled by RESPEC is set to a normal depth measured from the surface provided to RESPEC by DHI. The slope for the normal depth was calculated along South Boulder Creek as it exits the model domain at Baseline Rd; the slope at this section of South Boulder Creek is 0.01 ft/ft.

The MIKEFLOOD local model uses a constant water depth of 1610 meters (5,282.2') ending at Baseline Rd.

Hydrograph Inputs

Hydrographs used in the HEC-RAS model were taken from CEM_BoundaryConditions_CDOT.xlsx sent to RESPEC by DHI on March 26, 2021. Hydrograph locations and peak flows from the hydrographs are shown in Figure 2 below.

Run Control Parameters

The HEC-RAS timestep is controlled by the Courant condition, this method allows the model to vary the time step by monitoring for instability. The timestep can be increased (to speed up runtime) or decreased (to reduce errors or numerical instability) during the simulation. The initial timestep is set to 1 minute with the maximum Courant number before halving the timestep set to 1, and the minimum Courant number before doubling the timestep set to 0.5. The Courant number is calculated as shown in Equation 1 below; U_{xy} is the magnitude of the velocity in the x and y directions, delta t is the timestep, delta x, y is the length interval as defined by the cell mesh size in the x and y direction.

$$C = \frac{U_x \Delta t}{\Delta x} + \frac{U_y \Delta t}{\Delta y} \leq C_{maximum} \qquad \text{Equation 1}$$

The maximum times a timestep can be halved is set to 6, allowing for a minimum timestep of 1.2 seconds. The maximum times a timestep can be doubled is set to 3, allowing for a maximum timestep of 8 minutes. The hydrograph output interval is set to 1 minutes, the detailed output interval is set to 1 hour, and the mapping output interval is set to 1 hour. The HEC_RAS model uses a mixed flow regime. Total runtime for the model is approximately 9 hours.

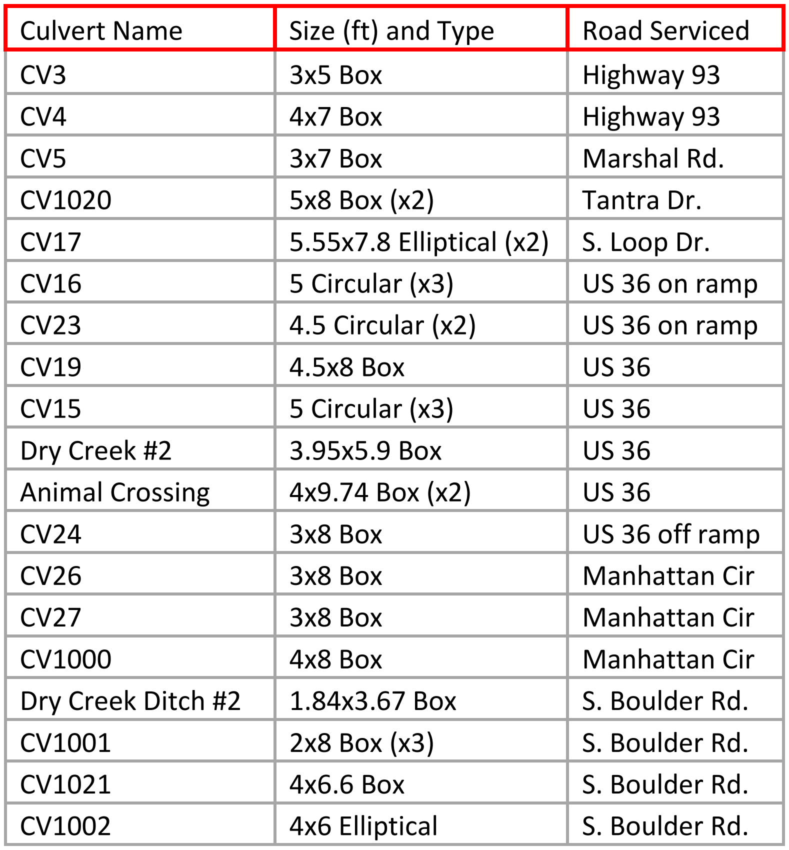
The MIKEFLOOD local model uses the Courant condition with a maximum Courant number of 0.8, with a maximum timestep of 1 second and a minimum timestep of 0.0001 seconds. Typical timesteps in the model vary between 0.17 and 0.01 second with an average timestep of 0.03 seconds.

Culverts and Bridges

Location and specific information for modeling culverts was taken from CEM_BoundaryConditions_CDOT.xlsx and the MIKEFLOOD model respectively. Table 1 below shows the culvert name and the road associated with each culvert.

The US 36 bridge is the only bridge simulated in the model that includes bridge piers, the surface associated with bridge located at South Boulder Road is channelized to allow flow to pass. Both RESPEC and DHI chose a full 2D solution for modeling the bridge along US 36, southeast of Table Mesa Drive. The piers were simulated with a 20 percent blockage to mimic potential blocking during a large event. The pier widths used by RESPEC were taken from the US 36 LOMR HEC-RAS model developed in 2018.

Table 1. Culvert Name and Location



Culvert Name	Size (ft) and Type	Road Serviced
CV3	3x5 Box	Highway 93
CV4	4x7 Box	Highway 93
CV5	3x7 Box	Marshal Rd.
CV1020	5x8 Box (x2)	Tantra Dr.
CV17	5.55x7.8 Elliptical (x2)	S. Loop Dr.
CV16	5 Circular (x3)	US 36 on ramp
CV23	4.5 Circular (x2)	US 36 on ramp
CV19	4.5x8 Box	US 36
CV15	5 Circular (x3)	US 36
Dry Creek #2	3.95x5.9 Box	US 36
Animal Crossing	4x9.74 Box (x2)	US 36
CV24	3x8 Box	US 36 off ramp
CV26	3x8 Box	Manhattan Cir
CV27	3x8 Box	Manhattan Cir
CV1000	4x8 Box	Manhattan Cir
Dry Creek Ditch #2	1.84x3.67 Box	S. Boulder Rd.
CV1001	2x8 Box (x3)	S. Boulder Rd.
CV1021	4x6.6 Box	S. Boulder Rd.
CV1002	4x6 Elliptical	S. Boulder Rd.

Figure 2 below shows locations and peak flows for hydrographs used in the HEC-RAS model.

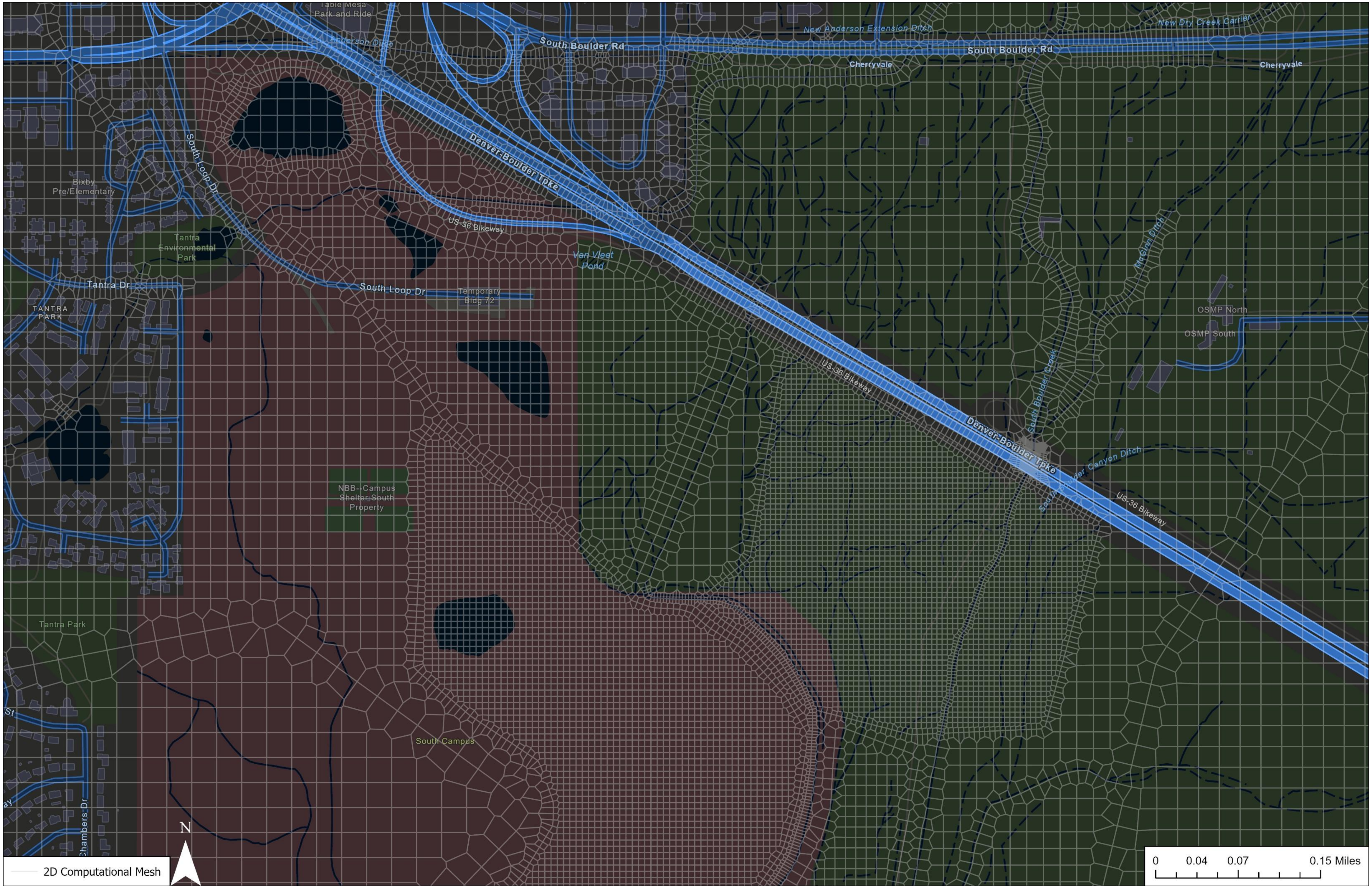
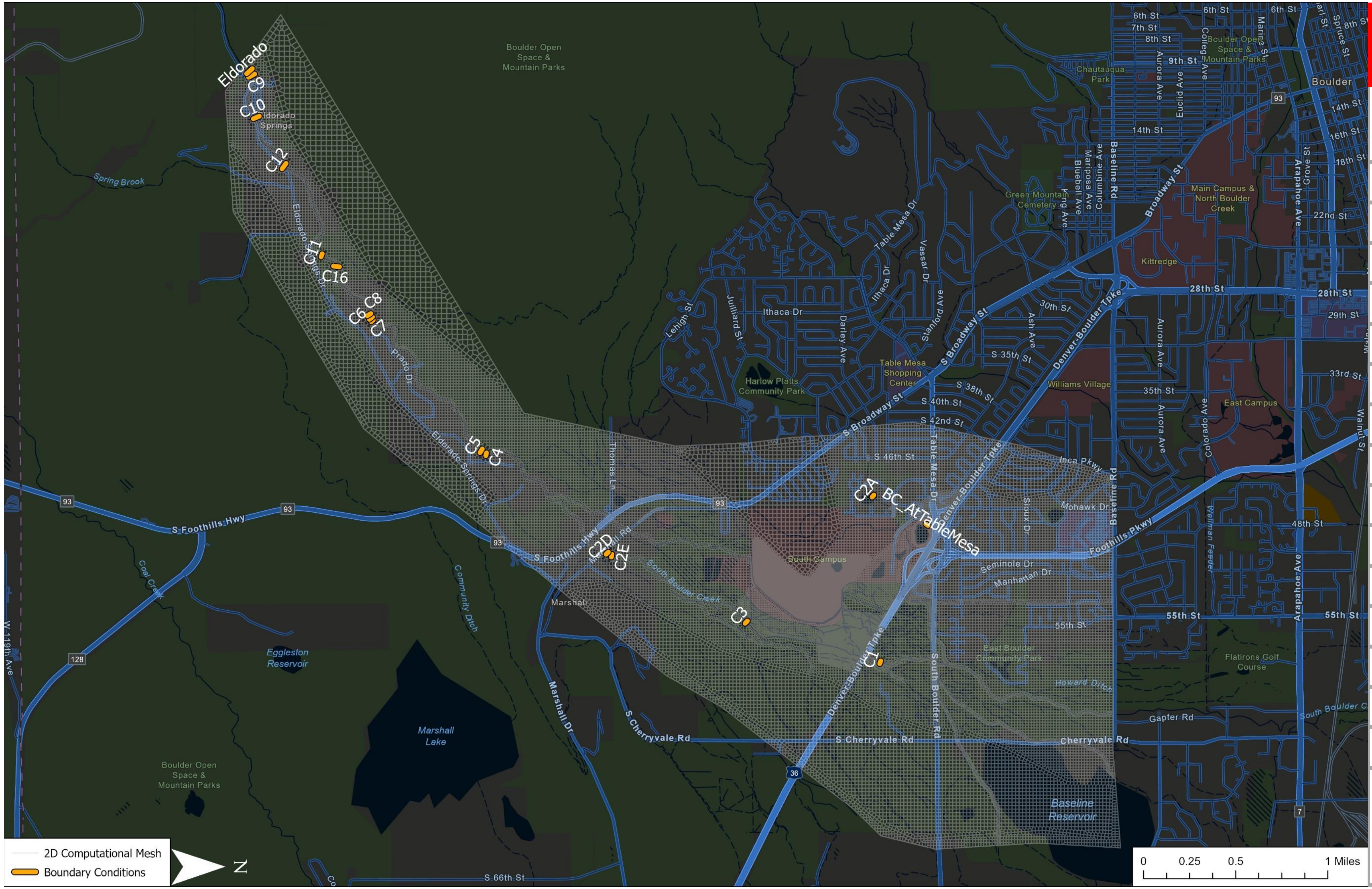


Figure 1. Variable mesh grid used to refine the model domain around US 36.



Input Name	Peak Flow (cfs)
Eldorado Springs	4521
C10	527
C9	151
C12	154
C11	424
C16	374
C8	77
C7	209
C6	162
C4	198
C5	434
C2D	233
C2E	71
C3	685
C1	454
C2A	690
BearCreek at Table Mesa	110

Figure 2. Hydrograph peak flow locations.

Comparisons between the MIKEFLOOD model and the HEC-RAS model were made by assessing peak flows at specific locations in the models as well as comparing maximum-depth rasters generated for each model. Table 2 below shows peak flows taken from the specified locations, as well as the difference and the percent difference between the model results at the defined locations. Figures 3A and 3B show the locations of these comparisons . CV19, CV15, and wildlife crossing in Table 2 refer to culverts in the model domain, while the other locations reference profile lines in the model domain.

The bridge located along South Boulder Road (approximately 1800' east of Manhattan Drive and South Boulder Road) is shown in Table 2 for both the MIKEFLOOD local model and the MIKEFLOOD corrected effective model.

Table 2. Computed 100-year peak flow difference between models.

Peak Flow Location	**MIKEFLOOD (cfs)**	**HEC-RAS (cfs)**	**Difference (cfs)**	**% Difference**
Approaching HWY 93	6643	7125	482	6.8
Approaching US36	7159	7370	211	2.9
US36 Bridge	3811	3906	95	2.4
US36 Split	2704	2795	91	3.3
Overtopping US36	2338	2423	85	3.5
CV19 (Anderson Ditch at US36)	191	227	36	18.8
CV15 (Viele Channel at US36)	803	742	-61	-7.6
S. Boulder Road Bridge Local Model	3076	2899	-177	-5.7
S. Boulder Road Bridge CEF Model	2734	2899	165	6.0
Wildlife Crossing	856	751	-105	-12.3

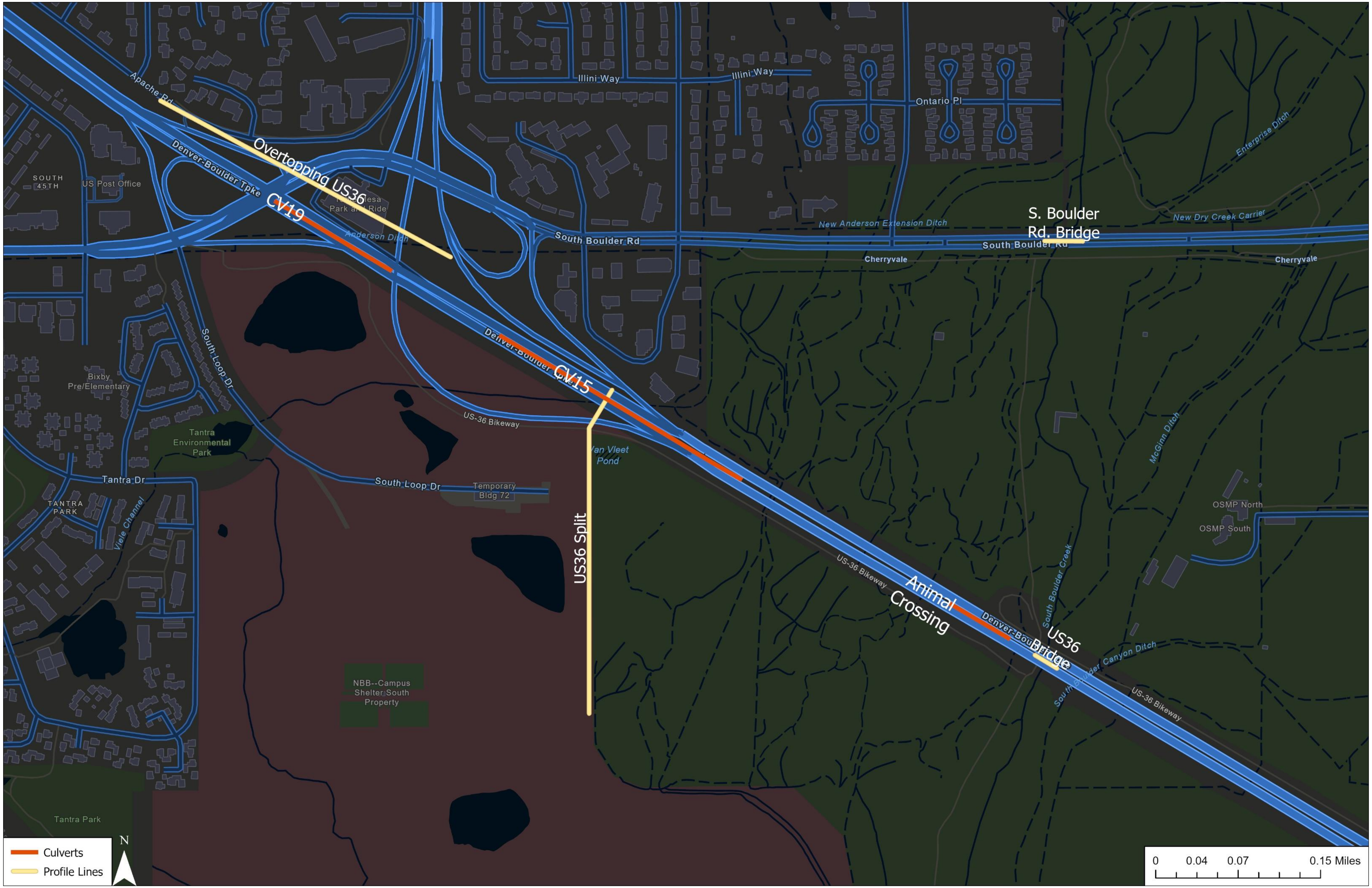


Figure 3A. Model comparison points.

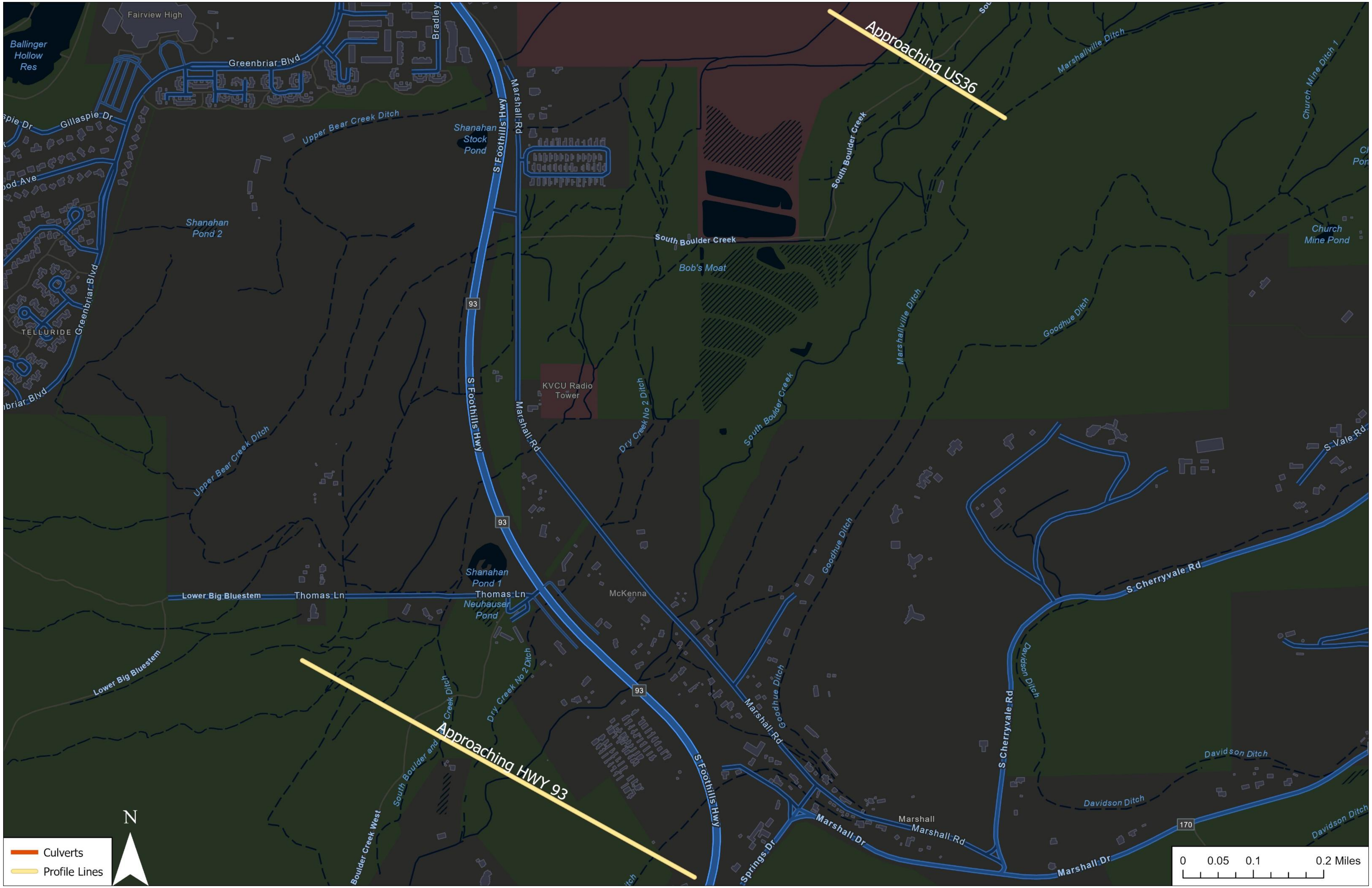


Figure 3B. Model comparison points.

Using the depth raster provided by DHI, RESPEC created a difference raster showing depth difference between the two models. Figure 4 below shows this difference raster. To more easily interpret the results of the raster, colors were assigned to groups of values representing a range of depth differences. The difference raster was calculated by subtracting the depth raster produced by the HEC-RAS model from that of the MIKEFLOOD model; negative values indicate that depths in the MIKEFLOOD raster are greater than those in the HEC-RAS raster, and positive values indicate that depths in the HEC-RAS raster are greater than those in the MIKEFLOOD raster.

Figure 4. Difference raster, HEC_RAS - MIKEFLOOD

RESULTS COMPARISON

Figure 5 below shows the statistical breakdown of the difference raster; the mean value of the difference raster between the HEC-RAS model and the MIKEFLOOD model is 0.00048′ with a standard deviation of 0.56′. As shown in Figure 5, the vast majority of the of the model domain between the two models matches very closely. As shown on Figure 4 above, there are two locations with depth differences greater than 2′ between the MIKEFLOOD results and the HEC-RAS results, these areas are shown in red and green on Figure 4. Location 2 in Figure 4 shows average depth differences of -1.1′.

Location 1 on Figure 4 has an average difference of -2.5′; this area is located north of South Boulder Creek, west of Highway 93. This region is just downstream of where the MIKEFLOOD model transitions from a 1D model to a 2D model. The HEC-RAS model starts approximately 2.6 miles upstream of the 2D portion of the MIKEFLOOD model. In general, 2D models better consider floodplain storage/attenuation which naturally result in differences in water surface elevations and flows. Figure 6 below illustrates the floodplain differences upstream of Highway 93 between the HEC-RAS results and the MIKEFLOOD results. As shown on Figure 6, flow in the HEC-RAS results layer extends further north initially than the MIKEFLOOD results, given the difference in storage/attenuation between 1D and 2D models, it is not surprising to see differences in this region of the model.

Culvert crossings at Highway 93 in the MIKEFLOOD model are simulated using a 1D approach, because of this, the depth raster provided by DHI does not show depth information in Figure 6 upstream of downstream of the culvert crossings.

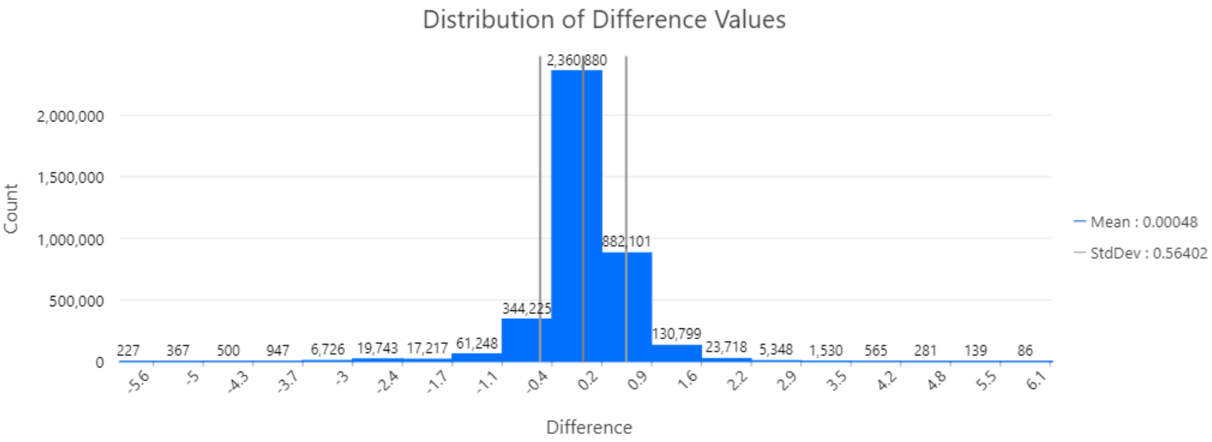


Figure 5. Difference Raster Statistics.

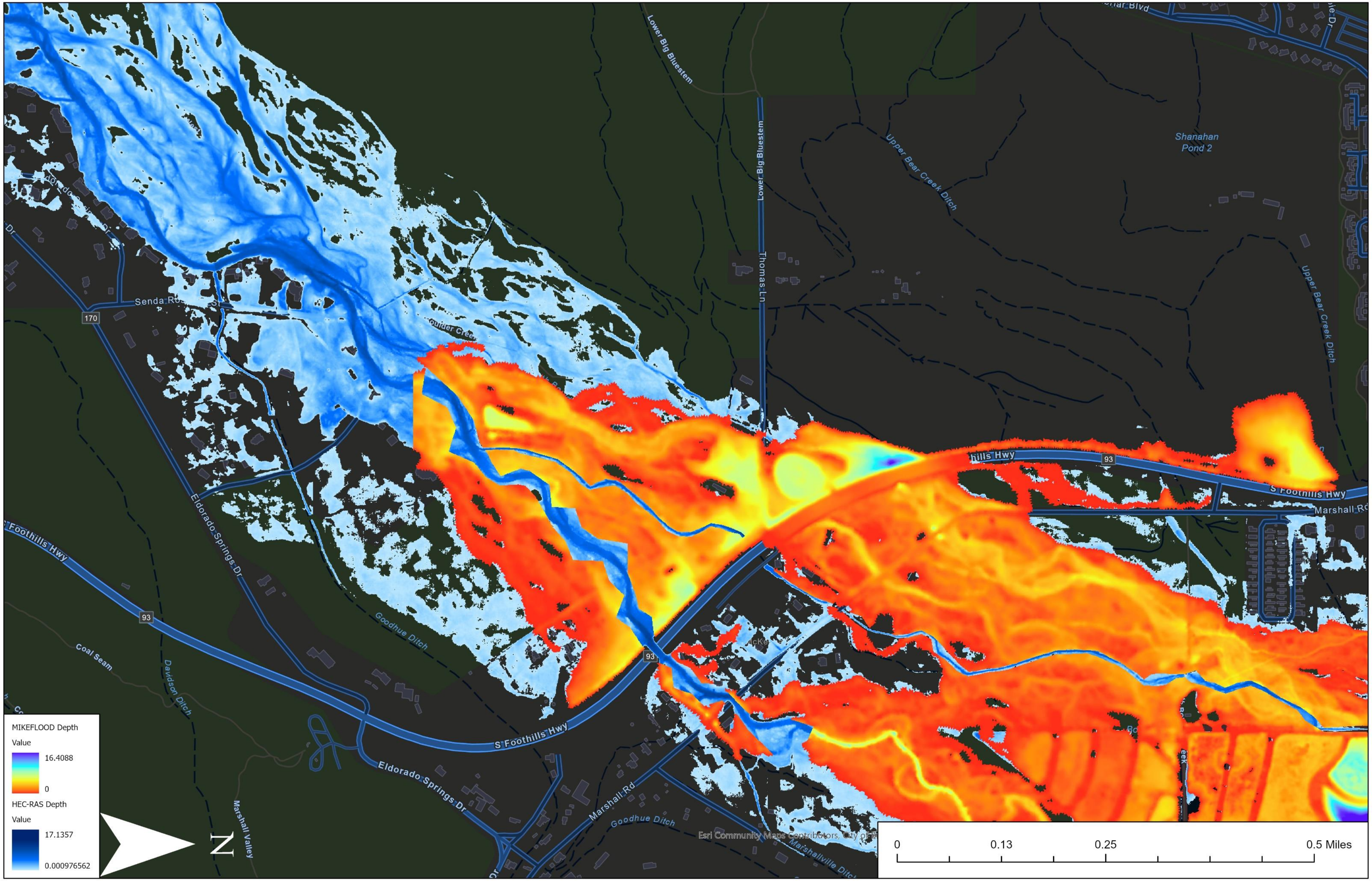


Figure 6. HEC-RAS results vs. MIKEFLOOD results upstream of Highway 93 (Location1)

Location 2 shown on Figure 4 above has an average difference of -1.1′ over topping US 36. The MIKEFLOOD model uses a much smaller grid size in this location than the HEC-RAS model, 30′x30′ in the HEC-RAS model compared with between 3′ and 6′ cell spacings in the MIKEFLOOD model. This difference in cell size around this hydraulically complex section of the model could account for depth difference in the models. While there are differences in the reported depth between the two models, peak flows through the region are relatively similar when considering over topping of US 36 in this area.

Location 3 shown in Figure 4 above, located south of South Boulder Road is a point in the MIKEFLOOD model where flow is again transitioned back into a 1D model. The MIKEFLOOD results show considerably more flow over topping South Boulder Road than the HEC-RAS model at location 3. Figure 7 below shows the results layers from the HEC-RAS model and the MIKEFLOOD model at this location The MIKEFLOOD model shows a maximum depth overtopping South Boulder Road of approximately 5.5′ while the HEC-RAS model shows a max depth of approximately 2.5′. Like the area upstream of Highway 93, the transition from 1D to 2D could impact attenuation/ storage, leading to the difference in results.

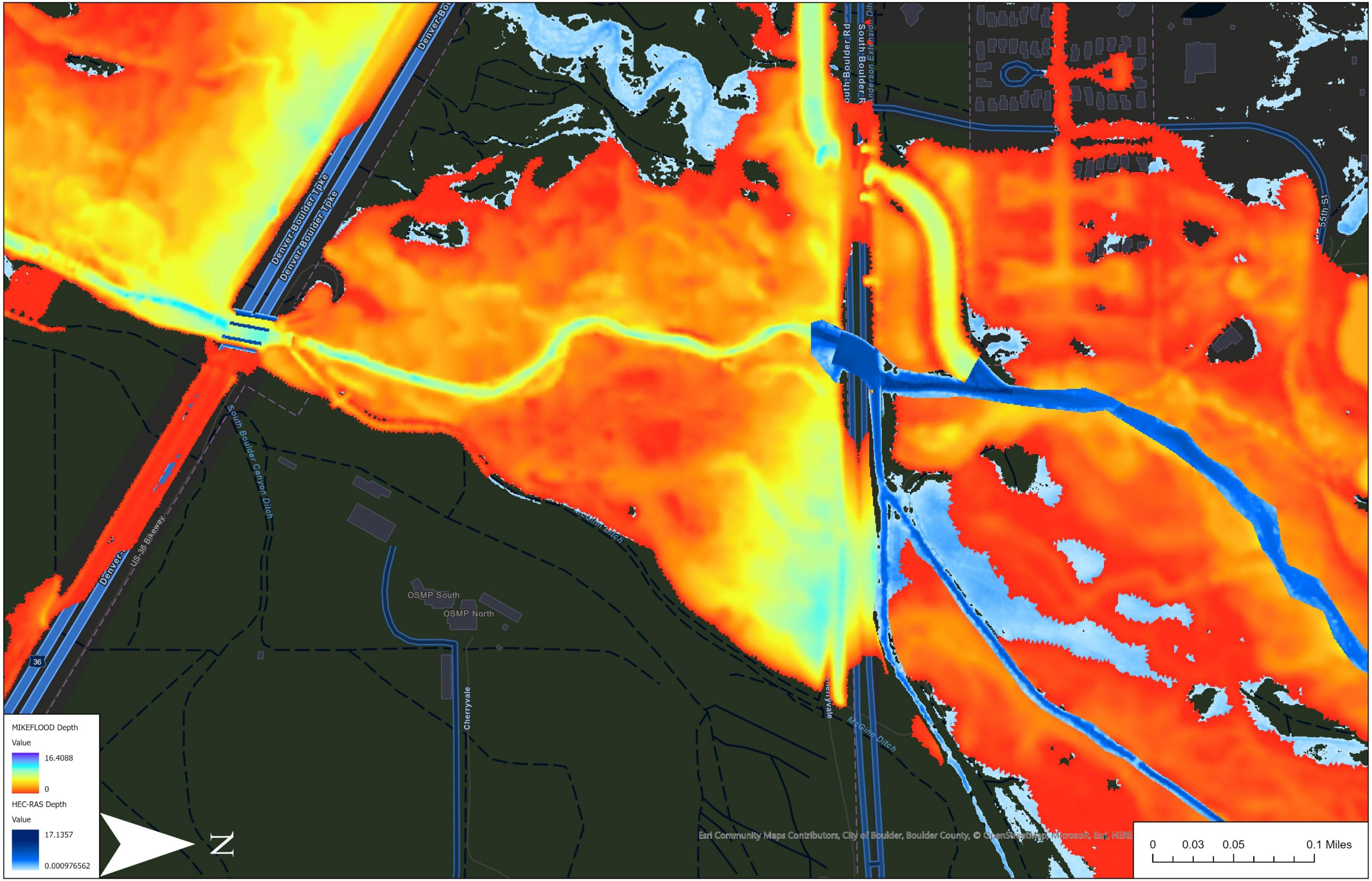


Figure 7. Results comparison at South Boulder Road. (Location 3)

POTENTIAL CALIBRATION METHODS

Table 2 indicates that flow differences exist through major points in the model, calibration of the model could potentially reduce these flow differences. Potential points of calibration are outlined below.

- Calibrate weir coefficients throughout the HEC-RAS model.
  - Culvert flow shows the greatest differences between the two models. For this study, the culverts were not calibrated. Calibrating the culverts could decrease the discrepancies between the two models.

The surface information where Viele channel crosses US 36 required considerable modification for the culverts to properly function. This is an area where the models differ by a large extent from one another. Additionally, there appears to be stormwater infrastructure present in this location not reflected in the HEC-RAS model. Calibrating the surface changes between models, and accounting for the storm network present in the area could provide additional calibration to the HEC-RAS model.

CONCLUSION

The mean difference between the MIKEFLOOD flood depths and the HEC-RAS flood depths is 0.00048′ across the regions where the model results overlap with a standard deviation of 0.56′. Grid cell resolution could account for some of the discrepancies between the models, additionally, the inherent differences between a 1D/2D approach and a full 2D approach to modeling the floodplain could potentially influence the depth differences shown in Figure 4. Floodplain extent between the two models is very similar, with only slight differences between the models showing up in the floodplain itself. We conclude that the resulting floodplain delineation and 100-year water surface elevations using the HEC-RAS 2D model are comparable, within reason, to those determined using MIKEFLOOD. Additionally, we conclude that HEC-RAS 2D could be a reasonable model to use for FEMA floodplain delineation and regulation in the City given sufficient culvert and surface calibration.

APPENDIX C

GEOTECHNICAL ANALYSES

APPENDIX C.1

EMBANKMENT SEEPAGE AND STABILITY MATERIAL PROPERTIES

TECHNICAL MEMORANDUM

Project No. 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Adam Prochaska, P.E., Ph.D., P.G. - RJH Consultants, Inc.**

DATE: **July 19, 2021**

SUBJECT: **South Boulder Creek Regional Detention Project**
Embankment Seepage and Stability Material Properties

1.0 Introduction

1.1 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present material properties used for seepage and stability analyses for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City).

Project-specific geotechnical data and typical published values for similar materials were reviewed to estimate appropriate values required as inputs into GeoStudio, a multifunctional computer modelling software.

1.2 Methodology

Materials properties were estimated using the following resources:

- Field and laboratory test results obtained during RJH's Phase II geotechnical investigations.
- A summary of fill material properties from RJH's Phase I and Phase II geotechnical investigations.
- A typical cross section of the embankment in Attachment A.
- Typical published values for similar materials.

1.3 Summary of Material Properties

A summary of the material properties that will be used for seepage and stability modeling are presented in Table 1.

TABLE 1
EMBANKMENT MATERIAL PROPERTIES FOR SEEPAGE AND STABILITY ANALYSES

	Foundation Soil	Embankment Shell	Embankment Filter	Embankment Core	Pierre Shale	Barrier Wall
Moist Unit Weight (pcf)	119	121	114	128	138	120
Saturated Unit Weight (pcf)	130	129	132	132	139	120
Drained Friction Angle, Φ (deg.)	25	28	36	28	40	8
Drained Cohesion (psf)	0	0	0	0	0	0
Total Stress Friction Angle (deg.)	17	17	NA(3)	17	0	0
Undrained Cohesion (psf)	31	31	NA	31	4000	50
Bilinear Normal(1) (psf)	180	130	NA	130	4767	356
Vertical Hydraulic Conductivity, K_v (cm/s)	2E-03	5E-03	3E-02	7E-06	1E-07	1E-07
Anisotropic Ratio (Kv/Kh)	0.1	0.2	0.5	0.14	1	1
Horizontal Hydraulic Conductivity, K_h (cm/s)	2E-02	3E-02	6E-02	5E-05	1E-07	1E-07
Porosity, η	0.36	0.38	0.35	0.35	0.28	0.48
GeoStudio Material(2)	Sand	Sand	Sand	Clay	Clay	Clay

Notes:

1. Bilinear normal is the normal stress where the drained and undrained strength envelopes intersect.
2. Volumetric water content functions and unsaturated hydraulic conductivity functions were estimated using these example materials that are built-in to the GeoStudio database.
3. NA means the property is not applicable for the material type.

2.0 Development of Material Properties

2.1 General

The embankment is anticipated to include a central core, upstream and downstream shells, and a downstream filter/drain zone. The embankment core is anticipated to be composed of clay or clayey sand material derived from existing onsite fill. The exact core configuration has not yet been selected. The embankment shell will consist of random fill that could range from fine grained to granular material encountered in excavations onsite. Earthfill for the embankment core and shells will need to be imported from offsite if sufficient quantities of onsite borrow is not available. A commercial sandy material will likely need to be imported for the embankment filter/drain.

Foundation materials beneath the embankment are anticipated to consist of existing fill (mostly clayey sand) throughout most of the embankment alignment and alluvium (generally sand and gravel) beneath the right abutment. Pierre Shale underlies the surficial soils. We anticipate a soil-bentonite barrier wall will be constructed through the foundation soils to provide a seepage barrier.

The following phase relationships were used to calculate unit weight and porosity values.

$$\gamma_m = \gamma_d * (1 + w) \qquad \text{(Equation 1)}$$

$$\gamma_s = (1 - 1/G_s) * \gamma_d + \gamma_w \qquad \text{(Equation 2)}$$

$$n = ((G_s * \gamma_w / \gamma_d) - 1) / (1 + ((G_s * \gamma_w / \gamma_d) - 1)) \qquad \text{(Equation 3)}$$

A specific gravity, G_s, of 2.7 was used when calculating the saturated unit weight and porosity of materials.

2.2 Foundation Soil

The foundation soil along the embankment alignment includes both fill and alluvium. Historical records documenting fill placement activities have not been identified. Typical fill materials were most commonly clayey sand or lean clay with low plasticity fines. Fine grained fill ranged from very soft to very stiff and was mostly very stiff. SPT N-values ranged from 1 to 71 and were most commonly 11 to 31, however SPT tests also frequently encountered refusal. Typical alluvial materials include poorly graded sand and poorly graded sand with silt and gravel.

To be conservative, we developed strength properties that would be representative of clayey sand, and we developed hydraulic conductivity properties that would be representative of sand and gravel. Material properties for the foundation soil were developed based on Phase II laboratory tests and material properties used in the Project groundwater model.

Moist Unit Weight
The dry unit weight of five samples tested in the foundation soil averaged 107 pcf. The moisture content of the five samples averaged 10.8 percent. Using the averaged values and Equation 1, a moist unit weight of 119 pcf was selected.

Saturated Unit Weight
Using the average dry unit weight of 107 pcf and Equation 2, a saturated unit weight of 130 pcf was selected.

Drained (Effective Strength Parameters)
Based on the plasticity results of samples in the fill, the sample with the highest plasticity index (ignoring an outlier point) was 25. Using published data relating the plasticity index to the friction angle for clays, we selected a friction angle of 25 degrees. The friction angle calculated from this approach is more conservative than the triaxial test results from onsite fill. This level of conservatism is appropriate for this level of design.

A drained cohesion of 0 psf was selected. Interpretation of triaxial data is presented in the Phase II Geotechnical Data Report (RJH, in progress).

Undrained Strength
Based on the triaxial tests performed on three specimens of clayey sand with gravel fill, an undrained cohesion of 31 psf and a total stress friction angle of 17 degrees was selected.

Interpretation of triaxial data is presented in the Phase II Geotechnical Data Report (RJH, in progress).

Composite Strength
We used a composite strength envelope for foundation soil. The bilinear normal (normal stress where the drained and undrained strength envelopes intersect) is 180 psf.

Vertical Hydraulic Conductivity
The hydraulic conductivity of foundation soil was developed for sand and gravel. A vertical hydraulic conductivity of $2x10^{-3}$ cm/s was selected to be consistent with material properties used for Alluvium Zone A in the preliminary baseline groundwater model.

Anisotropic Ratio
An anisotropic ratio (Kv/Kh) of 0.1 was selected to be consistent with the value used for Alluvium Zone A in the preliminary baseline groundwater model.

Horizontal Hydraulic Conductivity
Hydraulic conductivities are consistent with the material properties used for Alluvium Zone A in the preliminary baseline groundwater model. We selected a horizontal hydraulic conductivity of $2x10^{-2}$ cm/s.

Porosity
The porosity was selected to be 0.36 based on Equation 3 and the dry unit weight of 107 pcf.

2.3 Embankment Shell

The embankment shell is anticipated to be constructed with on-site borrow material that could range from fine grained to coarse grained material. To be conservative, we developed strength properties that would be representative of fine-grained fill, and we developed hydraulic conductivity properties that would be representative of coarse-grained fill.

Moist Unit Weight
Based on typical published values for a compacted poorly graded sand, we selected a dry unit weight of 105 pcf and a moisture content of 16 percent. The selected dry unit weight corresponds to approximately 95 percent compaction and a maximum dry unit weight of 110 pcf. Using the dry unit weight, moisture content, and Equation 1, we selected a moist unit weight of 121 pcf. Selected values are also representative of clayey fill.

Saturated Unit Weight
Using a dry unit weight of 105 pcf and Equation 2, a saturated unit weight of 129 pcf was selected.

Drained (Effective) strength parameters
Using typical published values for a clayey material, a friction angle of 28 degrees and a drained cohesion of 0 psf was selected.

Undrained Strength
The embankment shell might be similar material to the foundation soil depending on onsite material availability. A total stress friction angle of 17 degrees and an undrained cohesion of 31 psf were selected. These values are the same as those selected for the foundation soil. In our opinion this is conservative because the embankment will likely be compacted to a higher degree than the existing onsite fill.

Composite Strength

A composite strength envelope for the embankment shell was used. The bilinear normal is 130 psf.

Vertical Hydraulic Conductivity

Using typical published values for sandy embankment shell material, a vertical hydraulic conductivity of $5x10^{-3}$ cm/s was selected.

Anisotropic Ratio

Using typical published values, an anisotropic ratio of 0.2 was selected.

Horizontal Hydraulic Conductivity

Using the vertical hydraulic conductivity and the anisotropic ratio, the horizontal conductivity was calculated to be $3x10^{-2}$ cm/s.

Porosity

Using a dry unit weight of 105 pcf and Equation 3, a porosity of 0.38 was selected.

2.4 Filter

Filter and drain materials will be modeled as a homogenous zone for simplicity. Properties for the filter material were estimated from published mechanical properties of ASTM C33 fine aggregate.

Moist Unit Weight

Based on published values for the ASTM C33 fine aggregate, a maximum dry unit weight of 111 pcf and an optimum moisture content of 3 percent was selected. Using Equation 1, a moist unit weight of 114 pcf was selected.

Saturated Unit Weight

Using 111 pcf as the dry unit weight of concrete sand and Equation 2, the saturated unit weight of 132 pcf was selected.

Drained (Effective) Strength Parameters

Based on published values, a friction angle of 36 degrees and a drained cohesion of 0 psf was selected.

Undrained Strength

This value is not applicable for sandy materials.

Composite Strength

This value is not applicable for sandy materials.

Vertical Hydraulic Conductivity

Based on published values and engineering experience, a vertical hydraulic conductivity of $3x10^{-2}$ cm/s was selected.

Anisotropic Ratio:

The published range of values for the anisotropic ratio for a compacted drain is 1 to 0.25. We selected 0.5 as the anisotropic ratio.

Horizontal Hydraulic Conductivity
Use the vertical hydraulic conductivity and the anisotropic ratio to calculate the horizontal hydraulic conductivity. The calculated horizontal hydraulic conductivity is 6×10^{-2} cm/s.

Porosity
Using a dry unit weight of 111 pcf and Equation 3, a porosity of 0.35 was selected.

2.5 Embankment Core

The embankment core is anticipated to be constructed of clay to clayey sand material. Material properties were developed using typical published values for a clayey sand.

Moist Unit Weight
Based on published values, a dry unit weight of 110 pcf and a moisture content of 16 percent were selected. The selected dry unit weight corresponds to approximately 95 percent compaction and a maximum dry unit weight of 116 pcf. Using Equation 1, a moist unit weight of 128 pcf was selected.

Saturated Unit Weight
Using a dry unit weight of 110 pcf and Equation 2, a saturated unit weight of 132 pcf was selected.

Drained (Effective) Strength Parameters
Using typical published values, a friction angle of 28 degrees and a drained cohesion of 0 psf were selected. These values are representative of a lean clay and are conservative for a clayey sand.

Undrained Strength
The embankment shell might be similar material to the foundation soil depending onsite material availability. A total stress friction angle of 17 degrees and an undrained cohesion of 31 psf were selected. These values are the same as those selected for the foundation soil. In our opinion this is conservative because the embankment will likely be compacted to a higher degree than the existing onsite fill.

Composite Strength
A composite strength envelope for the embankment core was used. The bilinear normal is 130 psf.

Vertical Hydraulic Conductivity
Based on published data for an embankment core, we selected a vertical hydraulic conductivity of 3×10^{-6} cm/s. This is the initial selection for the vertical hydraulic conductivity of intact core. This was subsequently adjusted to account for cracking as described in the following sections and a vertical hydraulic conductivity of 7×10^{-6} cm/s was used for analyses.

Anisotropic Ratio
Using typical published values, select 1/7 as the anisotropic ratio.

Horizontal Hydraulic Conductivity
The horizontal hydraulic conductivity of the core calculated from the vertical hydraulic conductivity and anisotropic ratio was about 2×10^{-5} cm/s. However, the embankment core could develop cracks because of drying. Therefore, we accounted for the possibility of cracking during our seepage analyses by altering the horizontal hydraulic conductivity of the embankment core.

The following equation was used to estimate the equivalent saturated hydraulic conductivity of the embankment core with transverse cracks:

$$k_h = (k_1 * z_1 + k_2 * z_2)/(z_1 + z_2) \qquad \text{(Equation 4)}$$

We considered the following to calculate an equivalent saturated hydraulic conductivity of the embankment core using Equation 4:

- The length of the embankment is approximately 1,690 ft and the horizontal hydraulic conductivity of intact fill was 2×10^{-5} cm/s.
- The width of the transverse cracks was 0.5 inch with an average occurrence about every 100 feet along the length of the dam based on *Embankment Dam Cracking, James Sherard, 1973.*
- The horizontal hydraulic conductivity of the transverse crack was considered to be equal to the horizontal hydraulic conductivity of the filter zone, 6×10^{-2} cm/s. The filter zone will intercept a seepage path through the embankment and therefore will control the hydraulic conductivity of a crack. Development of the filter zone hydraulic conductivity is presented in Section 2.4.
- Eighteen 0.5-inch cracks and 17 100-foot sections of intact embankment core were included in Equation 4.

A horizontal saturated hydraulic conductivity of 5×10^{-5} cm/s was calculated using Equation 4 and selected to use for the embankment core during analyses. Based on this value and the selected anisotropy, the vertical hydraulic conductivity of the cracked core is 7×10^{-6} cm/s.

Porosity
Using a dry unit weight of 110 pcf and Equation 3, a porosity of 0.35 was selected.

2.6 Pierre Shale

Pierre shale underlies the foundation soil along the embankment alignment. The Pierre shale properties are not anticipated to vary widely, and therefore data from beyond the limits of the embankment footprint were considered while developing the material properties.

Moist Unit Weight
Based on the average of the dry unit weights and moisture contents from the Phase II laboratory results, the average dry unit weight is 122.1 pcf and the average moisture content is 12.6. Using Equation 1, a moist unit weight of 138 pcf was selected.

Saturated Unit Weight
Using a dry unit weight of 122.1 pcf and Equation 2, a saturated unit weight of 139 pcf was selected.

Drained (Effective) Strength Parameters
Using triaxial data from the Phase II investigation, the drained friction angle for the fully softened strength is 40 degrees and the drained cohesion is zero. The fully softened strength is lower than the peak strength, so using the fully softened strength parameters is more conservative.

A triaxial laboratory test was also performed as part of the Phase I investigation, but the results were generally higher than the Phase II triaxial data. Therefore, the Phase II data was used to be conservative. Interpretation of triaxial data is presented in the Phase II Geotechnical Data Report (RJH, in progress).

Undrained Strength
Based on the triaxial data and UCS data from the Phase II investigation, the undrained strength is 4000 psf. Interpretation of triaxial data is presented in the Phase II Geotechnical Data Report (RJH, in progress).

Composite Strength
A composite strength envelope was used for Pierre shale. The bilinear normal is 4767 psf.

Anisotropic Ratio
Based on engineering judgement, use an anisotropic ratio of 1. The hydraulic properties of Pierre Shale are not anticipated to significantly affect the results because this formation is much less permeable than the overlying soil.

Horizontal Hydraulic Conductivity
Based on the packer test data from the Phase II geotechnical investigation, a horizontal hydraulic conductivity of 1×10^{-7} cm/s was selected

Vertical Hydraulic Conductivity
Using the horizontal hydraulic conductivity and the anisotropic ratio, the vertical hydraulic conductivity is calculated to be 1×10^{-7} cm/s.

Porosity
Using a dry unit weight of 122 pcf and Equation 3, a porosity of 0.28 was selected.

2.7 Barrier Wall

The barrier wall material properties were developed based on RJH's experience with typical soil-bentonite backfill material.

Moist Unit Weight
Based on our experience with soil-bentonite backfill, a moist unit weight of 120 pcf was selected.

Saturated Unit Weight
Based on our experience with soil-bentonite backfill, the saturated unit weight is approximately equal to the moist unit weight. A saturated unit weight of 120 pcf was selected.

Drained (Effective) Strength Parameters
Based on engineering judgement, we selected 8 degrees as the drained friction angle and 0 psf as the drained cohesion.

Undrained Strength
Based on engineering judgement, we selected an undrained cohesion of 50 psf and a total stress friction angle of 0 degrees.

Composite Strength
A composite strength envelope for the barrier wall was used. The bilinear normal is 356 psf.

Vertical Hydraulic Conductivity
Based on our experience with soil-bentonite backfill, we selected a vertical hydraulic conductivity of 1x10^{-7} cm/s.

Anisotropic Ratio
Based on our experience with soil-bentonite backfill, we selected an anisotropic ratio of 1.

Horizontal Hydraulic Conductivity
Based on our experience with soil-bentonite backfill, we selected a horizontal hydraulic conductivity of 1x10^{-7} cm/s.

Porosity
Based on our experience with soil-bentonite backfill, we used a dry density of 88 pcf and Equation 3 to calculate the porosity. We selected a porosity of 0.48.

3.0 References

Freeze, R.A. & Cherry, J. A. (1979). Groundwater. Prentice Hall, Englewood, Cliffs, New Jersey.

Hunt, R. E. (2005). *Geotechnical Engineering Investigation Handbook.* CRC Press.

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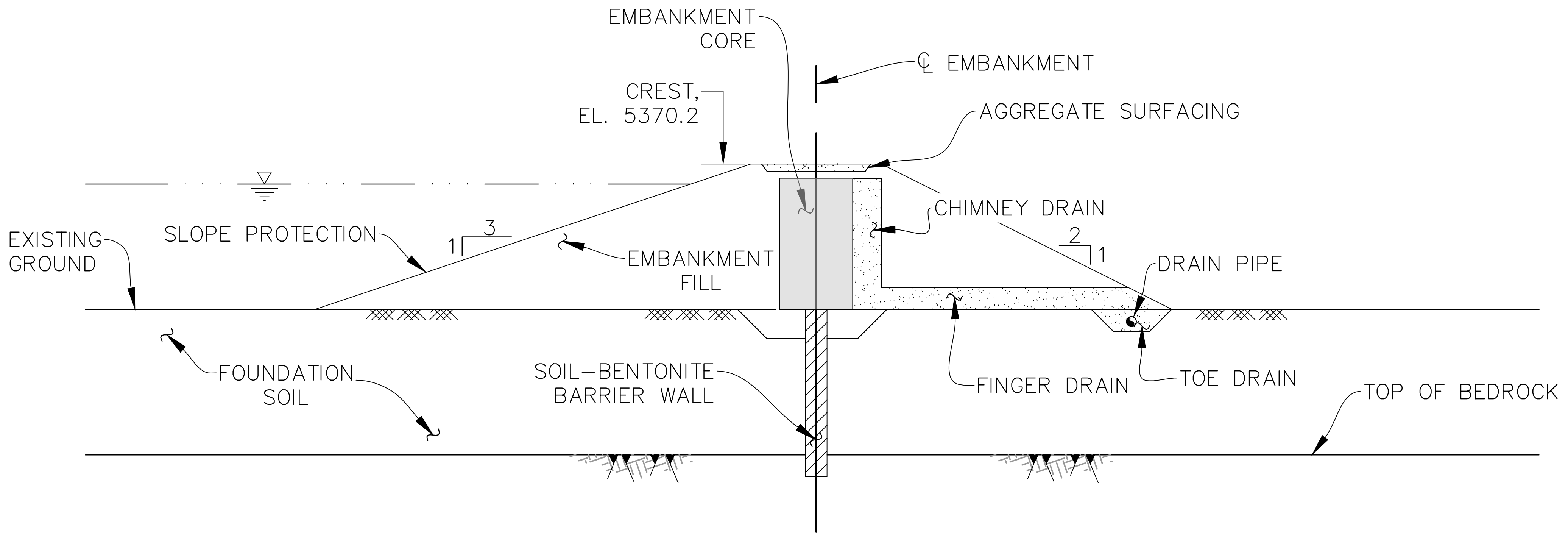
Shephard, James L. (1973). *Embankment Dam Cracking*. Publication of John Wiley and Sons, Incorporated.

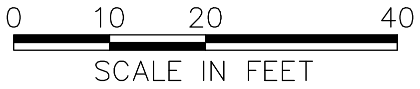
United States Army Corps of Engineers Geotechnical and Structural Laboratory (2008). ERCD/GSL TR-08-2: Mechanical and Physical Properties of ASTM C-33 Sand.

United States Bureau of Reclamation (2014). *Design Standard No. 13: Embankment Dams.* January.

4.0 Appendices

Attachment A: Embankment Cross Section and Geometry





\\RJHSHP19\PROJECTS\16134 – SOUTH BOULDER CREEK\CAD\FIGURES\OPTION 1 – EMB.DWG 7/1/2021 2:16 PM

NOT FOR CONSTRUCTION

NOTES:

1. EMBANKMENT GEOMETRY IS BASED ON CONCEPT LEVEL DESIGN.

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION
PROJECT

PROJECT NO. 16134

TYPICAL SECTION - EARTHEN EMBANKMENT

December 2020

Figure 1

APPENDIX C.2

EMBANKMENT SEEPAGE AND STABILITY ANALYSES

TECHNICAL MEMORANDUM

Project No. 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Adam Prochaska, P.E., Ph.D., P.G. - RJH Consultants, Inc.**

DATE: **July 7, 2021**

SUBJECT: **South Boulder Creek Regional Detention Project**
Stability and Seepage Analyses for Embankment Typical Maximum Section

---

1.0 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present analysis techniques, model results, and conclusions for seepage and stability analyses performed for the earthen embankment for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City).

Two-dimensional seepage and stability analyses were performed using Seep/W and Slope/W, respectively, which are part of the GeoStudio 2021 software package.

2.0 Evaluated Section

Analyses were performed for one generalized typical cross section of the proposed embankment. The evaluated cross section was conservatively developed at a location near the maximum embankment section and where Viele Channel is closest to the embankment. A plan and section figure of the evaluated embankment section are presented in Attachment A.

The natural ground surface elevation at the evaluated section is Elevation (El.) 5354.0 and the top of bedrock is at El. 5339.0 based on RJH's Phase II borings. Foundation soils at the evaluated section and beneath much of the proposed embankment consist of fill that was previously placed to reclaim the CU South campus after mining operations. Alluvial soils will comprise the embankment foundation near its right abutment. Our analyses considered a range of material properties for foundation soil that are representative of both the existing fill and alluvium.

The bottom of Viele Channel was modeled at El. 5343.0 based on Project topography and the bottom of Pond 3 was considered to be at bedrock elevation (El. 5339).

The modeled embankment cross section has a 17-foot-wide crest at El. 5370.2, and 4H:1V upstream and downstream slopes. The embankment is anticipated to include a central core, upstream and downstream shells, and a downstream filter/drain zone. The exact core and filter configurations have not yet been selected, so we considered a 10-foot-wide vertical core and a 4-foot-wide chimney filter with a toe drain. A relatively narrow central core was selected for evaluation because sufficient fine-grained borrow material might not exist onsite for construction of a broad core or homogenous embankment. We included a 3-foot-wide barrier wall beneath the embankment that connects to the embankment core and extends 5 ft into bedrock.

Grading for the detention excavation has not yet been finalized; for these analyses we considered a 20-foot offset between the upstream toe of the embankment and the top of the detention excavation, a 4H:1V slope to the bottom of the detention excavation, and a bottom of detention excavation at El. 5343.0.

The evaluated section extended 390 feet upstream and 560 feet downstream of the embankment centerline, which in our opinion is adequate for reducing edge-effects on model results near the embankment.

3.0 Material Properties

Material properties that were used for seepage and stability modeling are presented in Table 3.1. Development of these material properties is presented in the *Embankment Seepage and Stability Material Properties* (RJH, 2021) analyses memorandum.

TABLE 3.1
EMBANKMENT MATERIAL PROPERTIES FOR SEEPAGE AND STABILITY ANALYSES

	Foundation Soil(3)	Embankment Shell(4)	Embankment Filter	Embankment Core	Pierre Shale	Barrier Wall
Moist Unit Weight (pcf)	119	121	114	128	138	120
Saturated Unit Weight (pcf)	130	129	132	132	139	120
Drained Friction Angle, Φ (deg.)	25	28	36	28	40	8
Drained Cohesion (psf)	0	0	0	0	0	0
Total Stress Friction Angle (deg.)	17	17	NA(5)	17	0	0
Undrained Cohesion (psf)	31	31	NA	31	4000	50
Bilinear Normal(1) (psf)	180	130	NA	130	4767	356
Vertical Hydraulic Conductivity, K_v (cm/s)	2E-03	5E-03	3E-02	7E-06	1E-07	1E-07
Anisotropic Ratio (Kv/Kh)	0.1	0.2	0.5	0.14	1	1
Horizontal Hydraulic Conductivity, K_h (cm/s)	2E-02	3E-02	6E-02	5E-05	1E-07	1E-07
Porosity, η	0.36	0.38	0.35	0.35	0.28	0.48
GeoStudio Material(2)	Sand	Sand	Sand	Clay	Clay	Clay

Notes:

1. Bilinear normal is the normal stress where the drained and undrained strength envelopes intersect.
2. Volumetric water content functions and unsaturated hydraulic conductivity functions were estimated using these example materials that are built-in to the GeoStudio database.
3. Foundation soil could range from fine-grained fill to coarse-grained alluvium along the embankment alignment. The tabulated strength values are representative of fill and the tabulated hydraulic properties are representative of alluvium. A lower hydraulic conductivity equal to that of the embankment core was also considered for some seepage analyses to represent fill foundation soils.
4. Embankment shell fill could range from fine-grained to coarse-grained materials. The evaluated material properties considered a relatively low strength (representative of fine-grained fill) and a relatively high hydraulic conductivity (representative of coarse-grained fill).
5. NA means the property is not applicable for the material type.

4.0 Analysis Settings

4.1 Seepage Analysis Settings

Seep/W is a two-dimensional finite element seepage model. We discretized the modeled cross section using a square mesh with 1-foot elements. We performed analyses that evaluated steady state seepage conditions for various hydraulic loads. This approach is very conservative because the reservoir pool for a flood control dam will normally be empty and, water will only be detained briefly in the reservoir following large flood events, which will not be sufficient time for steady state conditions to develop.

Total head and water rate boundary conditions were used. Total head boundary conditions were used to simulate specific surface water and groundwater levels. Total head boundary conditions allow water to enter or exit the model as required to maintain the specified head at the location of the boundary condition. The water rate boundary condition with a seepage face review was generally applied to the ground surface on the down-gradient side of the model and to the toe drain pipe. The water rate boundary condition allows water to exit the model if the calculated total head is higher than the elevation of the boundary.

4.2 Slope Stability Analysis Settings

The entry-exit analysis setting was used in Slope/W to search for potential failure surfaces. The entry-exit setting searches for failure surfaces that enter and exit the ground surface in user-specified locations. Safety factors were calculated using Spencer's Method, which considers both the force and moment equilibrium.

Critical failure surfaces were optimized. This means that after a critical failure surface was calculated, Slope/W adjusted the shape of the failure surface to identify if a slightly different shape would result in a lower safety factor.

The minimum failure surface depth was defined to be 5 feet as needed to force deeper, global failures and prevent the identification of shallow surficial failures that would not impact dam performance or safety.

5.0 Loading Conditions

Loading conditions and required safety factors were selected based on the United States Army Corps of Engineers (USACE) *EM 1110-2-1902* in accordance with the Colorado Office of the State Engineer Rules and Regulations. Based on USACE criteria and our opinions about site specific loading conditions, RJH analyzed the following loading conditions for the South Boulder Creek embankment:

1. Steady state conditions with an empty reservoir (seepage of groundwater into an empty detention excavation)
2. Empty reservoir at the end of construction
3. Steady state conditions from a full reservoir (maximum water surface El. 5364)
4. Rapid drawdown from a full reservoir to the bottom of the detention excavation

The required minimum safety factors for each loading condition are presented in Table 5.1.

TABLE 5.1
REQUIRED MINIMUM SAFETY FACTORS

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

6.0 Analyses

6.1 Steady State - Empty Reservoir

We evaluated seepage and stability conditions that would exist during steady state seepage when the reservoir is empty. In our opinion, this loading condition most accurately represents the normal operating conditions for the embankment.

6.1.1 Seepage Analysis Results

We performed four seepage analyses for the empty reservoir conditions:

- Empty 1: With the hydraulic material properties shown in Table 3.1 applied to the foundation soil and typical groundwater conditions in Viele Channel at El. 5345. This groundwater elevation was selected based on typical water levels measured in RJH's monitoring wells and stilling wells near the evaluated cross section. The selected high-permeable hydraulic material properties are representative of alluvial soil.
- Empty 2: With the hydraulic material properties of the embankment core applied to the foundation soil and typical groundwater conditions in Viele Channel. These hydraulic material properties are representative of lower-permeable material that could exist within the existing fill soils. The groundwater conditions are the same as the Empty 1 analysis.
- Empty 3: Similar to Empty 1 except with water at the top of Viele Channel's banks (El. 5354). This scenario represents a localized flood event downstream of the embankment where Viele Channel is flowing full but the detention excavation is empty.
- Empty 4: Similar to Empty 2 except with water at the top of Viele Channel's banks (El. 5354).

Table 6.1 presents the seepage results for each analysis.

TABLE 6.1
EMPTY RESERVOIR SEEPAGE RESULTS

Analysis	Exit Gradient (into Detention Excavation)	Flow Rate(1) (gpm per foot)	Flow Rate(1) (gpm for a 1,100-foot-long embankment)
Empty 1: High-permeable foundation soils and typical Viele Channel	<0.1	$8.2x10^{-6}$	$9.1x10^{-3}$
Empty 2: Low-permeable foundation soils and typical Viele Channel	<0.1	$4.9x10^{-6}$	$5.4x10^{-3}$
Empty 3: High-permeable foundation soils and bank-full conditions in Viele Channel	<0.1	$2.2x10^{-3}$	2.5
Empty 4: Low-permeable foundation soils and bank-full conditions in Viele Channel	0.3	$1.5x10^{-3}$	1.6

Note:
1. 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.

For all analyses, the phreatic surface remains below the embankment in the foundation soil, and the barrier wall effectively reduces seepage into the detention excavation. The predicted seepage conditions for empty reservoir conditions are acceptable based on our judgement.

The exit gradient into the detention excavation for the Empty 4 analysis is higher than the other Empty analyses. The calculated safety factor for Empty 4 is 3.7 which is lower than the recommended safety factor of 4 provided in the United States Bureau of Reclamation (USBR) *Design Standards No. 13: Embankment Dams* (page C2 and C3). However, this safety factor is acceptable because of the compounding conservatism incorporated into the model including modeling steady-state conditions. Steady-state conditions are conservative and unlikely to occur. Also, the low-permeable foundation soil is a fine-grained material and exit gradients are generally less applicable in fine grained materials. Piping due to high exit gradients is not a likely failure mode and does not represent a dam safety concern.

Steady state seepage of groundwater into the detention excavation from beneath the embankment is predicted to be minor during typical hydrogeologic conditions (5 to 15 gallons per day during Empty 1 and Empty 2 analyses). In our opinion, steady state seepage conditions simulated by Empty 3 and Empty 4 analyses will likely not occur because high flows in Viele Channel are anticipated to be short in duration. MODFLOW analyses will be used to perform additional groundwater modeling, and these results will be provided in a separate report.

6.1.2 Stability Results

We evaluated embankment stability of this loading condition with low permeable foundation soil properties and downstream water at the banks of Viele Channel (Empty 4 seepage analysis) for conservatism because this seepage scenario produced the highest phreatic surface through the model. Drained strengths were used for each material to evaluate embankment stability during steady state. The safety factors for the upstream and downstream slopes were 2.2 and 2.3, respectively, which satisfy the minimum required safety factor of 1.5.

A summary of slope stability safety factors is presented in Section 7.

6.2 End of Construction

We evaluated the stability conditions that would exist at the end of construction using the phreatic surface from the Empty 4 seepage analysis for conservatism. Bilinear strengths were applied to low-permeable materials, which conservatively considers the lesser of either drained or undrained strengths. The safety factors for both the upstream and downstream slopes were 1.5 and 1.6, respectively, which satisfies the minimum required safety factor of 1.3. A summary of slope stability safety factors is presented in Section 7.

6.3 Steady State – Full Reservoir

We evaluated seepage and stability conditions that would exist during steady state seepage when the reservoir is at the maximum reservoir water elevation for a long period of time. This analysis technique is conservative because steady state seepage conditions are not anticipated to develop during short-term reservoir impoundments.

6.3.1 Seepage Analysis Results

We performed the following four steady state seepage analyses to evaluate potential ranges of foundation soil conditions and downstream hydraulic conditions:

- Full 1: With the hydraulic material properties shown in Table 3.1 applied to the foundation soil and an empty Viele Channel. The selected high-permeable hydraulic material properties are representative of alluvial foundation soil.
- Full 2: With the hydraulic material properties of the embankment core applied to the foundation soil and an empty Viele Channel. These hydraulic material properties are representative of lower-permeable material that could exist within the existing fill soils.
- Full 3: Similar to Full 1 except with bank-full conditions in Viele Channel (El. 5354), and
- Full 4: Similar to Full 2 except with bank-full conditions in Viele Channel (El. 5354).

We used a water surface elevation of El. 5364 for the full reservoir (which is about 0.2 feet above the spillway crest) and assigned the water surface elevation as total head boundary conditions along the ground surface of the detention excavation and the upstream slope of the embankment. We used a potential seepage face boundary condition to simulate an empty Viele Channel. For the groundwater conditions where Viele Channel is at bank-full water elevation, we assigned a total head boundary condition at El. 5354 to Viele Channel and all the ground surface downstream of Viele Channel.

Boundary conditions assigned to the toe drain pipe were a seepage face boundary condition drain for empty Viele Channel conditions and a total head boundary condition for analyses with bank-full Viele Channel conditions.

Table 6.2 presents the seepage results for each analysis.

TABLE 6.2
FULL RESERVOIR SEEPAGE RESULTS

Analysis	Exit Gradient	Factor of Safety for Heave in Foundation Soil (>4 required)	Flow Rate[(1)] (gpm per foot)	Flow Rate[(1)] (gpm for a 1,100 foot-long embankment)	Flow Rate through Toe Drain (gpm per foot)	Flow Rate through Toe Drain (gpm for a 1,100 foot-long embankment)
Full 1: High-permeable foundation soils and empty Viele Channel	0.1	11	0.05	54	0	0
Full 2: Low-permeable foundation soils and empty Viele Channel	0.6	1.8(2)	0.04	45	$3.7x10^{-2}$	41
Full 3: High-permeable foundation soils and bank-full conditions in Viele Channel	<0.1	11	0.04	45	$3.5x10^{-2}$	39
Full 4: Low-permeable foundation soils and bank-full conditions Viele Channel	<0.1	11(2)	0.04	45	$3.6x10^{-2}$	40

Note:

1. 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.
2. The value is provided for information purposed only. Factors of safety against heave are generally not applicable for fine grained soils similar to the modeled low-permeable foundation soils.

In our opinion, the predicted seepage conditions for all analyses are acceptable based on the recommended factors of safety provided in the USBR *Design Standards No. 13: Embankment Dams* (page C2 and C3) and judgement. The safety factor of heave for the Full 2 analysis is slightly lower than required. However, the safety factor of 1.8 is acceptable because: (1) the model conservatively considered steady-state conditions, which are unlikely to occur, (2) the model conservatively considered an empty Viele channel and during a flood event, Viele channel will likely be full of flood water, (3) exit gradients are generally less applicable for fine-grained materials like the low-permeable foundation soil included in the Full 2 and Full 4 analyses, (4) the flow rate through the low-permeable foundation soils is anticipated to be very minor.

Based on the results for the Full 4 analysis, flood conditions in Viele Channel could impact the ability of the embankment toe drainpipe to drain; however, this is anticipated to be a short-term condition. The model results show that the phreatic surface in the embankment is not predicted to rise appreciably above the finger drain even if steady state conditions are considered, which in our opinion is acceptable.

6.3.2 Stability Results

We evaluated embankment stability under steady state seepage conditions using the seepage results from analysis Full 4 because it was conservative (i.e., produced the highest phreatic surface of the four evaluated scenarios). Drained strengths were used for each material to evaluate embankment stability. The safety factors for the upstream and downstream slopes are each 2.0, which satisfy the minimum required safety factor of 1.4.

A summary of slope stability safety factors is presented in Section 7.

6.4 Rapid Drawdown

We evaluated stability of the upstream slope during a rapid drawdown event. We evaluated drawdown from the maximum water surface elevation (El. 5364) to the outlet pipe, which is at the bottom of the detention excavation (El. 5343).

Analyses were performed using the improved method (i.e., three-stage method) as described in *USACE Engineer Manual 1110-2-1902, Appendix G*. This method uses the lesser of the drained and undrained strength to calculate safety factors based on the stress to which the soil is consolidated prior to drawdown.

The calculated safety factor for the upstream slope was 1.1, which satisfies the minimum required safety factor of 1.1. In our opinion, this analysis technique is conservative because steady state seepage conditions are not anticipated to develop through the embankment and foundation during short-term reservoir impoundments.

7.0 Summary

A summary of the slope stability safety factors for the evaluated loading conditions is presented in Table 7.1.

TABLE 7.1
SLOPE STABILITY FACTOR OF SAFETY FOR EMBANKMENT

Loading Condition	Slope Side	Safety Factor	Required Minimum Safety Factor	Acceptable
End of Construction	Upstream	1.5	1.3	Yes
End of Construction	Downstream	1.6	1.3	Yes
Steady State - Empty	Upstream	2.2	1.5	Yes
Steady State -Empty	Downstream	2.3	1.5	Yes
Steady State - Full	Upstream	2.0	1.4	Yes
Steady State - Full	Downstream	2.0	1.4	Yes
Rapid Drawdown	Upstream	1.1	1.1	Yes

8.0 Conclusion

Based on the analyses performed; SEO, USACE and USBR requirements; and engineering judgement, we conclude the following about seepage and stability conditions for the proposed embankment:

- The reservoir will be normally empty. The barrier wall is predicted to effectively limit groundwater seepage into the reservoir during both normal downstream hydraulic conditions and flood conditions in Viele Channel. Seepage rates into the detention excavation from beneath the embankment are predicted to be much less than 1 gpm during normal groundwater conditions.
- Acceptable seepage conditions would exist if steady state seepage occurred at the maximum water surface elevation. The core, barrier wall, and toe drainpipe effectively manage seepage and generally keep the phreatic surface below the natural ground surface downstream of the dam (i.e., below the downstream shell). Transient loads should be considered in future design phases.
- In our opinion, seepage can be managed by a chimney filter and a toe drain that are connected by intermittent finger drains.
- Seepage and stability conditions are predicted to be acceptable for both types of foundation soil (lower-permeable fill versus higher-permeable alluvium). Higher exit gradients might exist if low-permeable fill is considered for the foundation soil but are acceptable based on the conservatism incorporated into the model.
- 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 drainpipe to drain.
- Acceptable stability conditions are expected to exist for all the evaluated loading conditions.
- The analyzed section of the embankment is approximately 1,100 feet in length, and the calculated seepage rate for full-reservoir conditions ranges from 45 to 54 gpm. The total predicted flow that is expected to enter the toe drain during steady state seepage from a full reservoir range from 0 to 40 gpm.

9.0 References

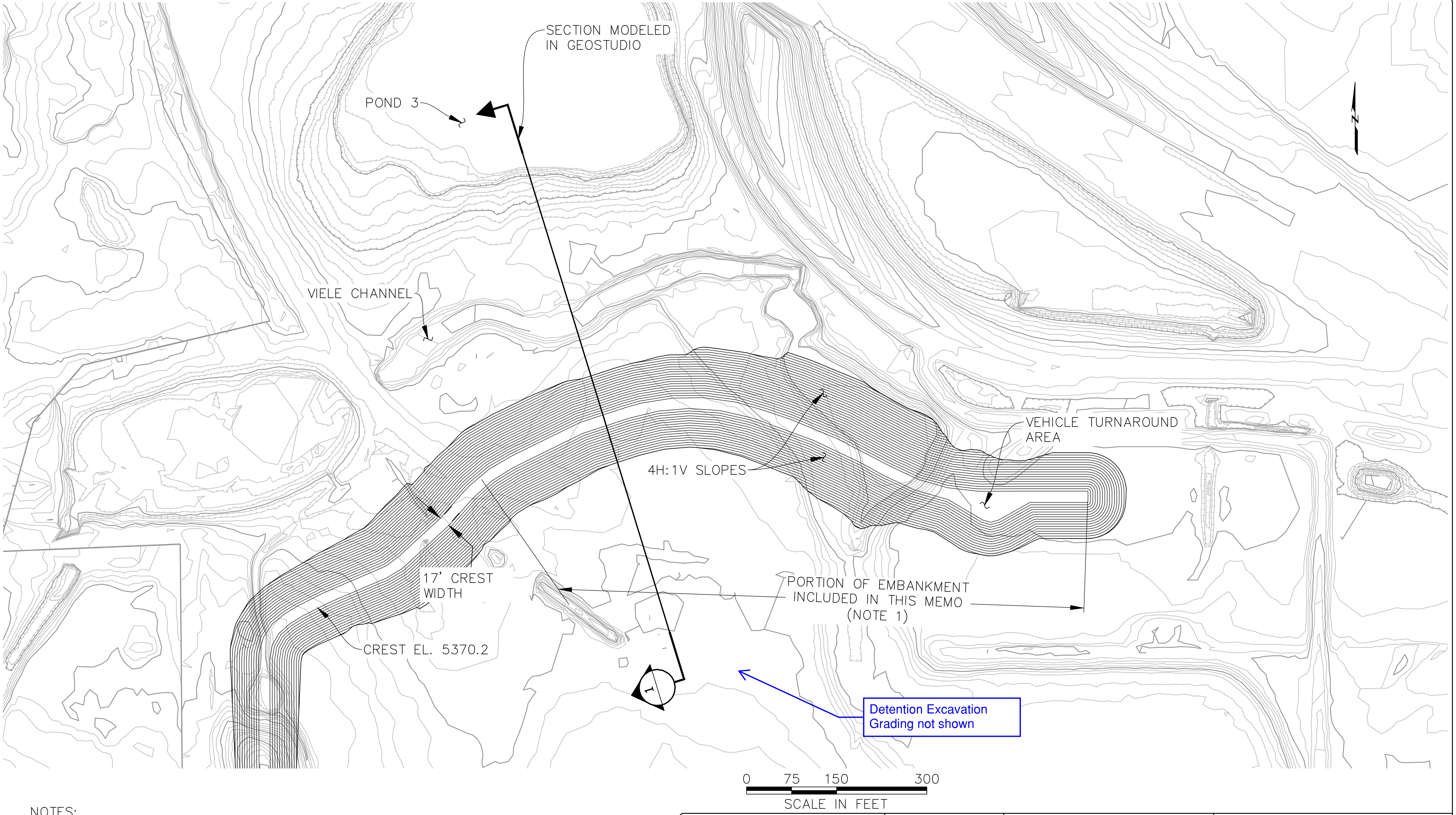
RJH Consultants, Inc. (2021). *Embankment Seepage and Stability Material Properties.*

United States Army Corps of Engineers. (2003). *Engineer Manual 1110-2-1902*.

United States Bureau of Reclamation. (2014). *Design Standards No. 13 Embankment Dams, Chapter 8: Seepage.*

10.0 Attachments

Attachment A: Figures



Detention Excavation
Grading not shown

NOTES:

1. THE WESTERN PORTION OF THE EMBANKMENT WILL BE INCLUDED IN FUTURE ANALYSES.

RJH
CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION
PROJECT

PLAN OF EMBANKMENT

PROJECT NO. 16134

July 2021

Figure 1

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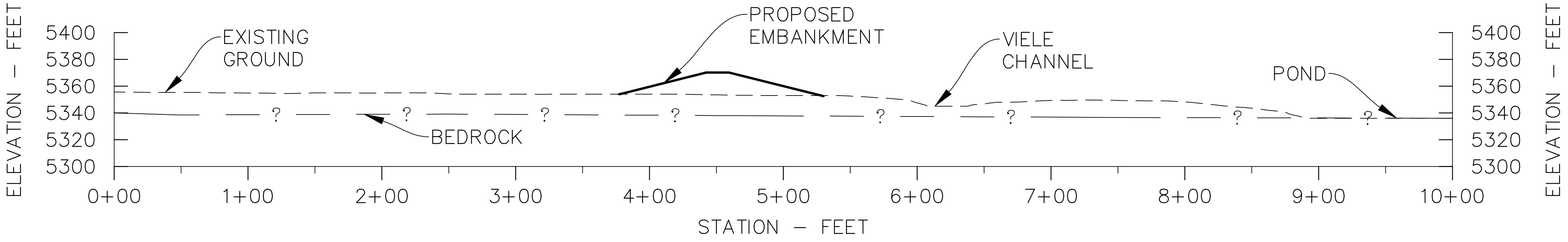

SECTION 1

0 50 100 200

SCALE IN FEET

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION
PROJECT

ANALYZED SECTION

PROJECT NO. 16134

July 2021

Figure 2

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APPENDIX D

SPILLWAY ANALYSES

APPENDIX D.1

SPILLWAY GEOSTRUCTURAL EVALUATION

TECHNICAL MEMORANDUM

Project No. 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Adam Prochaska, P.E., Ph.D., P.G. - RJH Consultants, Inc.**

DATE: **August 10, 2021**

SUBJECT: **South Boulder Creek Regional Detention Project**
Spillway Geo-Structural Analyses

1.0 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present the methodology and results of the spillway geo-structural analyses performed for the South Boulder Creek (SBC) Regional Detention Project (Project).

2.0 Background

The spillway will consist of a vertical, reinforced concrete wall that extends along the U.S. Highway 36 (US36) corridor. The spillway will be located to the south of the Colorado Department of Transportation (CDOT) right-of-way (ROW) on property owned by the City of Boulder Open Space and Mountain Parks (OSMP). The spillway will be approximately 2,700-feet long and will connect to the earthfill embankment at the west (left) end and to a US36 embankment at the east (right) end. The spillway crest elevation will be set to provide 1 foot of freeboard above the expected 100-year event water surface elevation. The height of the wall above existing ground will vary from about 5.5 to 10.5 feet.

Foundation conditions along the spillway consist of coarse-grained alluvium overlying Pierre Shale bedrock. Bedrock is expected to be about 19 feet below the ground surface near the left end of the spillway and 8 feet below the ground surface near the right end of the spillway. The spillway foundation evaluation for preliminary design consists 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 and to provide a seepage barrier to restrict flows through the coarse-grained alluvium during times of flood detention.

3.0 Regulatory Criteria

The dam will likely be classified as an extreme hydrologic hazard dam. The hydraulic loads evaluated during these analyses are consistent with guidelines presented in the SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO Rules) (SEO, 2020). The SEO Rules require that the spillway for an extreme hydrologic hazard dam be designed for the Probable Maximum Flood Event (PMF).

4.0 Geo-structural Analyses Inputs

4.1 Analysis Approach

RJH performed two-dimensional analyses using the DeepEX software program developed by Deep Excavation, LLC (2021). Nonlinear elastoplastic (i.e. Winkler springs) analyses were used with a 0.5-foot mesh density. 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).

American Concrete Institute (ACI) 318-19 code was used to calculate the design strength of the wall, which included strength reduction factors of 0.75 for shear and 0.9 for moment. The calculated design strengths were reduced by factors of 1.6 for 100-year hydraulic loads or 1.3 for PMF loads in accordance with United States Army Corps of Engineers (USACE) Engineering Manual EM 1110-2-2104 to obtain the allowable loads. The allowable loads were compared against unfactored hydraulic loads (load factor of 1) to evaluate wall capacity.

4.2 Evaluated Cross Sections

Analyses were performed for two representative cross sections at the locations shown on Figure 4.1. The external geometry and hydraulic loads modeled at each location are summarized in Table 4.1. External geometries were selected based on data from RJH's subsurface investigations (RJH, 2019; *In Progress*), and hydraulic loads were defined based on HEC-RAS modeling performed by RJH. Analyses were performed for secant pile foundations that extended varying depths below the top of bedrock as presented in Section 5.

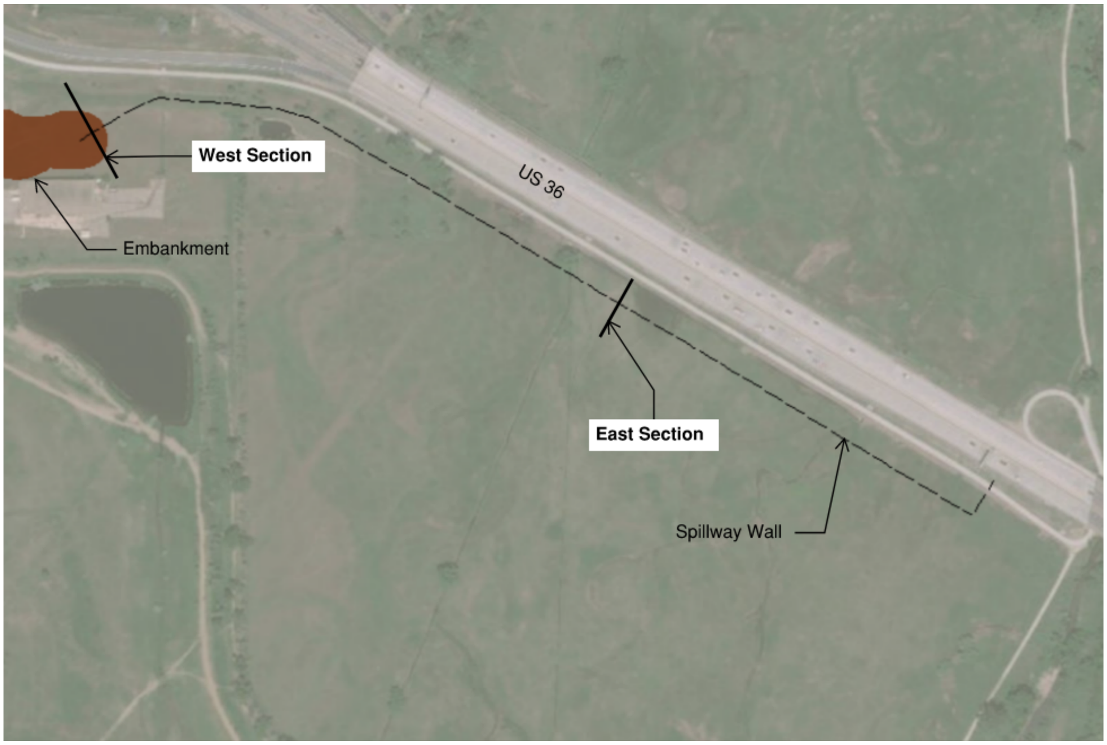

Figure 4.1 – Plan of Cross Section Locations

TABLE 4.1
CROSS SECTION EXTERNAL GEOMETRY AND HYDRAULIC LOADS

Feature	West Cross Section	East Cross Section
Ground Surface Elevation	5354.0	5358.0
Top of Bedrock Elevation	5335.4	5350.0
Top of spillway wall Elevation	5363.8	5363.8
100-year Load Water Surface Elevation	5363.8	5363.8
PMF Load Water Surface Elevation	5369.6	5368.9
Downstream Groundwater Elevation for no tailwater	5349.0	5357.0
Downstream water Elevation for PMF tailwater	5361.5	5367.3

4.3 Evaluated Spillway Properties

The analyzed wall consisted of an above-ground reinforced concrete wall and a below-ground secant pile wall. The modeled properties of these structures are summarized in Table 4.2. The spillway apron was not considered in the analyses, which is conservative. A concrete cap that is anticipated to connect the wall to the secant piles was also not modeled during this stage of analyses, and will be evaluated during subsequent design phases.

TABLE 4.2
SPILLWAY STRUCTURAL FEATURES

Feature	Above-ground concrete wall	Below-ground secant pile wall
Thickness	1.0 foot	4.0 feet diameter with 7.0 feet center-to-center spacing
Concrete compressive strength, f'_c (psi)	4,500	5,000
Rebar Grade	60	60
Minimum concrete cover for reinforcement (inches)	3	6
Vertical reinforcement	#7 rebar every 12 inches, each face	Eleven #9 bars(1)
Horizontal reinforcement	#7 rebar every 12 inches, each face	#5 rebar every 12 inches(1)

Notes:

1. Reinforcement was modeled in every other (alternating) secant pile.

4.4 Geotechnical Properties

Geotechnical material properties were selected for analyses based on information collected during RJH's subsurface investigations, typical published values for similar materials, and judgement. The properties used are summarized in Table 4.3.

TABLE 4.3
GEOTECHNICAL MATERIAL PROPERTIES

	Alluvium	**Pierre Shale**
Material Type / Behavior	Sand	Rock
Moist Unit Weight (pcf)	119	138
Saturated Unit Weight (pcf)	130	139
Friction Angle, ϕ' (deg.)	36	40
Drained Cohesion (psf)	5(1)	5(1)
Lateral Subgrade Modulus (K)(2) (kcf)	Varied from 200 to 400	Varied from 200 to 400

Notes:

1. A cohesion value of 5 psf was used to facilitate model convergence based on recommendations from DeepEX technical support.
2. For each analysis, the same lateral subgrade modulus value was applied to both alluvium and Pierre Shale because typical published ranges showed that significant overlap of this parameter exist throughout a wide range of geo-materials.

5.0 Analysis Results

5.1 Wall Deflections

RJH evaluated the horizontal deflections that are predicted to occur at the top of the spillway wall for various combinations of hydraulic load, lateral subgrade modulus, and embedment of the secant piles below the top of bedrock. Predicted deflections under 100-year flood hydraulic loads are presented on Figure 5.1 for the west section and Figure 5.2 for the east section. Predicted deflections under PMF hydraulic loads and PMF tailwater are presented on Figure 5.3 for the west section and on Figure 5.4 for the east section. Based on the model results shown on Figures 5.1 through 5.4, we selected an 8-foot embedment depth into bedrock for the secant pile wall.

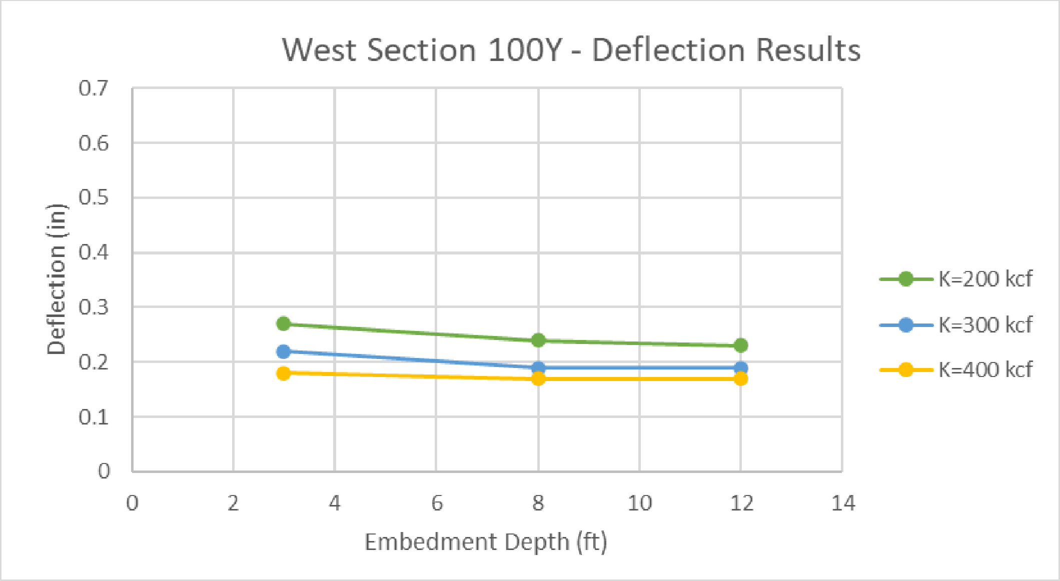

Figure 5.1 West Section Deflection Results for 100-Year Flood Loads

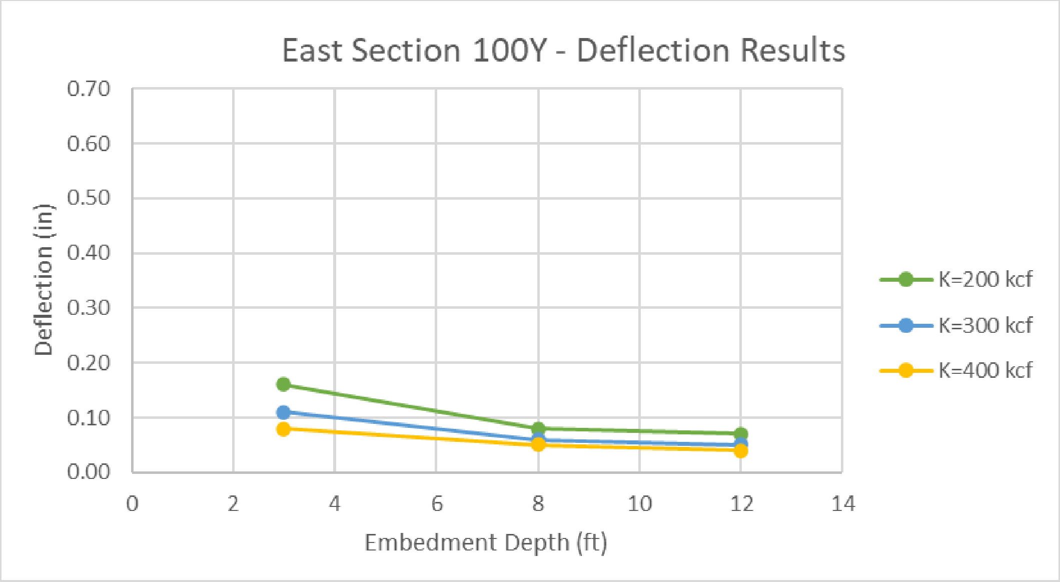

Figure 5.2 East Section Deflection Results for 100-Year Flood Loads

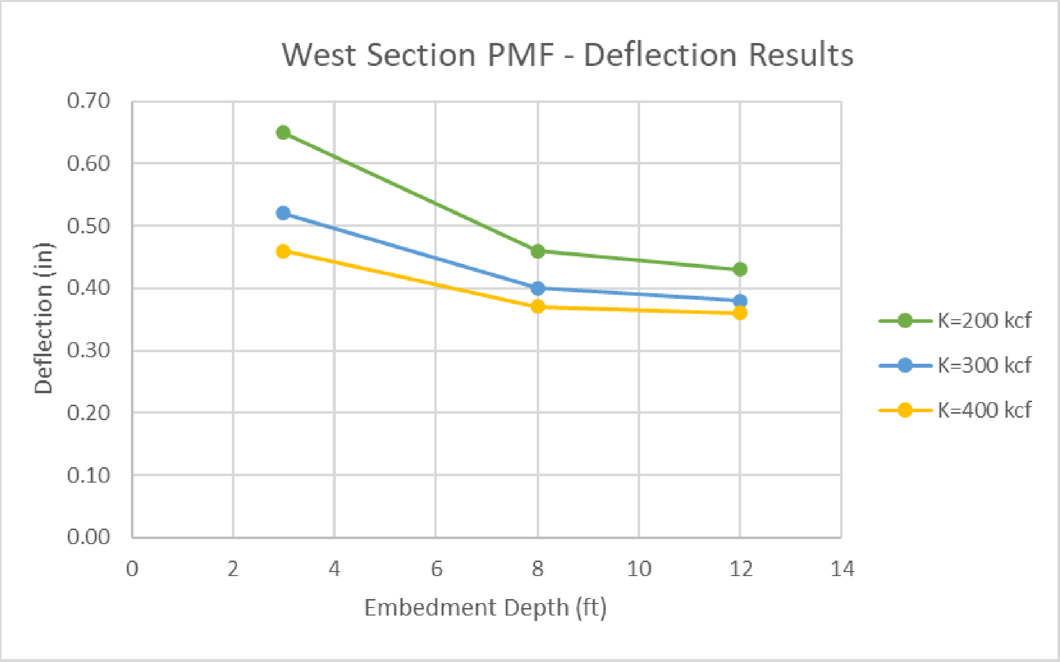

Figure 5.3 - West Section Deflection Results for PMF Loads

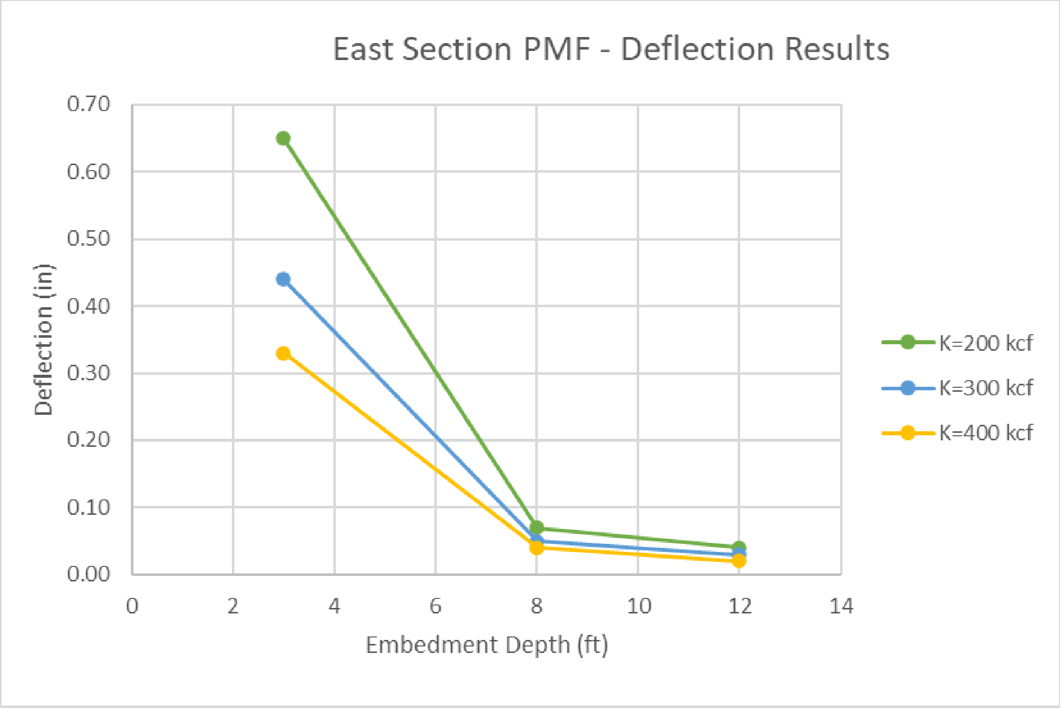

Figure 5.4 - East Section Deflection Results for PMF Loads

In our opinion an embedment depth of 8 feet below the top of bedrock is appropriate because:

1. An embedment depth of 8 feet below the top of bedrock is predicted to limit horizontal deflection at the top of the spillway wall to less than 0.5 inch at the west section and less than 0.1 inch at the east section during a PMF event.

2. An embedment depth of 8 feet below the top of bedrock is predicted to limit the horizontal deflection at the top of the spillway to less than 0.3 inch at the west section and less than 0.1 at the east section for the 100-year hydraulic load with no tailwater.
3. Based on the results on Figure 5.1 through Figure 5.4, extending the secant piles deeper than about 8 feet into bedrock is not predicted to significantly reduce wall deflections.
4. Extending the secant piles 8 feet below the top of bedrock is predicted to provide an adequate foundation seepage barrier.

5.2 Wall Structural Capacity

RJH evaluated the structural capacity of the wall configurations described in Section 4. Maximum applied shear and moments are summarized in Table 5.1 and model outputs are presented on Figures 5.3 through 5.6.

TABLE 5.1
STRUCTURAL CAPACITY FOR SPILLWAY WALL WITH 8-FOOT EMBEDMENT INTO BEDROCK AND LATERAL SUBGRADE MODULUS = 200 kcf1).

		Secant Pile Foundation		Above-Ground Concrete Wall			
Cross Section	Hydraulic Load	Maximum Shear (k/ft)	Maximum Moment (k-ft/ft)	Maximum Shear (k/ft)	Maximum Moment (k-ft/ft)	STR Shear	STR Moment
West	100-year flood with no tailwater	3.5	26.6	2.7	9.8	0.72	0.56
West	PMF flood with PMF tailwater	5.2	54.7	4.5	22.8	0.95	1.1(3)
East	100-year flood with no tailwater	1.2	5.9	0.9	2.0	0.25	0.12
East	PMF flood with PMF tailwater	1.0	5.8	0.5	1.7	0.11	0.08

Note:
1. Wall shear and moment were not highly sensitive to depth of bedrock embedment and lateral subgrade modulus value.
2. STR represents the ratio of unfactored applied load to allowable load. Values less than 1.0 mean structural capacity is adequate.
3. The inadequate moment capacity for this condition is limited to the bottom two feet of the spillway wall. The wall could be locally refined in future phases of design to provide adequate capacity.

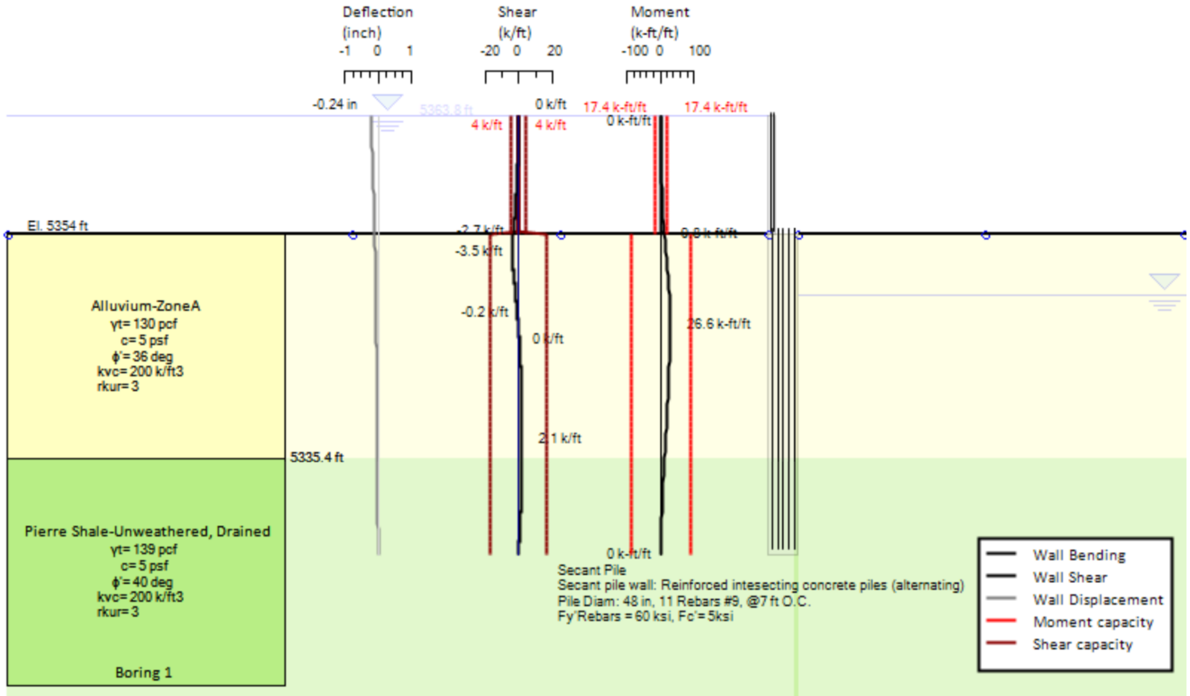

Figure 5.3 – West Section 100-Year Flood with No Tailwater

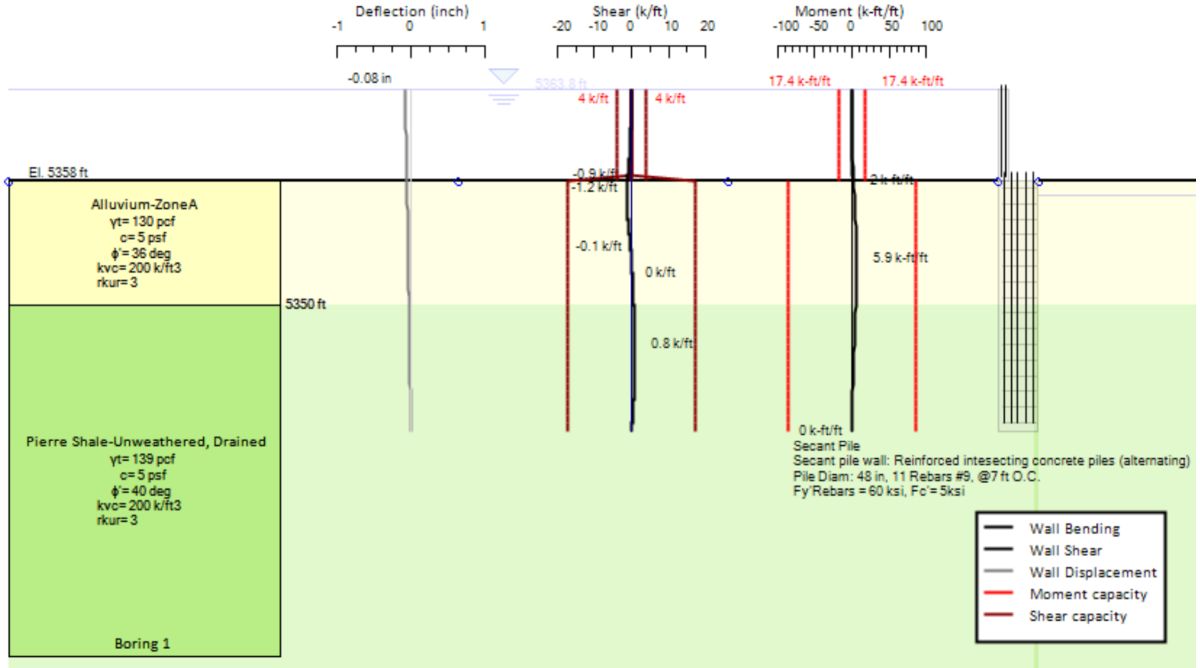

Figure 5.4 – East Section 100-Year Flood with No Tailwater

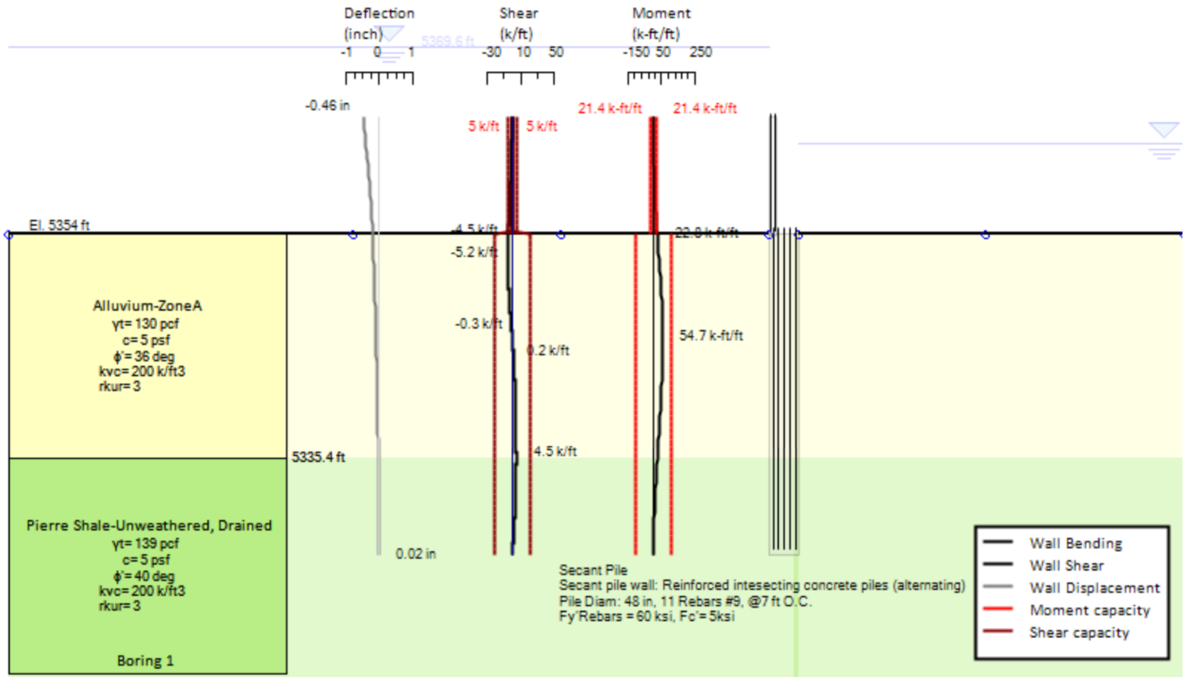

Figure 5.5 – West Section PMF Flood with PMF Tailwater

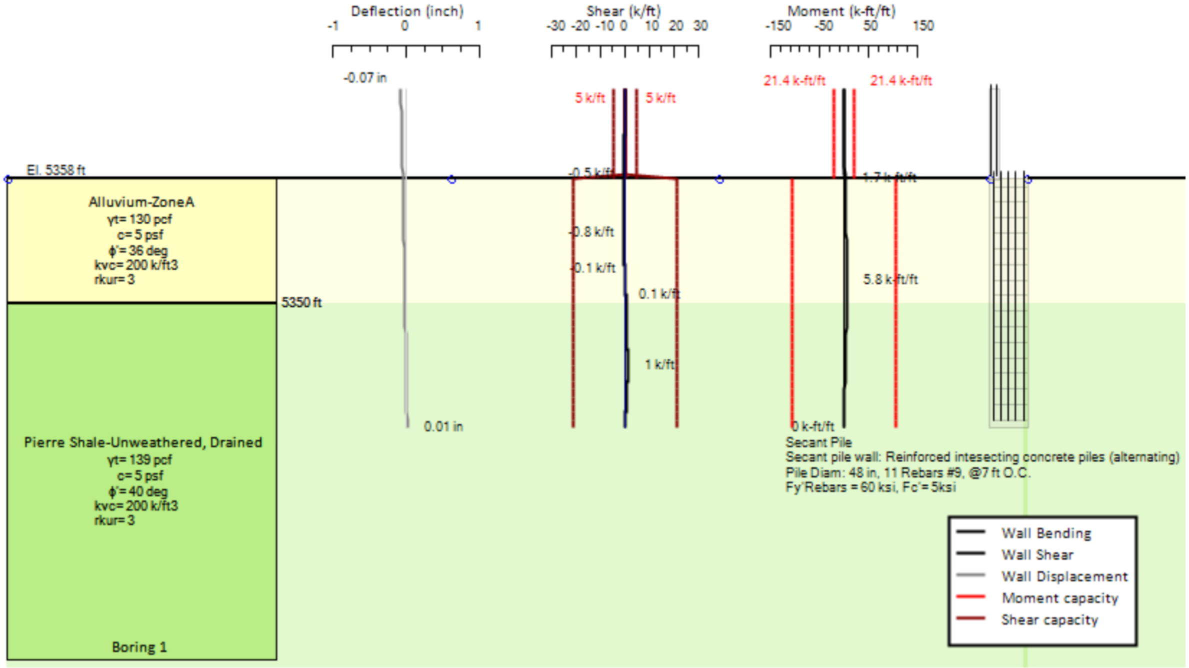

Figure 5.6 – East Section PMF Flood with PMF Tailwater

Based on the results summarized in Table 5.1, the evaluated wall configuration provides adequate capacity for each of the evaluated scenarios except for PMF loading at the west section. However, the inadequate moment capacity at the west section for the PMF load is limited to the bottom 2 feet of the spillway wall. We anticipate that the wall could be locally refined in future phases of design to provide adequate capacity in this area by either increasing reinforcement, thickening the wall, or through the pile cap.

6.0 Conclusions and Recommendations

We conclude the following based on the analyses presented in this memorandum:

- The secant piles should extend about 8 feet below the top of 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. Highly weathered or highly fractured rock encountered at the top of bedrock should be neglected when measuring the secant pile embedment during construction because the models did not account for highly weathered or highly fractured bedrock. Based on borings performed along the spillway wall, highly weathered bedrock might extend up to about 2 feet below the top of bedrock and highly fractured zones were generally not encountered in the bedrock.
- The structural information presented in Table 4.2 is a constructable configuration that appears to generally provide acceptable capacity throughout much of the spillway alignment, and in our opinion is a reasonable approximation to use for preliminary design.
- The following additional structural analyses should be performed in future phases of design:
    - Structural evaluations are required for the pile cap that will connect the spillway wall to the secant pile foundation.
    - Local structural enhancements are required to provide adequate capacity near the base of the wall in the western portion of the spillway.
    - Opportunities to develop a more efficient design could be achieved by varying the wall section along the length of the spillway alignment.

7.0 References

American Concrete Institute (ACI) (2019). Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary.

Deep Excavation, LLC (Deep Excavation) (2021). DeepEX (Version 21.0.8.0) [Computer software].

RJH Consultants, Inc. (RJH) (2019). Phase I Geotechnical Report, South Boulder Creek Regional Detention Project. August.

RJH Consultants, Inc. (RJH) (*In Progress*). Phase II Geotechnical Report, South Boulder Creek Regional Detention Project.

United States Army Corps of Engineers (USACE) (2016). Engineer Manual EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures. November 30.

APPENDIX D.2

SPILLWAY ENERGY DISSIPATION EVALUATION

RJH CONSULTANTS, INC.

TECHNICAL MEMORANDUM

Project 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Eric Hahn, P.E. - RJH Consultants, Inc.**

DATE: **August 10, 2021**

RE: **South Boulder Creek Regional Detention Project**
Spillway Energy Dissipation Evaluation

1.0 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present the methodology and results of the spillway energy dissipation evaluation performed for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City).

2.0 Background

The spillway will consist of a vertical, reinforced concrete wall that extends along the U.S. Highway 36 (US36) corridor. The spillway will be located to the south of the Colorado Department of Transportation (CDOT) right-of-way (ROW) on property owned by the City of Boulder Open Space and Mountain Parks (OSMP). The spillway will be approximately 2,700-feet long and will connect to the earthfill embankment at the west (left) end and to the US36 embankment at the east (right) end. The spillway crest elevation will be set to provide 1 foot of freeboard above the expected 100-year event water surface elevation. The height of the wall above existing ground will vary from 5.5 to 10.5 feet.

The spillway will discharge to the area between the spillway and the US36 roadway embankment. This area consists of both OSMP property and the CDOT ROW and includes a regional multi-use trail. A hydraulic evaluation is required to select and size an energy dissipation facility for the spillway. The energy dissipation facility will consist of a reinforced concrete spillway apron immediately downstream of the spillway wall.

3.0 Regulatory Criteria

The dam will likely be classified as an extreme hydrologic hazard dam; and for preliminary design, the City has requested that designs be developed for this classification. The hydraulic evaluations presented in this memorandum have been prepared consistent with guidelines presented in the SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO Rules) (SEO, 2020). The SEO Rules require that the spillway for an extreme hydrologic hazard dam be sized to convey the Probable Maximum Flood Event (PMF).

4.0 Hydraulic Analysis

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

- The existing ground generally slopes downward 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 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.

We used a two-dimensional, unsteady hydraulic model to model flow conditions at the spillway wall and the downstream channel.

4.1 Hydraulic Model

RJH developed a two-dimensional hydraulic model using HEC-RAS 5.0.7. Key model components are described as follows:

- A terrain model was prepared using a U.S. Geological Survey (USGS) 1-meter Digital Elevation Model (DEM), supplemented with data from the 2017 topographic survey and the Project features (i.e., embankment, detention excavation, spillway, etc.).
- A hydraulic mesh was delineated extending approximately 2.3 and 1.2 miles upstream and downstream of US36, respectively. The mesh was extended laterally to cover the entire flooded area during the PMF. A 15-foot cell size was used in the vicinity of the Project components. The cell size was increased to 100 feet at the upstream and downstream ends of the model to facilitate reasonable model run times. Break-lines with a maximum cell size of 10 feet were introduced into the model to adequately represent flow paths in the vicinity of roadways and channels.
- A Manning's n raster layer was prepared using data from the USGS National Land Cover Database (NLCD, 2016).
- The following culvert structures were modeled as connections inside the hydraulic mesh:
    - Four culverts along the Viele channel
    - Project outlet works consisting of dual 60-inch diameter pipes
    - Wildlife crossing through US36 consisting of dual 4-foot by 10-foot box culverts
- The Project spillway was modeled as a connection structure 14 inches wide. The discharge coefficient was selected as 3.35 using the Rehbock revised formula for sharp-crested weirs. In our opinion, this should provide a conservative evaluation for the spillway apron because it will result in increased flow over the spillway for a given water surface elevation.
- Normal depth was used for the boundary condition at the downstream end of the model. An inflow hydrograph was used for the boundary condition at the upstream

end of the model. The inflow hydrograph for the PMF was obtained from the Hydrology Report (RJH, 2020) and had a peak flow rate of 76,730 cfs.

- An unsteady model simulation was performed using a computation interval of 1 second and a 10 seconds mapping output interval.

4.2 Cross-Section Analysis

Three cross-sections were selected for the hydraulic evaluation of the spillway apron (from west to east). Cross-section locations are shown on Figure 4.1.

- Cross section at STA 5+82: The spillway wall is approximately 10.2 feet high. The maximum tailwater does not fully submerge the spillway wall because the glare guard is approximately 5.2 feet lower than the spillway wall. This is the location with the lowest tailwater depths.
- Cross-section at STA. 10+90: The spillway wall is approximately 10.2 feet high at this location. The spillway wall elevation is slightly above the top of the US36 glare guard, and at this location, the wall becomes fully submerged during high flow conditions.
- Cross-section at STA. 17+80: The spillway wall is approximately 6.9 feet high at this location and becomes fully submerged during the passage of the PMF. The US36 glare guard is approximately 0.8 feet higher than the spillway wall. This is the location where the spillway wall initially begins to overtop. This is also the location with the highest tailwater depths.

Results at cross-sections were extracted from the HEC-RAS model for different simulation times corresponding to increasing tailwater depths.

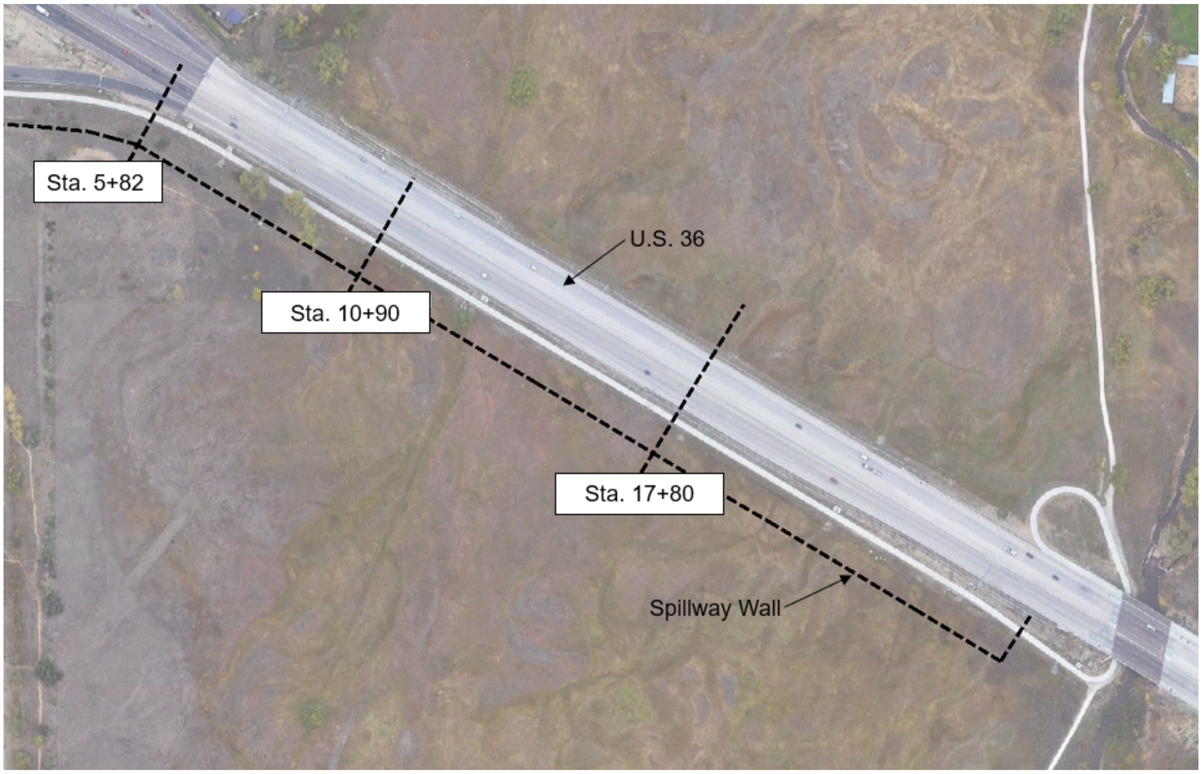

Figure 4.1 – Plan of Cross Section Locations

5.0 Theoretical Nappe Trajectory

RJH evaluated the theoretical nappe point of impact on the spillway apron assuming no tailwater and using the corresponding equations for nappe trajectory for sharp-crested weirs (Chow, 1959). A figure of the theoretical nappe trajectory is shown on Figure 5.1.

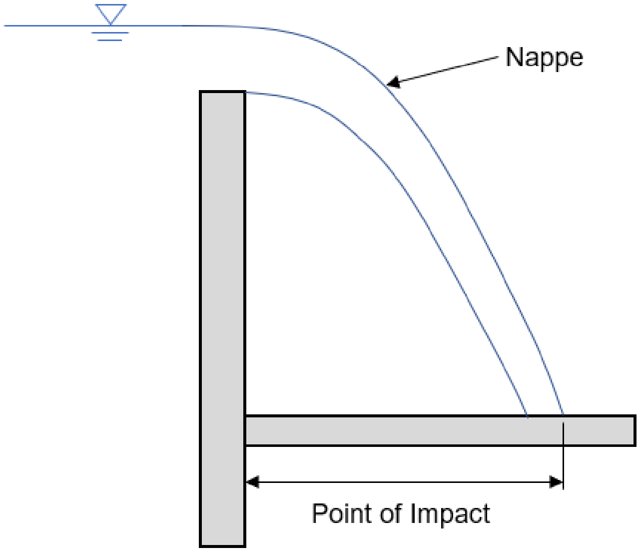

Figure 5.1 – Nappe Trajectory Schematic

Point of impact was calculated using the spillway wall height and the velocity and total flow head at the spillway crest. Distance from the downstream spillway toe to the point of impact is presented in Table 5. These points of impact would never actually develop in the field because tailwater impacts would submerge the nappe. The points of impact identified using this method were developed for comparison purposes to empirical methods.

TABLE 5.1
NAPPE TRAJECTORY - POINT OF IMPACT WITH NO TAILWATER

Station	Distance (ft)
5+82	13.2
10+90	10.3
17+80	5.6

6.0 Empirical Evaluation

RJH performed a review of technical papers related to drop-spillway energy dissipation. Most references were developed for shorter drop spillways, and design recommendations did not account appropriately for energy dissipation from high tailwater values.

We identified a technical report by the U.S. Bureau of Reclamation, Technical Report (TR) REC-ERC-74-9 *Hydraulic Model Studies of Plunge Basins for Jet Flow* (TR 74-9) (USBR,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. We are unsure how the nappe from an overflow weir would 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 preliminary design of the spillway apron and have endeavored a conservative application of this approach.

TR 74-9 includes empirical charts based on hydraulic laboratory modeling that can be used to predict theoretical scour hole depth, length, and width. The model study (USBR, 1974) was intended for outlet works plunge basin design. This required making some assumptions for its application to the spillway analysis:

- Jet Energy Head (H_v) was evaluated as the total head over the spillway crest, adding velocity head upstream the spillway (h_v) to the water depth over the crest (h_d).
- The width for square a gate was evaluated as the water depth over the spillway crest (h_d) for spillway unit length.
- The downstream distance to the jet point of impact on the tailwater was calculated using nappe trajectory equations described in the previous section.
- The impinging nappe extends across the tailwater until it reaches the terrain surface at a distance (X_L) from the spillway wall, following a linear trajectory with an angle of impingement that is evaluated in terms of the jet drop (Y) and energy head.

A schematic of the spillway dimensions used for the empirical evaluation is presented on Figure 6.1.

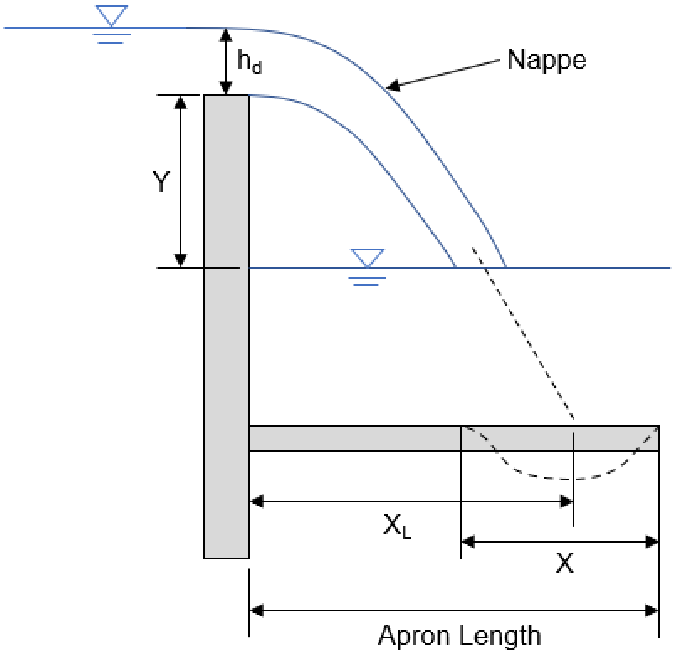

Figure 6.1 – Empirical Evaluation Schematic

Empirical charts from the TR 74-9 were used to estimate the length (X) for a scour hole excavated by the spillway overflow nappe. The spillway concrete apron length was then selected to generally match the predicted length of the scour hole.

Apron lengths were evaluated for incremental tailwater depth until the energy dissipation through tailwater showed negligible scour at the plunge basin. Apron length for incremental tailwater depths are presented in Table 6.1. The empirical evaluation resulted in longer spillway apron lengths than the theoretical nappe trajectory evaluation.

TABLE 6.1
CONCRETE APRON LENGTHS

	Cross section: 17+80 Spillway Wall Height: 6.9 ft		**Cross section: 10+90 Spillway Wall Height: 10.2 ft**		**Cross section: 5+82 Spillway Wall Height: 10.2 ft**	
Tailwater % Wall Height[1]	**Tailwater depth (ft)**	**Apron length (ft)**	**Tailwater depth (ft)**	**Apron length (ft)**	**Tailwater depth (ft)**	**Apron length (ft)**
10%	0.69	5.8	1.02	9.2	1.02	5.9
20%	1.38	8.6	2.04	10.7	2.04	9.5
30%	2.07	9.2	3.06	12.0	3.06	12.7
40%	2.76	9.8	4.08	12.3	4.08	15.1
50%	3.45	10.1	5.10	12.7	5.11	16.1
60%	4.14		6.12		6.13	
70%	4.83		7.14		7.15	
80%	5.52		8.16		8.17	
90%	6.21		9.18		9.19	
100%	6.90		10.20		10.21	

Notes: 1. Tailwater depth divided by the wall height.

Based on this evaluation, we conservatively selected the following apron lengths:

- Sta. 5+82: 18 feet
- Sta. 10+90: 14 feet
- Sta. 17+80: 12 feet

7.0 Recommendations

The unique hydraulic conditions at the spillway do not facilitate the direct use of standard engineering reference documents to size the energy dissipation facilities. It is possible that a more detailed evaluation could result in a decrease to the size of the concrete apron. This would likely require developing a computation fluid dynamic (CFD) 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.

8.0 References

RJH Consultants, August 2020. *South Boulder Creek Regional Detention Project Hydrology Report*.

Unites States Bureau of Reclamation. June 1974. *Hydraulic Model Studies of Plunge Basins for Jet Flow. Engineering and Research Center*.

V. T. Chow, 1959. Open-Channel Hydraulics.

APPENDIX D.3

SPILLWAY ABUTMENT STABILITY EVALUATION

RJH CONSULTANTS, INC.

TECHNICAL MEMORANDUM

Project 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Robert Huzjak, P.E. - RJH Consultants, Inc.**

DATE: **January 26, 2022**

SUBJECT: **South Boulder Creek Regional Detention Project**
Spillway Abutment Stability Evaluation Memorandum

1.0 Introduction

1.1 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present methodology, results, and conclusions for the stability evaluation of the spillway abutment for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City).

1.2 Background

Proposed Project facilities include a spillway along the south side of U.S. Highway 36 (US36). The right abutment of the spillway will connect to the US36 roadway fill. The right abutment of the spillway will be higher in elevation than the spillway control section, and 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 extreme flood events (i.e., the Probable Maximum Flood (PMF) event). 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.

RJH identified and selected four potential failure modes (PFM) that could occur during extreme loading events and potentially compromise the spillway abutment:

- **PFM #1: Abutment Breach from Spillway Flows**. This failure mode would 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. An illustration of PFM #1 is presented on Figure 1.
- **PFM #2: Abutment Breach from Abutment Overtopping**. This failure mode would 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. An illustration of PFM #2 is presented on Figure 2.
- **PFM #3: Abutment Breach from South Boulder Creek Flows**. This failure mode would 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. An illustration of PFM #3 is presented on Figure 3.

- **PFM #4: Seepage Instability of Abutment**. This failure mode would 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. An illustration of PFM #4 is presented on Figure 4.

This memorandum is organized as follows:

- Section 2 presents hydrologic modeling that was performed to estimate the hydraulic loads (water levels and velocities) near the spillway abutment.
- Sections 3 through 6 present various analyses to evaluate how the hydraulic loads will affect the spillway abutment.
- Section 7 presents a summary of key conclusions.

2.0 Hydrologic Modeling

2.1 General

HEC-RAS 5.07 software from the U.S. Army Corps of Engineers (USACE) was used to perform hydraulic modeling of the proposed facilities for the Project. A two-dimensional (2D) approach was considered appropriate to adequately model flow paths with a significant lateral component over the left bank floodplain of South Boulder Creek during a PMF event.

The following hydraulic loading (HL) conditions were developed using HEC-RAS to evaluate stability of the spillway abutment:

1. HL1: PMF flows with the Project components installed and the US36 glare guard in place along the US36 median. This loading condition evaluated flows that overtop US36 downstream of the detention facility after flowing through the spillway (PFM #1).
2. HL2: Similar to HL1 described above, except with the glare guard not in place along US36. This loading condition was used to evaluate PFM #1.
3. HL3: PMF flows for existing conditions with the glare guard in place (i.e., no Project components constructed). This loading condition was modeled to provide a baseline that could be used to evaluate the relative effects that Project components would have on erosion of US36 during PMF flows.
4. HL4: PMF flows near the US36 spillway abutment. This loading condition was used to evaluate PFM #2.
5. HL5: PMF flows through South Boulder Creek and over the US36 embankment near the bridge abutment. This loading condition was used to evaluate PFM #3.

2.2 Model Geometry

2.2.1 Terrain

A 1-meter digital elevation model (DEM) obtained from the U.S. Geological Survey (USGS, 2020) was used to build a base terrain of existing conditions. This DEM included major bridge openings under US36 and South Boulder Road and was considered suitable for hydraulic modelling. The base terrain does not incorporate culvert openings, which is conservative for modeling extreme events.

Three additional independent DEMs were created using HEC-RAS Mapper to simulate the existing glare guard along US36 and the proposed embankment and spillway wall; these surfaces were merged with the topographic DEM as appropriate for the various simulated conditions.

A Manning´s “n” roughness coefficient raster layer covering the model extents was created from the land cover layer obtained from the 2016 National Land Cover Database from USGS. Each land cover class was assigned a roughness coefficient value following guidance from Colorado SEO (SEO, 2020).

A default Manning’s “n” value of 0.06 was used for any cell without a land cover class assigned.

2.2.2 2D Flow Area

The model consists of a 2D Flow Area that extends from HW93 to Baseline Road. Model extents are presented on Figure 5.

A base 100-foot square cell size was used for the flow area. The mesh was refined down to 15-foot square cell size near Project facilities and surrounding areas upstream and downstream of Project facilities.

Break lines with 5 foot spacing were introduced along major road-way embankments, the US36 glare guard, and the Project embankment and spillway, to improve the accuracy of lateral flow path patterns.

2.3 Analysis

2.3.1 Boundary Conditions

The inflow hydrograph used as an upstream boundary condition for the model corresponds to the 6-hour local storm PMP event over the watershed portion located downstream of Gross Reservoir (RJH, 2020).

The peak inflow hydrograph and the inflow volume are 76,731 cfs and 21,628 ac-ft respectively, and the hydrograph base time is approximately 15 hours. The hydrograph is presented on Figure 6.

2.3.2 Model Settings

A HEC-RAS unsteady flow plan was created for each hydraulic loading scenario, combining the corresponding geometry with the common unsteady flow file that incorporates boundary conditions.

The same computation settings were used for all of the unsteady plans, including: 2-second Computation Interval, 10-second Mapping Output Interval, 5-minute Hydrograph Output Interval, and 10-minute Detailed Output Interval.

2.4 Results

A profile line along the US36 embankment was used to identify the location of the initial overtopping location along the US36 embankment, and to select the overtopping hydrograph. Water surface elevations and depth time series were obtained at the desired locations using RASMapper.

Characteristics of the hydrographs that are predicted to impact the spillway abutment are summarized in Table 1. The data in Table 1 are HEC-RAS outputs and independent of the WinDAM modeling (see Section 3). The hydraulic loading conditions in the table are further presented in Sections 3 through 6. The overtopping hydrographs are presented on Figures 7 through 9.

TABLE 1
SUMMARY OF OVERTOPPING HYDROGRAPHS

Potential Failure Mode	Hydraulic Loading Condition	Summary	Flow Duration (hours)	Peak Flow (cfs)	Peak Flow Depth (feet)
PFM #1	HL1	US36 Overtopping with Project Components with Glare Guard	6.8	39,670	4.1
PFM #1	HL2	US36 Overtopping with Project Components without Glare Guard	6.5	45,170	4.2
PFM #1	HL3	US36 Overtopping with Existing Conditions with Glare Guard	6.5	30,470	3.4
PFM #2	HL4	Erosion near Spillway Abutment	5.0	122	5.8
PFM #3	HL5	Erosion near US36 Bridge	18.0	14,058	11.5

3.0 PFM #1: Abutment Breach from Spillway Flows

3.1 General

We used the computer program WinDAM, developed by the Natural Resources Conservation Service (NRCS), to setup and perform erosion analyses of the US36 embankment for various flood loading conditions. WinDAM contains numerous numerical processes for routing flows through reservoirs, dams, and spillways and evaluating possible erosion from those flows. For our analyses, we used WinDAM specifically to estimate the three-dimensional erosion of an embankment (i.e., US36) during an overtopping flow event.

We used WinDAM to evaluate erosion of US36 during the following hydraulic loading (HL) conditions:

1. HL1: PMF flows with the Option 1 100-year Project components installed and the US36 glare guard in place along the US36 median. This loading condition evaluated flows that overtop US36 downstream of the detention facility after flowing through the spillway.
2. HL2: Similar to HL1 described above, except with the glare guard not in place along US36.
3. HL3: PMF flows for existing conditions with the glare guard in place (i.e., no Project components constructed).

HL1 and HL2 were performed to estimate the expected extents of US36 erosion and evaluate whether this erosion could encroach on the spillway abutment. HL3 was performed to estimate whether erosion of US36 is predicted to occur without Project facilities in place. The HL3 results were compared to those from HL1 and HL2 to evaluate the overall effect that Project facilities would have on US36 erosion during the PMF.

3.2 HL1

3.2.1 General

WinDAM was initially used to model overtopping and erosion of the US36 roadway embankment with the glare guard in place for HL1. First, a base-case analysis was performed for HL1 using representative model inputs. The base-case analysis is presented in Section 3.2.2. Next, sensitivity analyses were performed to evaluate the stability of model results and the sensitivity of results to changes in model inputs. The sensitivity analyses for HL1 are presented in Section 3.2.3.

The model extents for HL1 extend along the US36 embankment from the Foothills Parkway on-ramp to South Boulder Creek. The embankment within the model extents generally consists of 4H:1V upstream and downstream slopes. The slopes are lightly vegetated and US36 pavement is present along the crest of the embankment. A plan and profile of the embankment within the model extents is presented on Figures 10 and 11, respectively.

3.2.2 HL1 - Base Case Scenario

3.2.2.1 Hydraulic Loading

RJH used HEC-RAS to model the PMF with the proposed Project features and the glare guard in place. RJH used the HEC-RAS results to identify the location of overtopping along US36, the magnitude of the overtopping hydrograph, and the magnitude of the tailwater hydrograph (north of US36). Based on the HEC-RAS model results, the embankment will first overtop approximately 1,145 feet west of the US36 bridge over South Boulder Creek, which is not the lowest portion along the US36 profile. The overtopping location is shown on Figures 10 and 11.

The hydrograph of the portion of flood flows that are predicted to flow across US36 at the overtopping location is presented on Figure 7. The approximate flow duration for the hydrograph for HL1 is 6.8 hours and the peak flow is expected to be 39,670 cfs. The peak flow depth along the US36 embankment is estimated to be 4.1 feet. The tailwater rating curve that is predicted to exist north of US36 at the overtopping location is presented in Table 2.

TABLE 2
TAILWATER RATING DATA FOR HL1

Tailwater Elevation (ft)	Reservoir Outflow (cfs)
5354.00	0
5355.94	2,500
5356.76	5,000
5357.19	7,500
5357.55	10,000
5357.81	12,500
5358.05	15,000
5358.23	17,500
5358.38	20,000
5358.50	22,500
5358.62	25,000
5358.75	27,500
5358.85	30,000
5358.98	32,500
5359.09	35,000
5359.18	37,500
5359.26	39,671

3.2.2.2 US36 Material Properties

WinDAM models an embankment as a homogenous material. RJH developed material properties for the US36 fill based on available data, guidance from NRCS documents, and judgment. We used reasonable average material properties based on available data for the base-case model. The material properties are summarized in Table 3 below.

TABLE 3
MATERIAL PROPERTIES FOR WINDAM MODEL

Material Property	Value
Total Unit Weight (pcf)	127.5
Erodibility, $K_{d,}$ (ft/h)/(psf)	2
Undrained Shear strength (psf)	1,200
Critical Shear Stress (psf)	0.0009

WinDAM neglects the US36 pavement because the embankment is modeled as a homogenous material. This is conservative for evaluating erodibility of the underlying embankment fill.

3.2.2.3 US36 Geometry Inputs and Model Settings

The embankment crest elevation and height generally vary along the length of the embankment profile (Figure 11). We selected an embankment crest elevation of 5360.5 ft, an embankment width of 110 feet, and an embankment base elevation of El. 5354 based on the embankment geometry at the initial overtopping location. For simplicity in the model, we considered that the embankment crest was relatively horizontal along the length of US36 for the base-case analyses. We input a slightly lower crest elevation at the location where HEC-RAS predicted overtopping to initiate so that WinDAM would initiate overtopping of the embankment at the correct position along the profile. The final scour width for each analysis is centered around the initial overtopping location identified in the HEC-RAS model.

RJH did not include a principal spillway or auxiliary spillways as part of the analysis. WinDAM requires the user to enter a reservoir stage-storage table because the program will route the input flood hydrograph through the reservoir and over the embankment. Since the overtopping hydrograph obtained from HEC-RAS had already been routed through the site, we intentionally assigned very minimal storage so the entire hydrograph would be routed over US36.

3.2.2.4 Results

The final scour width for the base-case scenario of HL1 was about 16 feet (along US36). The final headcut length extended 13 feet upstream of the downstream edge of the embankment crest. Erosion was predicted to extend for the full height of the embankment (6.5 feet). The maximum overtopping head is 3.7 feet. The edge of the breach is approximately 920 feet from the proposed spillway abutment location.

A summary of the results for HL1 are included in Table 6 in Section 3.5.

3.2.3 HL1 Sensitivity Analysis

Sensitivity analyses were performed for HL1 to evaluate how reasonable changes in input parameters would affect the predicted erosion. We performed eight sensitivity analyses by individually adjusting one input parameter at a time that was anticipated to increase the predicted erosion. The parameters that were adjusted during sensitivity analyses and the analysis results are summarized in Table 6 (Section 3.5).

Each of the sensitivity analyses resulted in varying widths and lengths of erosion; however for all scenarios the erosion was predicted to extend for the full height of the US36 embankment. Based on the results shown in Table 6, the erodibility results are most sensitive to the embankment fill material properties. The poorer embankment fill properties considered during the sensitivity analyses are summarized in Table 6 and were based on typical published ranges. Even if poorer material properties are considered, the final scour width for HL1 is predicted to be 108 feet, which would not extend closer than about 880 feet from the proposed spillway abutment location. An aerial view of the erosion extents for the base case scenario and the sensitivity analysis with poorer material properties is presented on Figure 12.

3.3 HL2

3.3.1 General

HL2 was performed to model the overtopping and erosion of the US36 embankment without the glare guard in place.

RJH developed the overtopping location, hydrograph data, and tailwater data for HL2 based on HEC-RAS model results. The initial overtopping location was approximately 850 feet west of the US36 bridge over South Boulder Creek as shown on Figures 10 and 11. The hydrograph of flows that are predicted to flow across US36 at the overtopping location for HL2 is presented on Figure 7. The approximate flow duration for the hydrograph for HL2 is 6.5 hours and the peak flow is expected to be 45,170 cfs. The peak flow depth along the US36 embankment is estimated to be 4.2 feet. The tailwater rating curve that is predicted to exist north of US36 at the overtopping location is presented in Table 4.

TABLE 4
TAILWATER RATING DATA FOR HL2

Tailwater Elevation (ft)	Reservoir Outflow (cfs)
5355.00	0
5356.81	2,500
5357.47	5,000
5357.99	7,500
5358.44	10,000
5358.79	12,500
5359.09	15,000
5359.35	17,500
5359.57	20,000
5359.75	22,500
5359.88	25,000
5359.97	27,500
5360.03	30,000
5360.10	32,500
5360.17	35,000
5360.26	37,500
5360.33	40,000
5360.41	42,500
5360.47	45,000

We selected an embankment crest elevation of 5361.5 ft, an embankment width of 110 feet, and an embankment base elevation of 5355 ft based on the embankment geometry at the initial overtopping location for HL2.

First, a base-case analysis was performed for HL2 using representative material properties for the US36 embankment fill (see Table 3 for material properties). One sensitivity analysis was performed for HL2 with poorer material properties. This scenario was selected for analysis because the erodibility results were most sensitive to the embankment fill material properties based on results for HL1.

3.3.2 Results

The final scour width for the HL2 base-case analysis was about 21 feet (along US36). The final headcut length was 17 feet and extended for the full height of the embankment (6.5 feet). The maximum overtopping head was 4.0 feet. The final scour width and final headcut length for HL2 are slightly greater than the results for HL1.

The sensitivity analysis for HL2 with poorer material properties resulted in a complete breach of the US36 roadway embankment. The breach width was 155 feet.

A summary of the results for HL2 are included in Table 6 in Section 3.5. An aerial view of the erosion extents for the HL2 base case scenario and the sensitivity analysis with poorer material properties is presented on Figure 12.

3.4 HL3

3.4.1 General

HL3 was performed to model the overtopping and erosion of the US36 embankment based on existing conditions without proposed Project features.

RJH developed the overtopping location, hydrograph data, and tailwater data for HL3 based on HEC-RAS model results. The initial overtopping location was located approximately 2,150 feet west of the US36 bridge over South Boulder Creek, and near the Foothills Parkway on-ramp. The hydrograph of flows that are predicted to flow across US36 at the overtopping location for HL3 is presented on Figure 7. The approximate flow duration for the hydrograph for HL3 is 6.5 hours and the peak flow is expected to be 30,470 cfs. The peak flow depth along the US36 embankment is estimated to be 3.4 feet. The tailwater rating curve that is predicted to exist north of US36 at the overtopping location is presented in Table 5 below.

TABLE 5
TAILWATER RATING DATA FOR HL3

Tailwater Elevation (ft)	Reservoir Outflow (cfs)
5350.00	0
5353.73	2,500
5353.97	5,000
5354.15	7,500
5354.27	10,000
5354.32	12,500
5354.36	15,000
5354.46	17,500
5354.53	20,000
5354.60	22,500
5354.68	25,000
5354.75	27,500
5354.92	30,470

The embankment at the initial overtopping location for HL3 is generally wider and shorter compared to the HL1 and HL2 analyses. We selected an embankment crest elevation of 5357.5 ft, an embankment width of 190 ft, and an embankment base elevation of 5350 ft based on embankment geometry at the initial overtopping location for HL3.

First, a base-case analysis was performed for HL3 using representative material properties for the US36 embankment fill (see Table 3 for material properties). Two sensitivity analyses were performed for HL3. One analysis included poorer material properties because the erodibility results were most sensitive to the embankment fill material properties based on results for HL1. The second analysis extended the dam base to the bottom of Viele channel (El. 5338).

3.4.2 Results

The final scour width for the HL3 base-case analysis was about 10 feet (along US36). The final headcut length was 7 feet. Erosion was predicted to extend 7.5 feet deep. The maximum overtopping head was 3.3 feet. The extent of erosion for HL3 is predicted to be less than that for HL1 and HL2.

Two sensitivity analyses were performed for HL3, one with poorer material properties and one with the base of the embankment extending to the bottom of Viele Channel. The sensitivity analysis for HL3 with poorer material properties resulted in a scour geometry that was about 47 feet wide, 46 feet long, and 7.5 feet deep. The sensitivity analysis with the base of the embankment extending to the bottom of Viele Channel resulted in a scour geometry that was about 10 feet wide, 7 feet long, and up to 19.5 feet deep.

A summary of the results for HL3 are included in Table 6 in Section 3.5.

3.5 Summary of Results

Erosion results for PFM #1 (HL1, HL2, HL3) are summarized in Table 6.

TABLE 6
SUMMARY OF RESULTS FOR HL1, HL2, AND HL3

Sensitivity Analysis Number	Property Changed	Basis for Evaluation	Final Scour Width(1) (ft)	Final Headcut Length(2) (ft)	Final Erosion Depth(3) (ft)	Maximum Overtopping Head(4) (ft)
HL1 - BASE			16	13	6.5(6)	3.7
HL1-S1	No tailwater data input	Erosion might increase if tailwater was not present north of US36 to dissipate energy.	15	11	6.5(6)	3.7
HL1-S2	Bare downstream slope	Erosion might increase if the downstream slope was soil-covered instead of grass-covered.	20	18	6.5(6)	3.7
HL1-S3	Poorer material properties ◦Erodibility = 10 (ft/hr)/(psf) ◦Undrained shear strength = 835 psf ◦Critical shear stress = 0 psf	Erosion might increase if the embankment fill was more erodible. Unit weight was not adjusted because this value is relatively well-defined and is not expected to vary widely.	108	102	6.5(6)	3.7
HL1-S4	2:1 Upstream and downstream slopes instead of 4:1	Erosion might increase if water flowed down a steeper downstream slope. The extent of erosion might also widen if the overall embankment base was narrower.	21	18	6.5(6)	3.7
HL1-S5	Varied elevation dam profile ◦Used actual elevation profile along US36 instead of a horizontal profile	Evaluate how the results were affected by modeling US36 as a horizontal embankment.	14	9	3.5(6)	7.2
HL1-S6	Shorter dam height ◦Base of Dam El. 5358 instead of El. 5354.	The extent of erosion might widen if there was a shorter embankment cross section to erode through.	18	14	2.5(6)	3.7
HL1-S7	Taller dam height ◦Dam Crest El. 5367 instead of 5361.	Conservatively simulate a taller dam in response to the results from HL1-S6.	18	15	12.5(6)	3.7
HL1-S8	Narrow V-Notch in dam profile instead of broad notch	Evaluate how the results were affected by the geometry of the notch at the first location of overtopping.	19	15	6.5(6)	4.0
HL2 - BASE			21	17	6.5(6)	4.0
HL2-S1	Poorer material properties ◦Erodibility = 10 (ft/hr)/(psf) ◦Undrained shear strength = 835 psf ◦Critical shear stress = 0 psf	Erosion might increase if the embankment fill was more erodible. Unit weight was not adjusted because this value is relatively well-defined and is not expected to vary widely.	155	162(5)	6.5(6)	4.0
HL3 - BASE			10	7	7.5(6)	3.3
HL3-S1	Poorer material properties ◦Erodibility = 10 (ft/hr)/(psf) ◦Undrained shear strength = 835 psf ◦Critical shear stress = 0 psf	Erosion might increase if the embankment fill was more erodible. Unit weight was not adjusted because this value is relatively well-defined and is not expected to vary widely.	47	46	7.5(6)	3.3
HL3-S2	Base of the dam extend to bottom of Viele Channel (El. 5338).	The extent of erosion might be affected by a taller dam height.	10	7	19.5(6)	3.3

Notes:
1. Along the model profile.
2. Reported as the extent of erosion upstream from the downstream side of the embankment crest.
3. Total depth of erosion below the crest.
4. The maximum overtopping head is a similar measurement to the “Peak Flow Depth” measurement in Table 1. The “Peak Flow Depth” measurement is an output of HEC-RAS (and not an input into WinDAM) and the maximum overtopping head is an output of WinDAM. The values in the two tables generally agree.
5. The embankment completely breached in this analysis. Final headcut length is the width of the embankment, including the slopes.
6. The predicted erosion extended to the total height of the embankment.

4.0 PFM #2: Abutment Breach from Abutment Overtopping

4.1 General

Erosion modeling was performed to model overtopping and erosion of the US36 embankment between the spillway abutment and the US36 glare guard and to support design of erosion protection on the spillway abutment. The WinDAM model used hydrologic inputs from HL4 to estimate the erosion.

The model profile extended from the US36 glare guard to the spillway abutment at the US36 embankment. A plan and profile of the model extents are presented on Figures 13 and 14, respectively. In this model, we considered the profile of the modeled embankment to be the length between the spillway abutment and the US36 glare guard. We considered the slopes of the embankment cross section to be the slopes of the US36 embankment (4H:1V) and we considered the crest width in the model to be a width slightly larger than the width of the spillway at the abutment location (20 feet). The slopes are lightly vegetated and US36 pavement extends for a portion of the crest profile. The base of the dam elevation was considered to be the elevation of the US36 multi-use path.

RJH developed the overtopping location, hydrograph data, and tailwater data for the model based on HEC-RAS model results. The initial overtopping location is at the lowest point along the model profile, at the spillway abutment location, as shown on Figures 13 and 14. The hydrograph of flows that are predicted to flow across the model profile is presented on Figure 8. The approximate flow duration for the hydrograph is 5.0 hours and the peak flow is expected to be 122 cfs. The peak flow depth along the profile is calculated to be 5.8 feet. The tailwater rating curve that is predicted at the downstream end of the profile is presented in Table 7.

TABLE 7
TAILWATER RATING DATA FOR HL4

Tailwater Elevation (ft)	Reservoir Outflow (cfs)
5368.97	0.00
5369.11	12.15
5369.20	13.47
5369.31	15.26
5369.41	17.08
5369.49	18.80
5369.59	21.18
5369.71	24.58
5369.81	27.91
5369.90	31.43
5369.99	35.07
5370.10	39.55
5370.20	44.03
5370.30	48.50
5370.41	53.43
5370.50	57.94
5370.60	62.58
5370.70	67.54
5370.80	72.17
5370.90	77.18
5371.00	81.81
5371.10	86.79
5371.20	91.84
5371.30	97.03
5371.40	102.63
5371.50	107.53
5371.60	112.41
5371.70	117.38
5371.80	122.17

Based on HEC-RAS model results, overtopping of the abutment is predicted to initiate from water that flows from southeast to northwest. Flood waters rise nearly uniformly on both sides of the abutment, and therefore overtopping occurs as submergence with very little differential head cross the abutment.

4.2 Results

The results for the base case scenario for the model did not produce any scour, erosion, or headcut along the model profile. The maximum overtopping head was 4.5 feet, but negligible differential head existed across the wall during the overtopping event and therefore erosion did not occur.

Two sensitivity analyses were performed, one with poorer material properties and one with no tailwater data input. The sensitivity analysis for HL4 with poorer material properties resulted in no erosion similar to the base case scenario. The sensitivity analysis with no tailwater resulted in a scour geometry that was about 4 feet wide, 4 feet long, and up to 4 feet deep.

A summary of the results for the base case scenario and sensitivity analyses are presented in Table 8. An aerial view of the erosion extents for the HL4 sensitivity analysis with no tailwater data is presented on Figure 15.

TABLE 8
SUMMARY OF RESULTS FOR PFM #2 (HL4)

Sensitivity Analysis Number	Property Changed	Basis for Evaluation	Final Scour Width[(1)] (ft)	Final Headcut Length[(2)] (ft)	Final Erosion Depth[(3)] (ft)	Max. Over-topping Head (ft)
HL4			0	0	0	4.5
HL4 – S1	Poorer material properties: - Erodibility = 10 (ft/hr)/(psf) - Undrained shear strength = 835 psf - Critical shear stress = 0 psf	Erosion might increase if the embankment fill was more erodible. Unit weight was not adjusted because this value is relatively well-defined and is not expected to vary widely.	0	0	0	5.8
HL4 – S2	No tailwater data input	Erosion might increase if tailwater was not present downstream of the spillway abutment to dissipate energy.	4	4	4[(4)]	3.1

Notes:

1. Along the model profile.
2. Reported as the extent of erosion upstream from the downstream side of the embankment crest.
3. Total depth of erosion below the crest.
4. The predicted erosion extended the total height of the modeled embankment (i.e., from the US36 multi-use trail to the crest of the spillway abutment location).

5.0 PFM #3: Abutment Breach from South Boulder Creek Flows

5.1 General

The third failure mode we evaluated was erosion of the spillway abutment from flows that remain in South Boulder Creek (upstream of the spillway) and flow beneath the US36 bridge because, erosion could potentially extend west along US36 and could compromise the integrity of the spillway.

We evaluated this potential failure mode using two different methods: (1) Estimating the required riprap diameter necessary to prevent erosion in the areas of high velocity flows, and (2) Using the WinDAM software to model the predicted lateral erosion along US36 near the US36 bridge over South Boulder Creek if no armoring (riprap) was provided.

5.2 Slope protection

We used procedures from FHWA HEC-23 to calculate the required size of riprap slope protection based on the expected flow depths and velocities. We estimated the degree of slope protection required at three locations around the abutment location. The locations are presented on Figures 16 and 17.

Velocities and flow depths predicted by HEC-RAS at each location are presented in Table 9.

TABLE 9
FLOW VELOCITY AND DEPTHS NEAR SPILLWAY ABUTMENT

Location	Velocity (ft/s)	Flow Depth (ft)
Upstream Toe of US36 Embankment	1	12
Downstream Toe of US36 Embankment	3	2
Along South Boulder Creek	5	12

Based on the predicted flow velocities and depths, and guidance from FHWA HEC-23, the median riprap diameter (D50) needed is about 5 inches along South Boulder Creek. The calculation from HEC-23 resulted in a median riprap diameter of 0.75 inch and 1.75 inches at the upstream and downstream toe, respectively, of the US36 embankment. We conclude that riprap is not anticipated to be required at these locations to protect against erosion.

5.3 Creek Erosion – WinDAM Model

5.3.1 General

The WinDAM modeling of the erosion near the US36 bridge over South Boulder Creek was setup similarly to the WinDAM models for PFM #1 and PFM #2. The WinDAM model used hydrologic inputs from HL5 to estimate the erosion.

RJH developed the hydrograph data, and tailwater data for the model based on HEC-RAS PMF results with the glare-guard in place. The model profile began approximately 50 feet east of South Boulder Creek and extended 470 feet west of South Boulder Creek and followed the US36 embankment, without the bridge over South Boulder Creek as shown on Figure 18. We conservatively neglected the concrete surfacing in the model.

The initial overtopping location was chosen at the low point of South Boulder Creek. A plan and profile of the model extents is presented on Figures 18 and 19. The hydrograph of flows that are predicted to flow across the model profile is presented on Figure 9. The approximate flow duration for the hydrograph is 18 hours and the peak flow is expected to be 14,060 cfs. The peak flow depth is estimated to be 11.5 feet. The tailwater rating curve that is predicted to exist north of US36 is presented in Table 10.

TABLE 10
TAILWATER RATING DATA FOR PFM #3 (HL5)

Tailwater Elevation (ft)	Reservoir Outflow (cfs)
5353.60	0.00
5354.10	778.00
5354.50	1,082.30
5355.10	1,669.00
5355.50	2,145.90
5356.00	2,870.40
5356.50	3,922.10
5357.00	5,307.00
5357.50	6,948.70
5358.00	8,786.20
5358.50	10,906.30
5359.00	13,156.20

We selected an embankment crest elevation and embankment width that were representative of the existing ground surface of the bridge abutments and the natural ground surface beneath the bridge. An embankment height is a required input of the model, and we considered the embankment height to be minimal (2 feet) at the initial overtopping location in the analysis to force the model to erode laterally instead of down the embankment.

First, a base-case analysis was performed for the model using representative material properties for the US36 embankment fill that are consistent with the material properties in the other WinDAM models.

5.3.2 Sensitivity analyses

Sensitivity analyses were performed for the model to evaluate how reasonable changes in input parameters would affect the predicted erosion. We performed four sensitivity analyses by individually adjusting one input parameter at a time that was anticipated to increase the predicted erosion. The parameters that were adjusted during sensitivity analyses and the analysis results are summarized in Table 11 in Section 5.3.2.

5.3.3 Results

Erosion results for PFM #3 (HL5) are summarized in Table 11. An aerial view of the erosion extents for the sensitivity analysis with no tailwater data is presented on Figure 20.

TABLE 11
SUMMARY OF RESULTS FOR HL5

Sensitivity Analysis Number	Property Changed	Basis for Evaluation	Final Scour Width[1] (ft)	Final Headcut Length[2] (ft)	Final Erosion Depth[3] (ft)	Max. Over-topping Head (ft)
HL5			**47**	**5.2**	**2[4]**	**14.4**
HL5 – S1	Shorter embankment height - 1 foot instead of 2 feet	Erosion may increase laterally with a shorter embankment height	45	1.2	1[4]	14.4
HL5 – S2	No tailwater data input	Erosion might increase if tailwater was not present downstream of the bridge opening to dissipate energy.	50	6.9	2[4]	14.4
HL5 – S3	Taller embankment height - 10 feet instead of 2 feet	The erosion depth may increase with a taller embankment.	123	146.8	10[4]	8.8
HL5 – S4	Shifted the model profile to start at the middle of South Boulder Creek.	This change will force all of the erosion to occur west of South Boulder Creek towards the spillway abutment.	30	7.7	2[4]	17.3

Notes:
1. Along the model profile.
2. Reported as the extent of erosion upstream from the downstream side of the embankment crest.
3. Total depth of erosion below the crest.
4. The predicted erosion extended the total height of the embankment.

6.0 PFM #4: Seepage Instability of Abutment

6.1 General

RJH performed seepage analyses through the abutment adjacent to the spillway to evaluate excessive seepage forces and uplift forces downstream of the spillway. Two-dimensional steady-state seepage analyses were performed using Seep/W, which is part of the GeoStudio 2021 software package.

Analyses were performed for one section of the spillway abutment along the US36 embankment nearest to the spillway. The analyzed cross section is similar to the cross section evaluated for PFM #2 and is presented on Figure 21.

The cross section has a 35 ft wide crest at El. 5364, and 20H:1V upstream and downstream slopes to simulate the fill that will be placed to route the multi-use trail over the spillway abutment. The embankment slopes down on the upstream and downstream sides to a flat section that is 42 feet wide that includes the US36 multi-use trail and is at El. 5363.5. The ground surface slopes down, at about a 5% grade, to the natural ground surface at El. 5359. The evaluated section extends approximately 290 feet upstream and 720 feet downstream of the crest centerline.

The material properties and stratigraphy were developed based on RJH geotechnical investigations, previous geotechnical investigations in the area, published data, and engineering judgement.

Material properties that were used for the seepage model are presented in Table 12.

TABLE 12
MATERIAL PROPERTIES FOR SEEPAGE ANALYSIS

	Alluvium	**US36 Embankment Fill**	**Pierre Shale**
Anisotropic Ratio (K_v/K_h)	0.1	0.17	1
Horizontal Hydraulic Conductivity, K_h (ft/s)	$2x10^{-5}$	$2x10^{-6}$	$3x10^{-9}$
Porosity, η	0.36	0.35	0.28
Geostudio Material(1)	Sand	Clay	Clay

Note:
1. Volumetric water content functions and unsaturated hydraulic conductivity functions were estimated using these example materials that are built-in to the GeoStudio database.

6.2 Analysis

Seep/W is a finite element seepage model. We discretized the modeled cross section using a square mesh with 1-foot elements. We performed analyses that evaluated steady-state seepage conditions.

Total head and water rate boundary conditions were used. Total head boundary conditions allow water to enter or exit the model as required to maintain the specified head at the location of the boundary condition and were used to simulate a specific reservoir level and groundwater condition. We applied a total head boundary to the upstream side of the model to simulate water detained at El. 5364, which is near the top of the spillway crest. We also applied a total head boundary to the downstream end of the model to represent natural far-field alluvial groundwater conditions, at El. 5354. The alluvial groundwater condition in the model is based on groundwater measurements in the piezometers installed on-site. The water rate boundary condition with a seepage face review was generally applied to ground surface downstream of the embankment. The water rate boundary condition allows water to exit the model if the calculated total head is higher than the elevation of the boundary.

We analyzed two conditions for seepage stability:

1. Uplift on the base of the low-permeable fill, beneath the downstream trail, by elevated water pressures within the alluvium.
2. Excessive exit gradients (heave) that occur where alluvium is exposed at the ground surface in the ditch.

We analyzed three seepage scenarios: (1) Using a typical value (see Table 13) for the permeability of the alluvium and bedrock (2) Using a higher permeable material, where the horizontal hydraulic conductivity for the alluvium (3) Using a higher permeable material for the bedrock.

6.3 Results

Steady-state seepage results are summarized in Table 13.

TABLE 13
SUMMARY OF SEEPAGE RESULTS FOR PFM #4

Basis	Alluvium Kh (ft/s)	Pierre Shale Kh (ft/s)	Uplift on fill FS(1)(2) (>2 required)	Exit Gradient(3)	Heave in alluvium FS(2) (>4 required)
Typical alluvium and bedrock permeability	$2x10^{-5}$	$3x10^{-9}$	1.4	0.08	13
Higher permeability alluvium and typical permeability bedrock	$6x10^{-4}$	$3x10^{-9}$	1.7	0.05	21
Typical permeability alluvium and higher permeability bedrock	$2x10^{-5}$	$5x10^{-6}$	1.4	0.10	10

Notes:
1. The uplift was calculated based on the groundwater results at the upstream end of the downstream multi-use trail.
2. Required safety factors are from USBR Design Standard No. 13 Embankment Dams.
3. The exit gradient was calculated based on the results the downstream ground surface contact between the fill and the alluvium.

The calculated safety factors for heave exceed the recommended values, but the safety factors for uplift are less than the recommended values. However, in our opinion the model results demonstrate that seepage conditions through the spillway abutment are acceptable for the following reasons:

- The modeled hydraulic scenarios were very conservative. We modeled steady state seepage conditions; however, we do not expect steady state seepage conditions to develop during short-term flood impoundments. We also modeled the floodwater to the top of the spillway wall with no tailwater, which is conservative.
- The modeled cross section was simplified and conservative. The model geometry did not include the following features that will improve seepage stability:
  - Chimney and blanket drains that will be included in the fill placed for the multi-use trail ramp.
  - A blanket drain that is anticipated to be constructed beneath the spillway apron at the downstream end of the model.
- Even with all of the compounded conservatism in the seepage modeling approach, the results demonstrate that the safety factor against uplift of the fill is still greater than 1.

7.0 Conclusions

Based on the information in this analysis memorandum, we conclude the following:

- The four potential failure modes evaluated in this memorandum are not predicted to adversely affect the stability of the spillway abutment.

- The WinDAM model results are most sensitive to the erodibility of the US36 embankment fill and the magnitude of the overtopping hydrograph. Model results are relatively less sensitive to other geometric input parameters.
- Erosion of the US36 embankment is predicted to be more extensive with Project components in place compared to existing conditions. This is likely caused by the post-Project overtopping hydrograph having a higher peak flow than the existing conditions hydrograph.
    - Erosion during a pre-Project PMF flood event is anticipated to remain on the westbound shoulder of US36, whereas post-Project erosion from a PMF flood is predicted to encroach onto the westbound traffic lanes.
- It is unknown whether the glare guard would stay in place during the PMF. Based on these analyses, the presence or absence of the glare guard does not affect our interpretation of the spillway abutment stability.
- The most critical US36 embankment model analyzed was the sensitivity analysis for HL2 with poorer embankment materials. This is the only analysis where the embankment fully breached.
- The erosion of the US36 embankment does not encroach on the proposed spillway abutment. Even if poorer material properties are considered for the US36 embankment fill (which is very conservative), erosion remained at least 550 feet away from the spillway abutment.
- Erosion is not anticipated to occur near the spillway abutment during overtopping of the abutment. This is reasonable because the magnitude of flow, velocity of flow, and differential head are very low at this location. Figure 22 presents a graph at two locations along the spillway abutment profile that supports our conclusions.
    - Some erosion around the spillway abutment resulted from the sensitivity analysis with no tailwater data input. This situation where no tailwater is present during the overtopping of the spillway abutment is very unlikely to occur and resulted in minimal erosion on the downstream end of the spillway abutment.
- The predicted erosion extents for PFM #3 are 5 feet wide (along SBC), 47 feet long, and 5 feet deep. Erosion is predicted to remain within the bridge opening and is not predicted to extend laterally through the US 36 fill.
- The seepage analysis results demonstrate that seepage conditions through the abutment will not create conditions that would adversely affect abutment stability.

8.0 References

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Temple, D.M., K.M. Ahring, and A.G. Davis. (1987). *Stability Design on Grass-Lined Open Channels, AH-667.* U.S. Department of Agriculture, Agricultural Research Service, Washington, DC.

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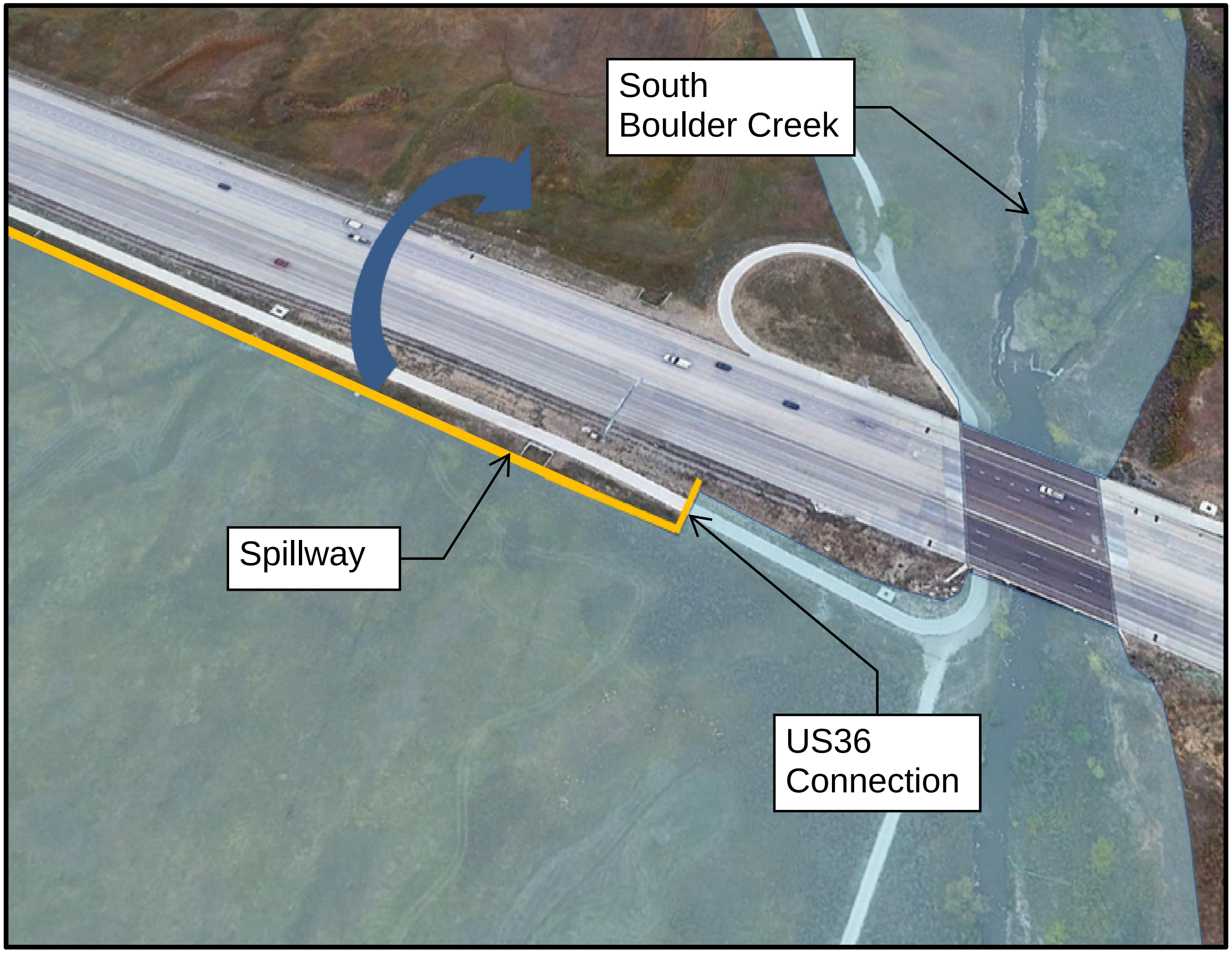
9.0 Attachments

Attachment A: Figures



NOTE:
1. BLUE ARROW INDICATES DIRECTION OF WATER FLOW.

REPRODUCE IN COLOR

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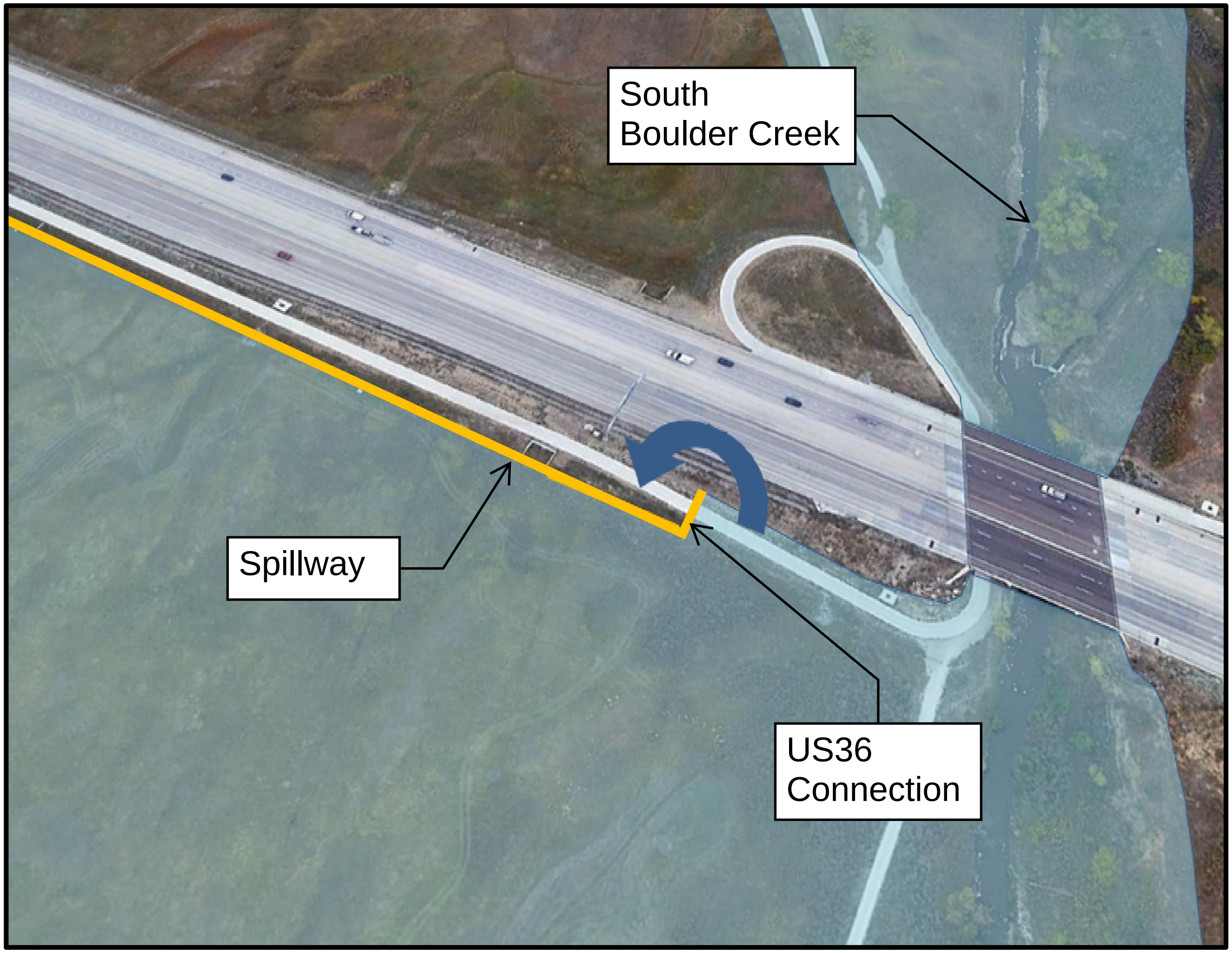
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

FLOW DIRECTION
FOR PFM #1

July 2021

Figure 1

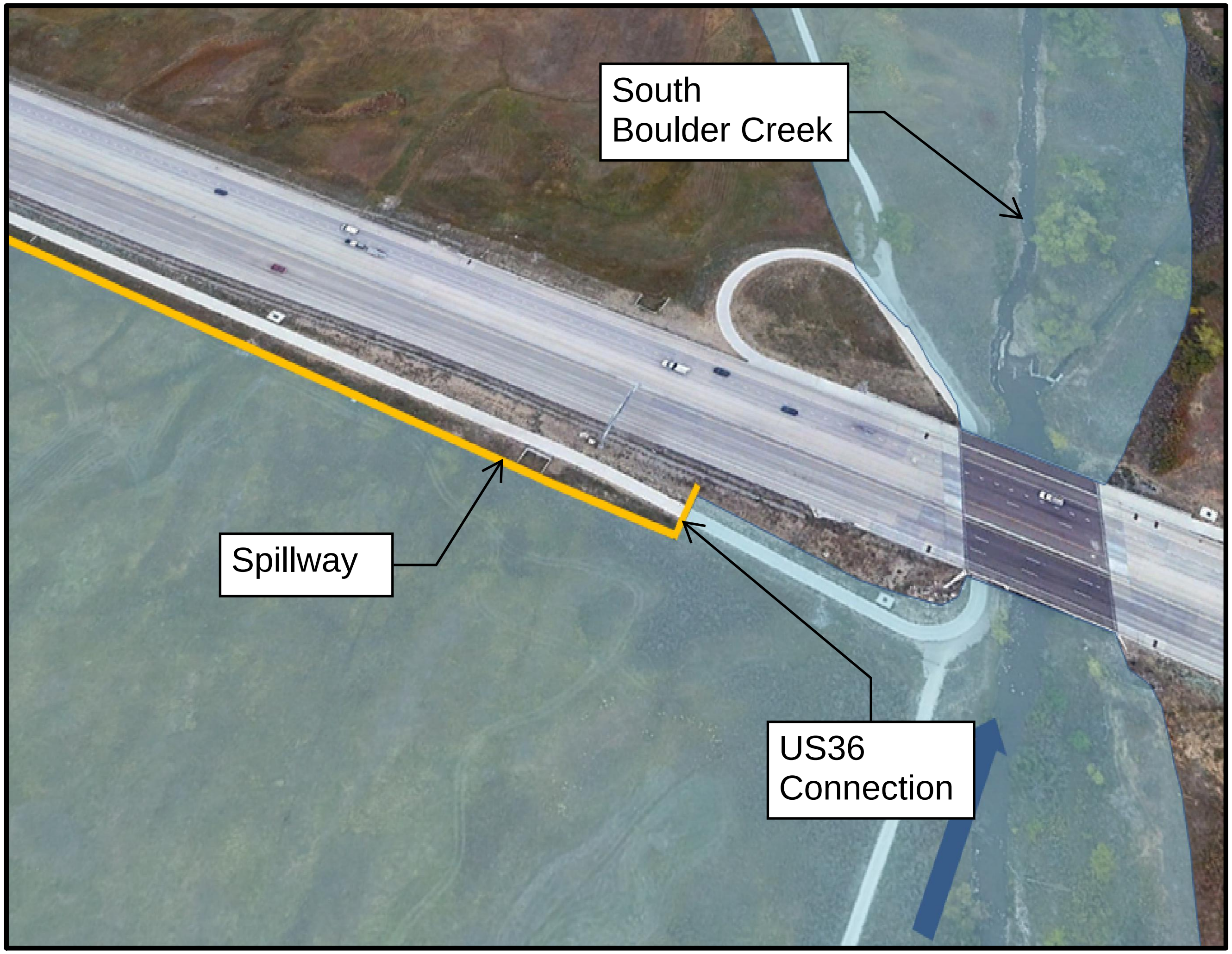


REPRODUCE IN COLOR

NOTE:
1. BLUE ARROW INDICATES DIRECTION
OF WATER FLOW.

RJH CONSULTANTS, INC.	SOUTH BOULDER CREEK REGIONAL DETENTION CONCEPT DESIGN	FLOW DIRECTION FOR PFM #2
	PROJECT NO. 16134	July 2021 Figure 2

P:\16134 – SOUTH BOULDER CREEK\CAD\FIGURES\2020-12-16_WINDAM_MEMO_FIGS.DWG 6/15/2021 10:31 AM



REPRODUCE IN COLOR

NOTE:
1. BLUE ARROW INDICATES DIRECTION
OF WATER FLOW.

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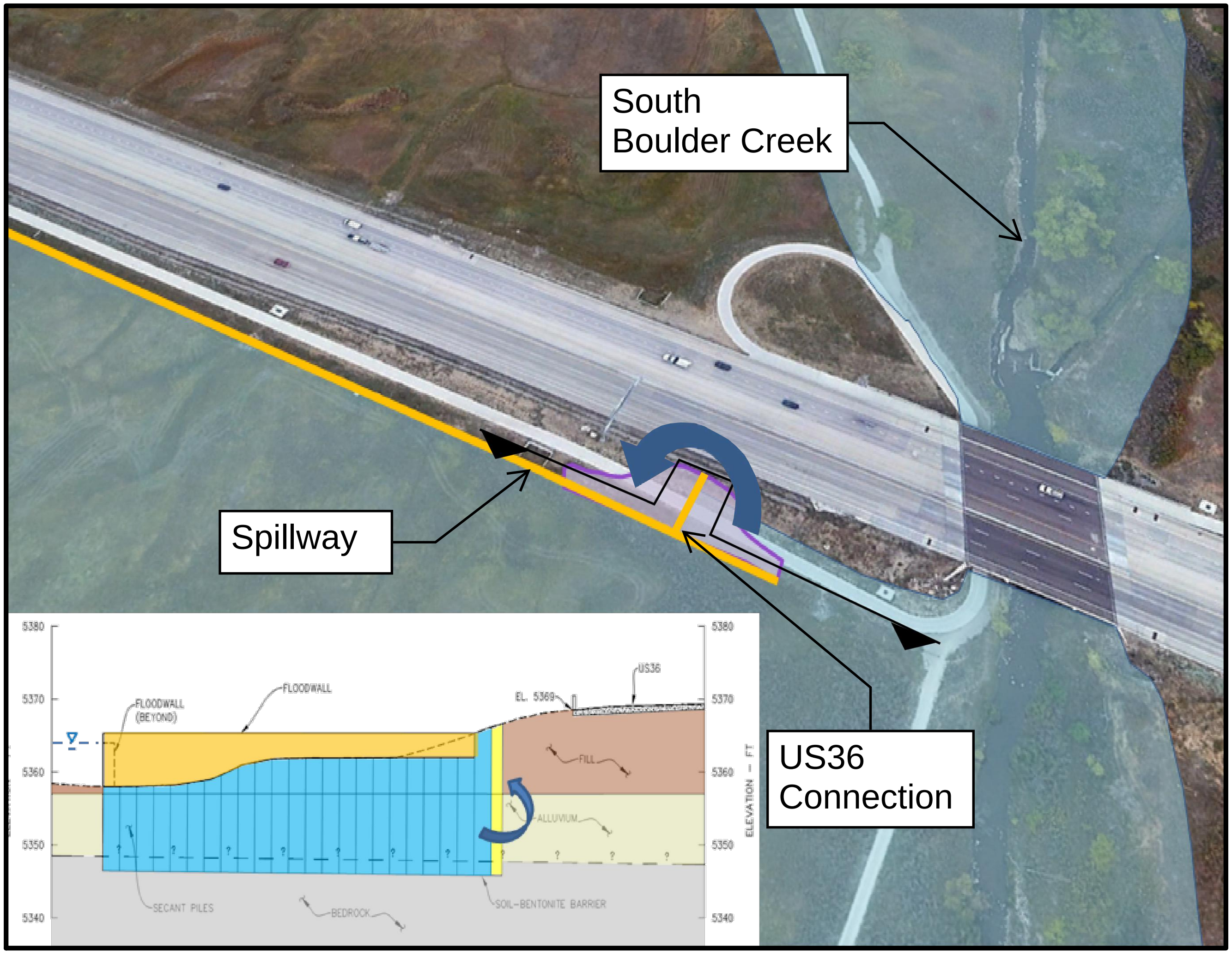
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

FLOW DIRECTION
FOR PFM #3

July 2021

Figure 3



NOTES:
1. PROFILE IS ORIENTED NNE/SSW THROUGH THE ABUTMENT CONNECTION AND IS LOOKING WSW.
2. BLUE ARROWS ON THE PLAN AND PROFILE SHOWS DIRECTION OF GROUNDWATER SEEPAGE.

REPRODUCE IN COLOR

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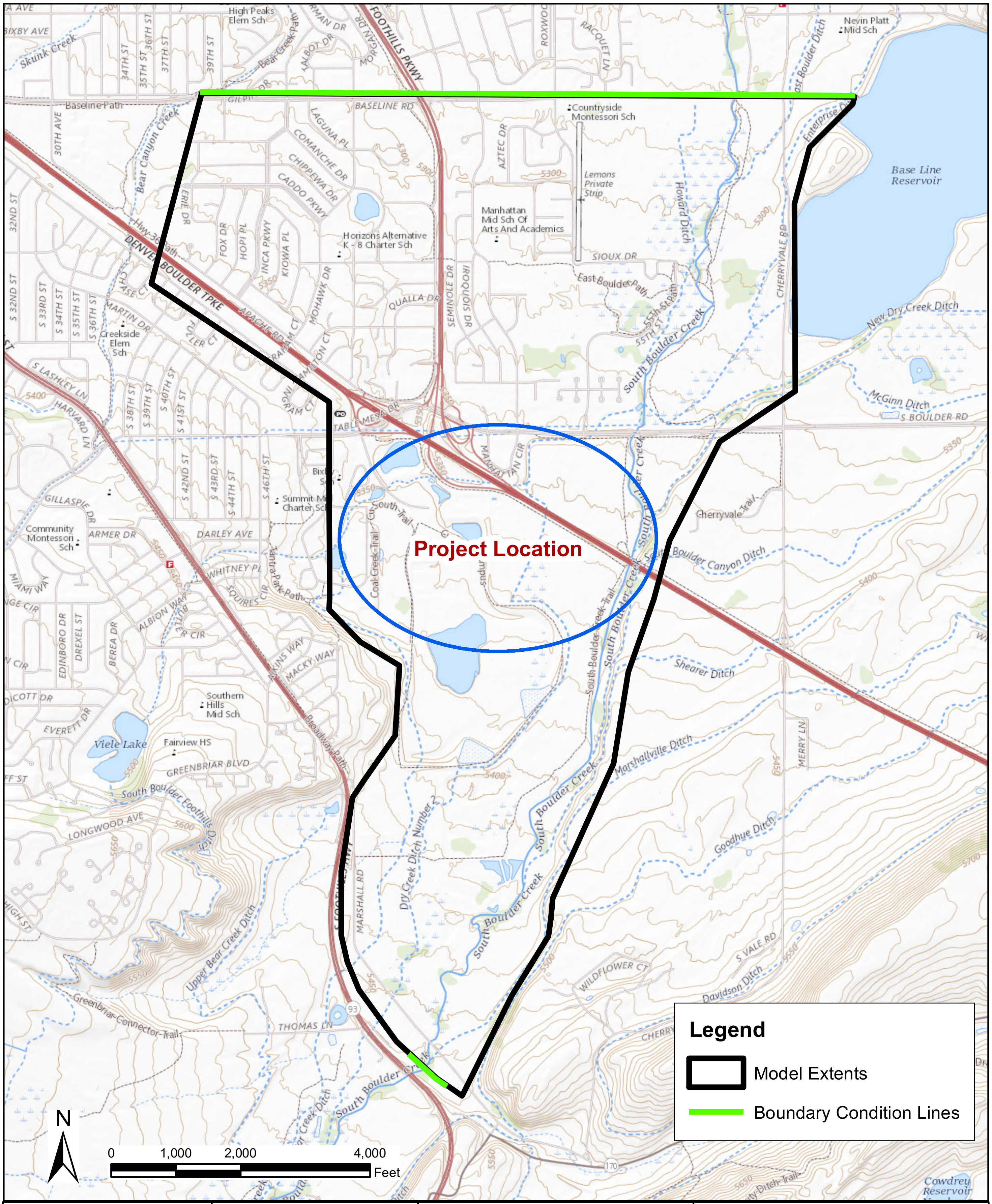
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

FLOW DIRECTION
FOR PFM #4

July 2021

Figure 4



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REPRODUCE IN COLOR

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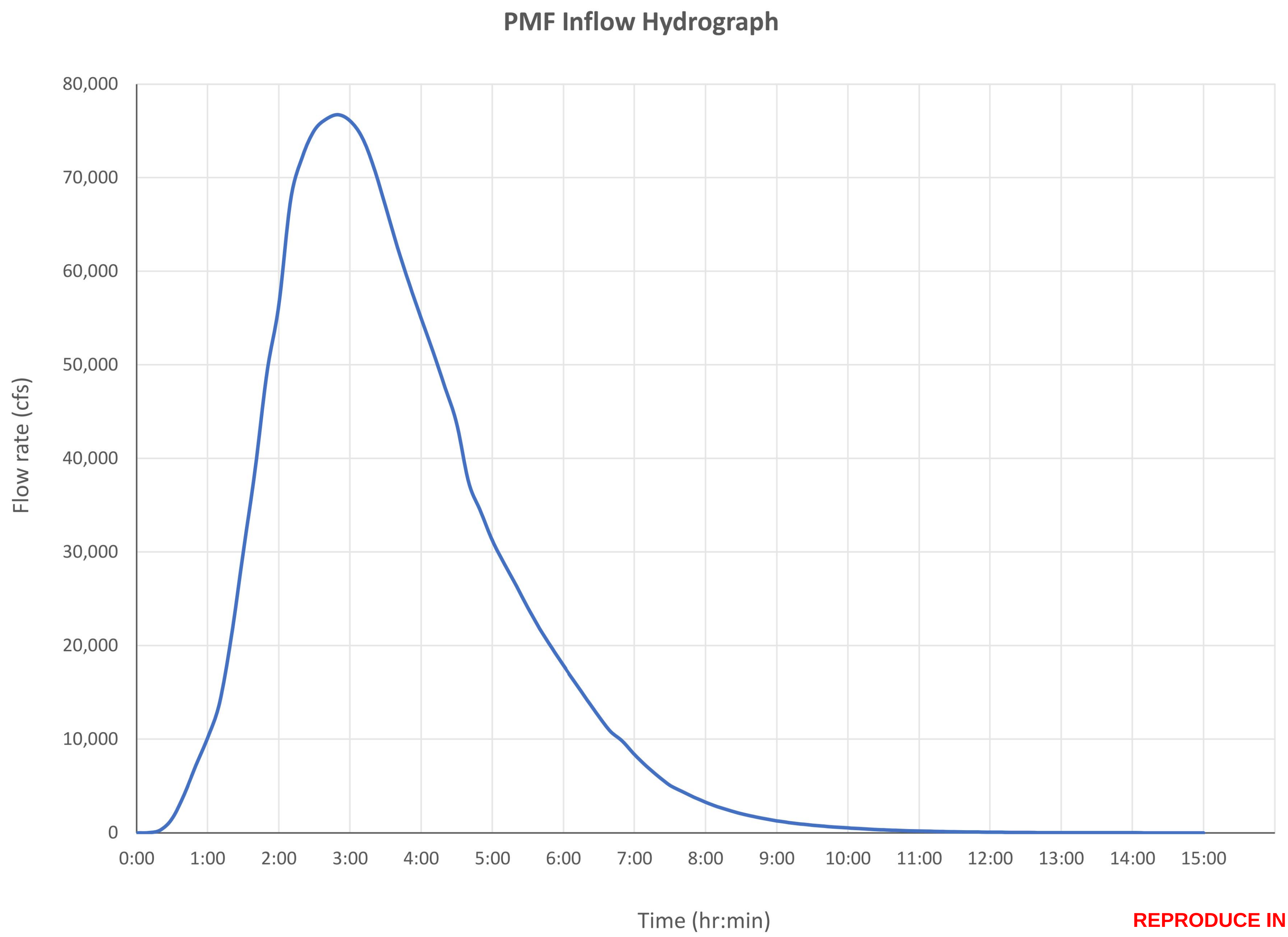
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

HEC-RAS
Model Extents

July 2021

Figure 5



REPRODUCE IN COLOR

NOTE:
1. HYDROGRAPH DATA CORRESPONDS TO THE 6-HOUR LOCAL STORM PMP EVENT OVER THE WATERSHED PORTION LOCATED DOWNSTREAM OF GROSS RESERVOIR.

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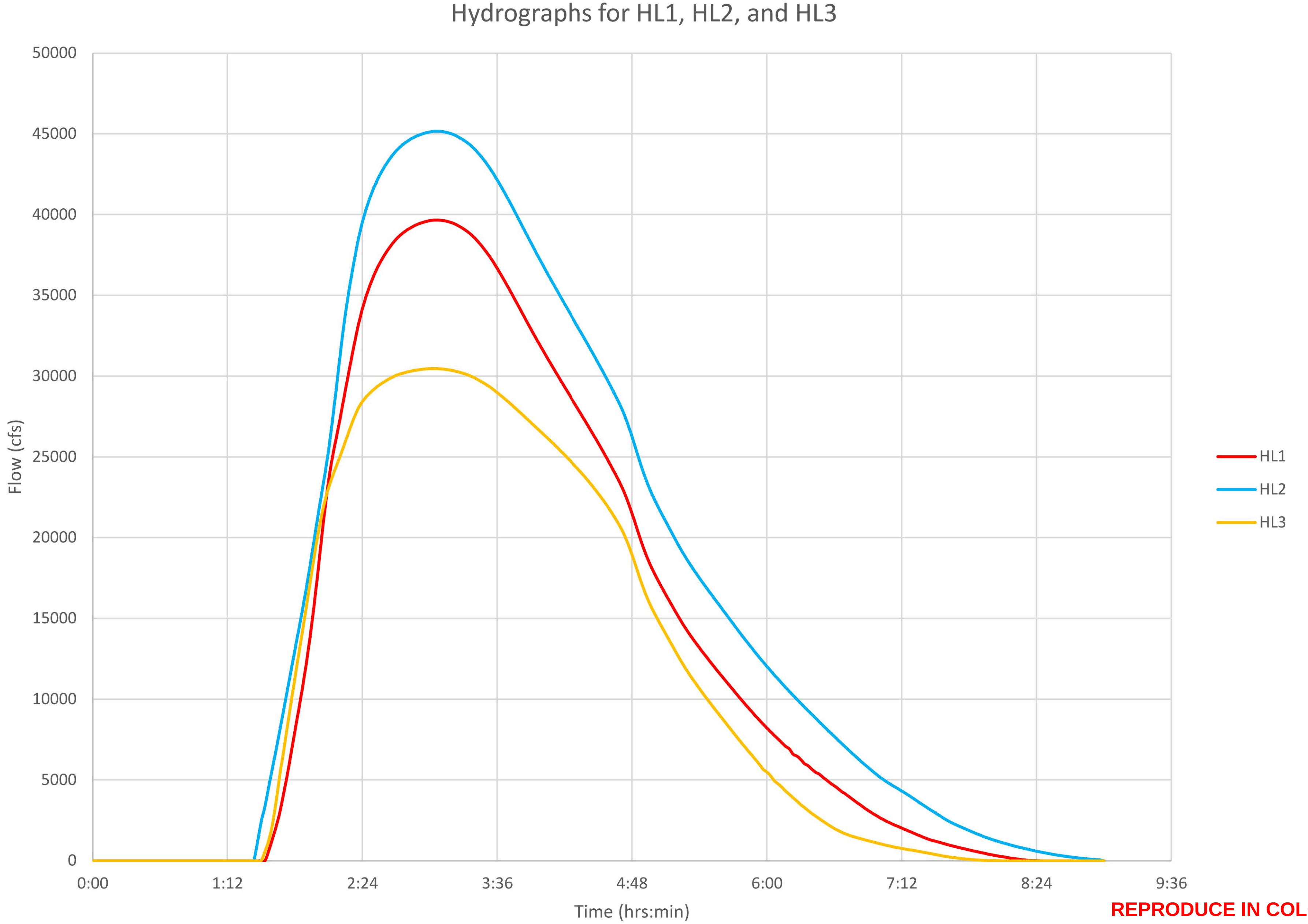
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

PMF INFLOW
HYDROGRAPH

July 2021

Figure 6



REPRODUCE IN COLOR

NOTE:
1. HYDROGRAPH DATA IS OBTAINED ALONG HL PROFILES FROM THE INFLOW HYDROGRAPH DATA IN HEC-RAS.

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SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

HYDROGRAPHS FOR HL1, HL2, AND HL3

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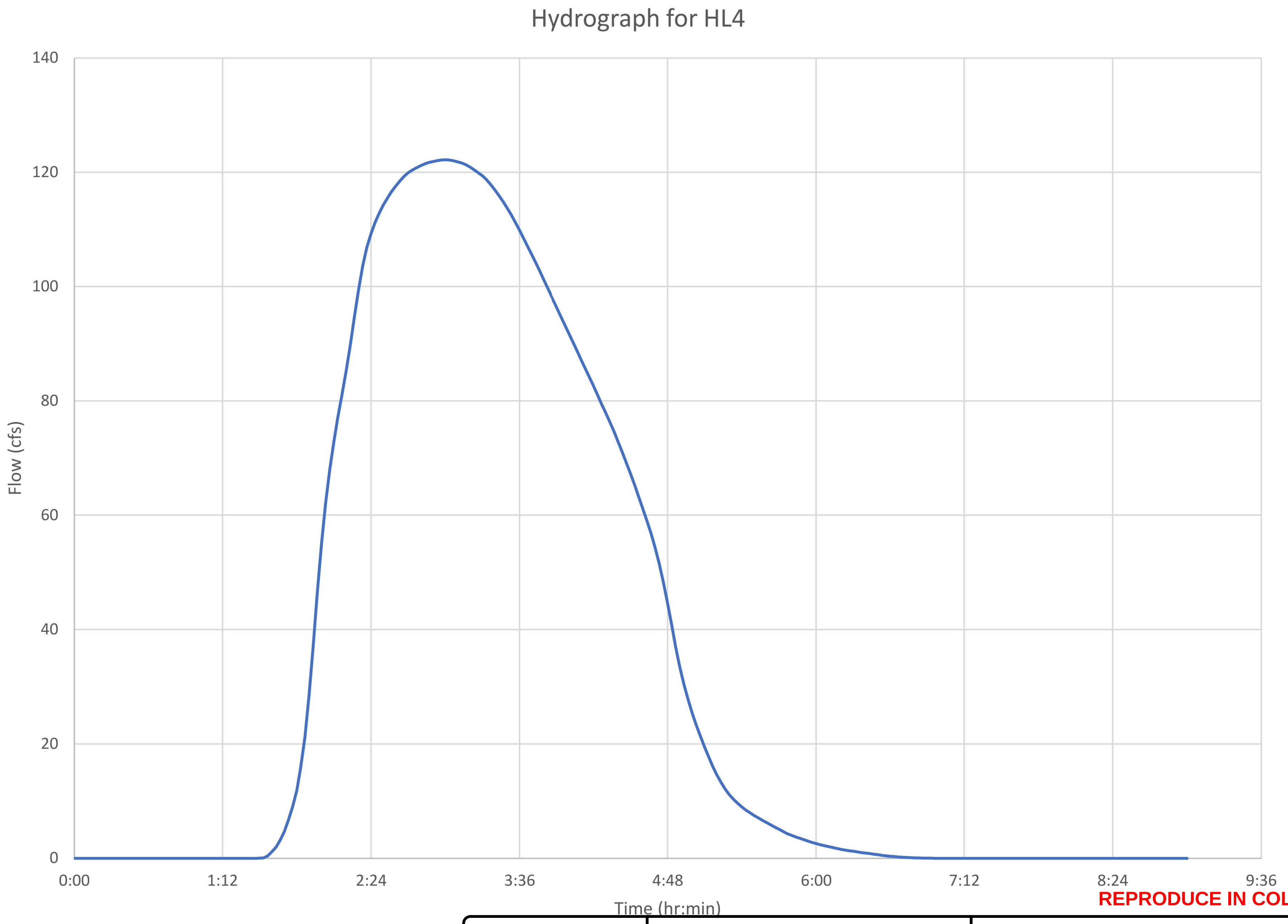
Figure 7



REPRODUCE IN COLOR

NOTE:
1. HYDROGRAPH DATA IS OBTAINED ALONG THE HL4 PROFILE FROM THE INFLOW HYDROGRAPH DATA IN HEC-RAS.

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REGIONAL DETENTION
CONCEPT DESIGN

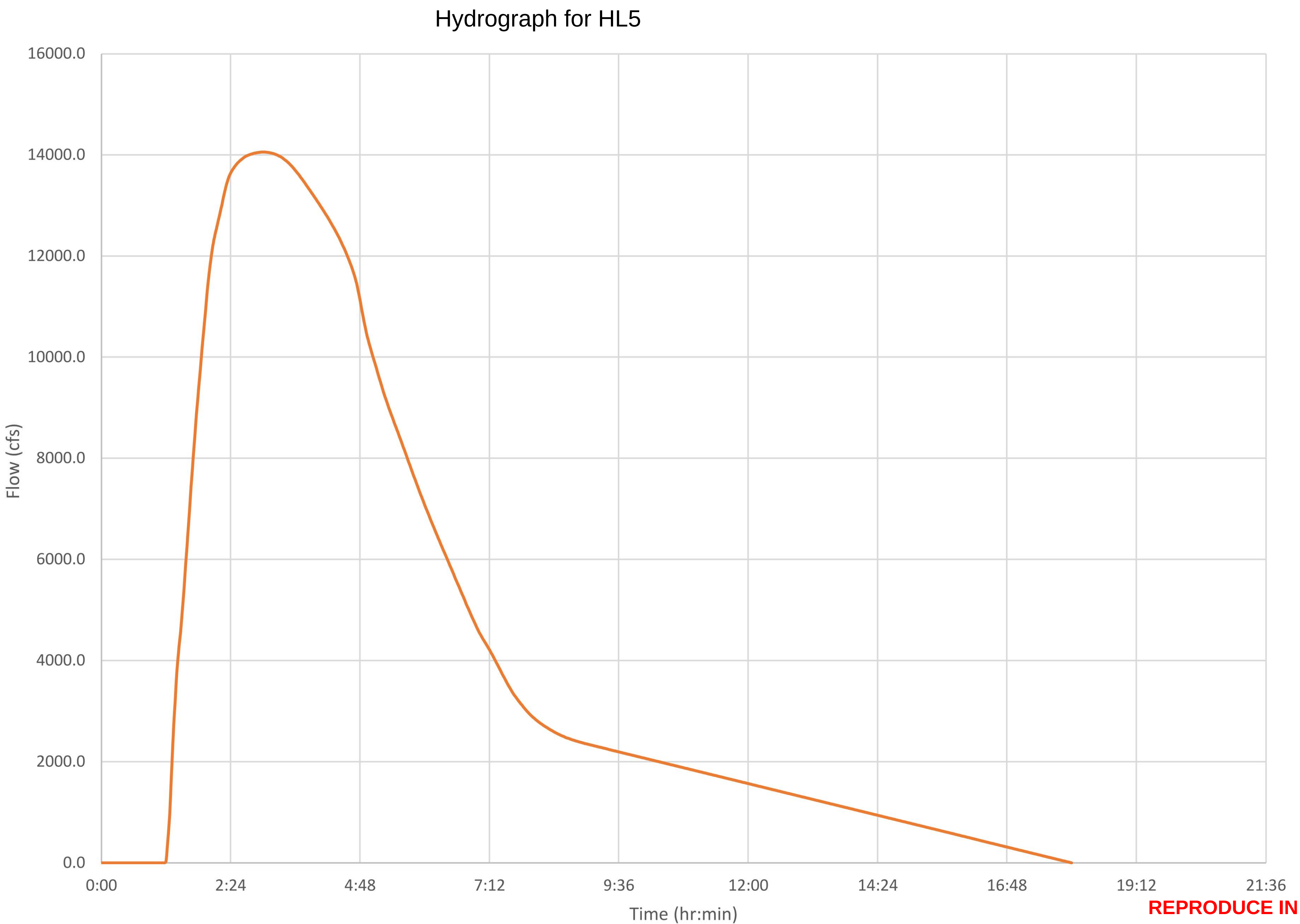
PROJECT NO. 16134

HYDROGRAPH FOR HL4

July 2021

Figure 8

P:\16134 – SOUTH BOULDER CREEK\CAD\FIGURES\2020-12-16_WINDAM_MEMO_FIGS_RECOVER.DWG 3/16/2022 12:11 PM



REPRODUCE IN COLOR

NOTE:
1. HYDROGRAPH DATA IS OBTAINED ALONG THE HL5 PROFILE FROM THE INFLOW HYDROGRAPH DATA IN HEC-RAS.

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SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

HYDROGRAPH FOR HL5
WINDAM MODEL

PROJECT NO. 16134

July 2021

Figure 9

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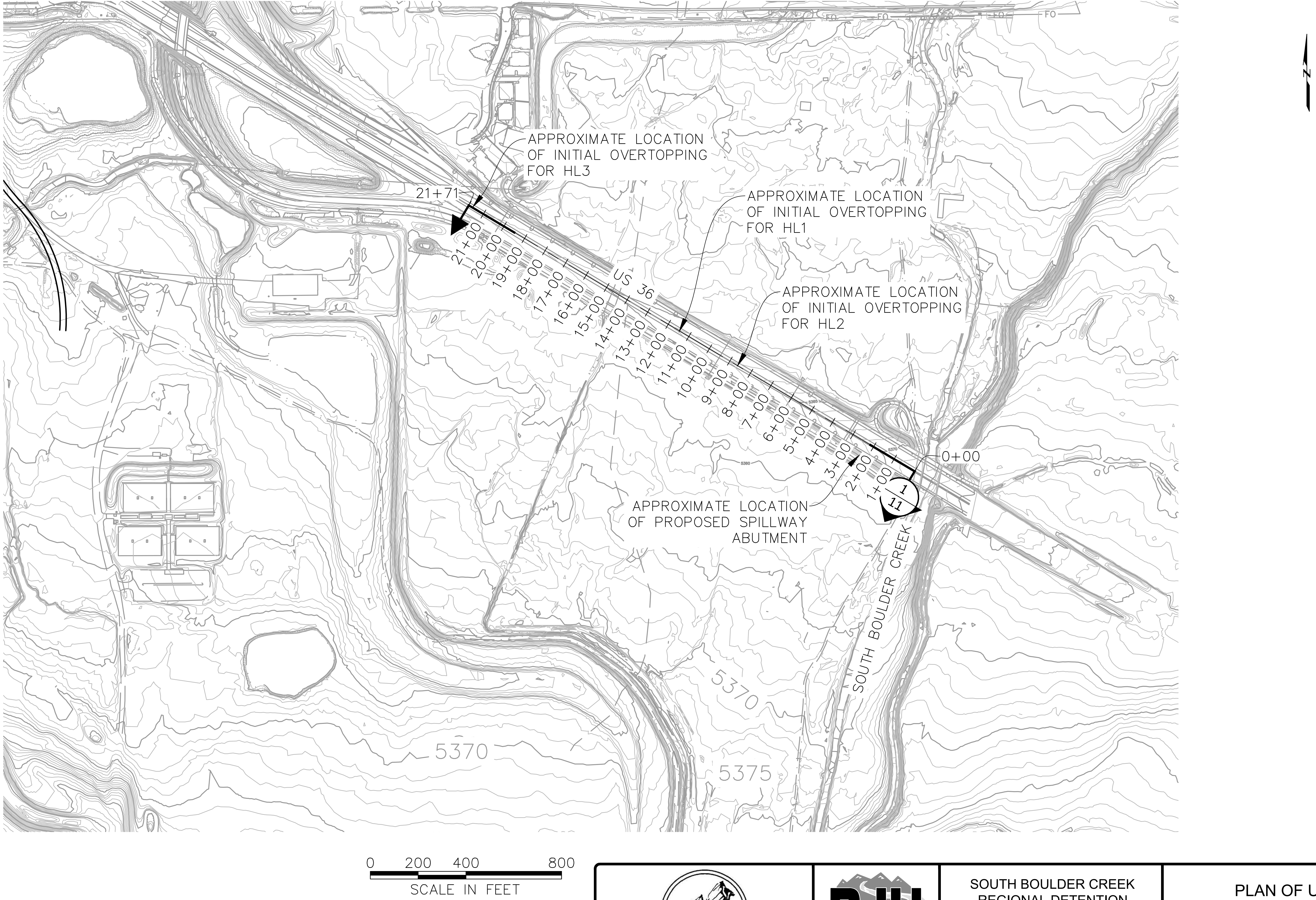

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CITY OF BOULDER

RJH CONSULTANTS, INC.

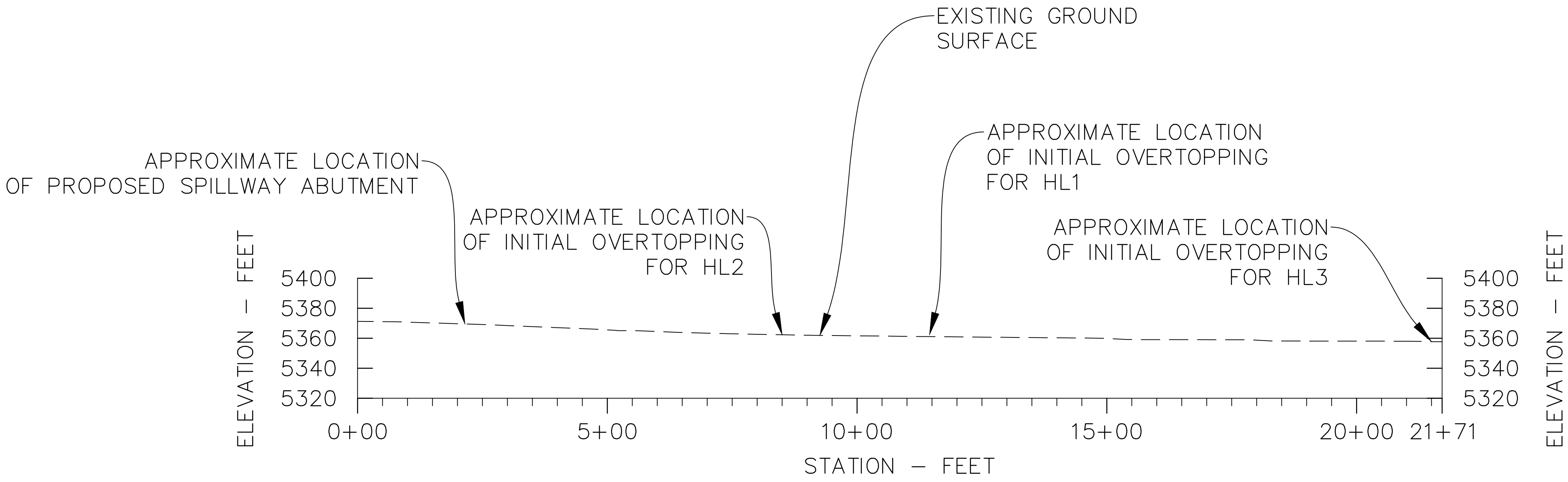
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PLAN OF US36
EMBANKMENT

PROJECT NO. 16134

July 2021

Figure 10



SECTION 1/10

NOTE:
1. US36 PROFILE IS LOOKING UPSTREAM TO MATCH THE CONVENTION USED BY THE WINDAM MODEL OUTPUTS.

0 50 100 200
VERTICAL SCALE IN FEET

0 150 300 600
HORIZONTAL SCALE IN FEET

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SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

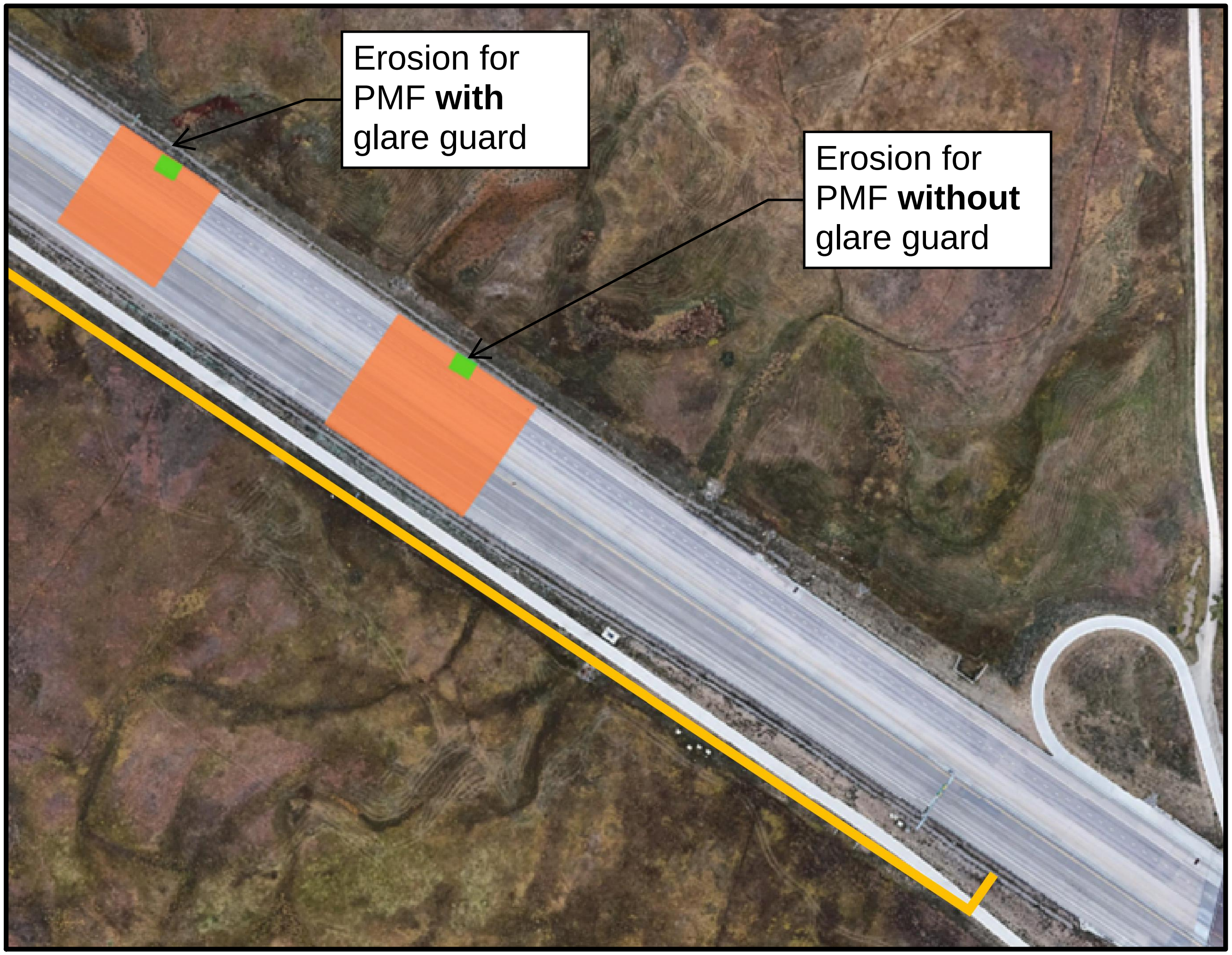
US36 EMBANKMENT PROFILE

PROJECT NO. 16134

July 2021

Figure 11

P:\16134 - SOUTH BOULDER CREEK\CAD\WORKING\2020-12-14_US36_PROFILE_WINDAM.DWG 2/25/2021 12:51 PM



🟩 Erosion extents for base case analysis

🟧 Erosion extents for sensitivity analysis with poorer material properties

REPRODUCE IN COLOR

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SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

EROSION FOR PFM
#1

July 2021

Figure 12

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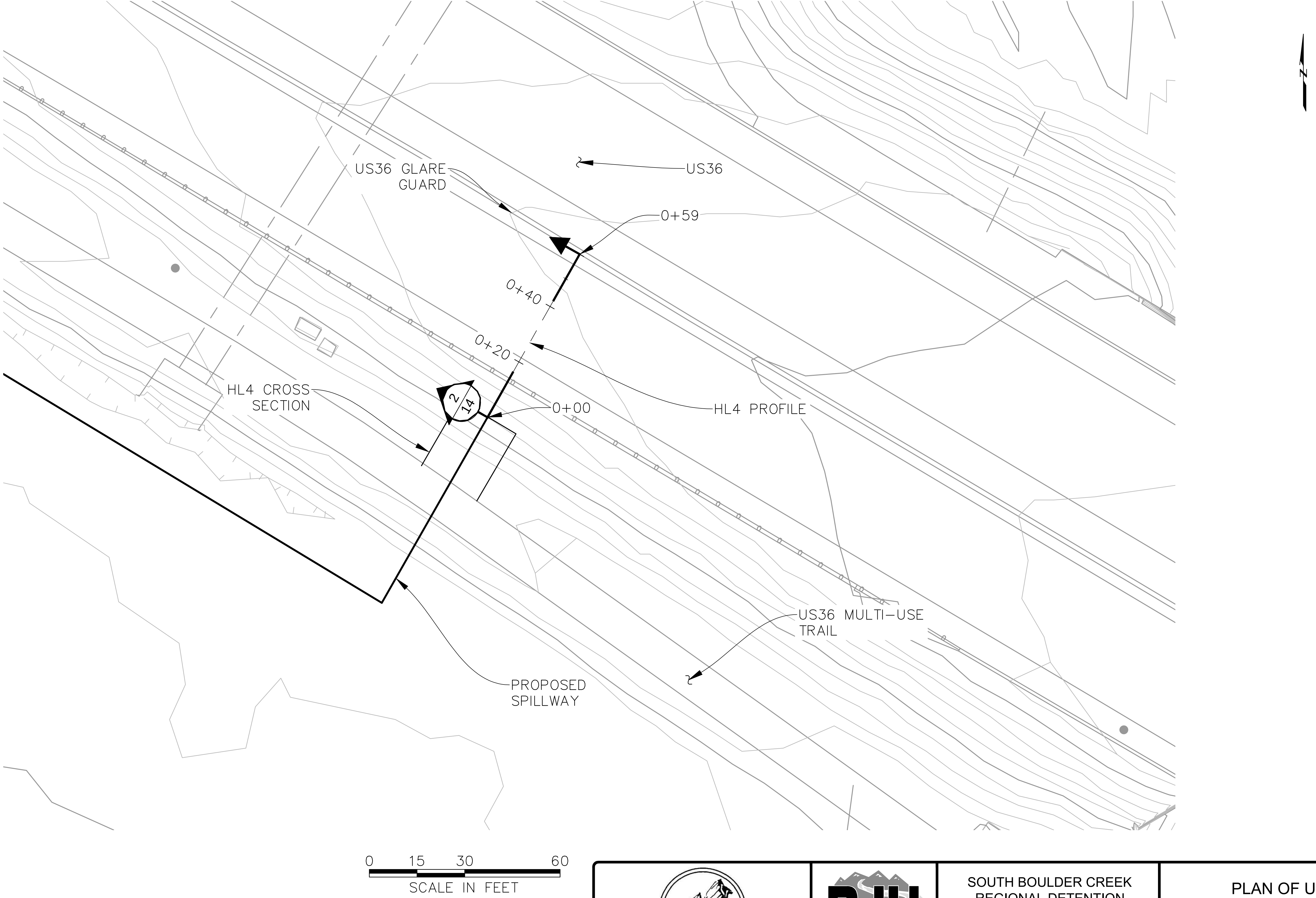

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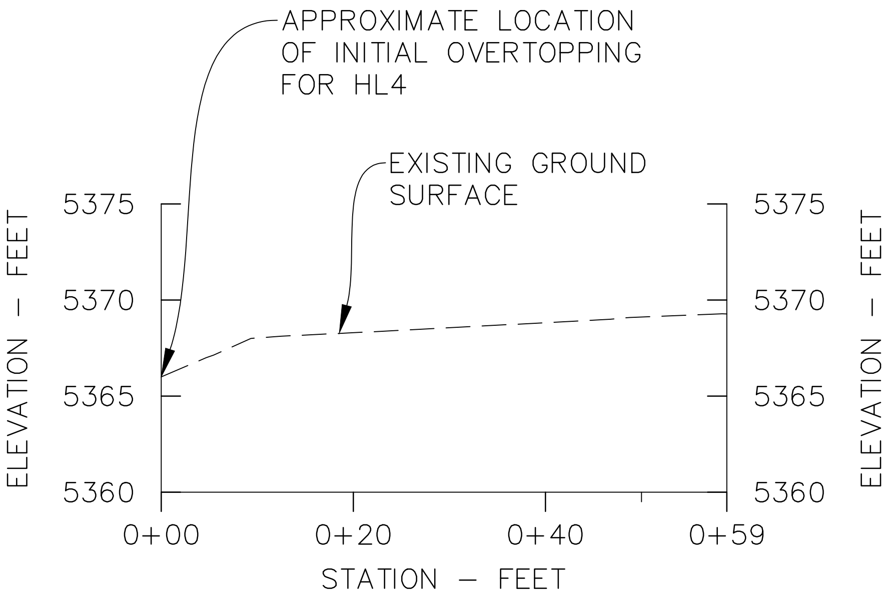
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

PLAN OF US36 EMBANKMENT - HL4

July 2021

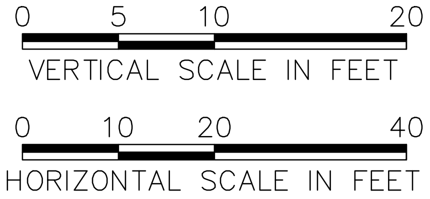
Figure 13


APPROXIMATE LOCATION OF INITIAL OVERTOPPING FOR HL4

EXISTING GROUND SURFACE

SECTION 2/13

NOTE:
1. PROFILE IS LOOKING UPSTREAM TO MATCH THE CONVENTION USED BY THE WINDAM MODEL OUTPUTS.



RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

US36 EMBANKMENT
PROFILE - HL4

PROJECT NO. 16134

July 2021

Figure 14

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REGIONAL DETENTION
CONCEPT DESIGN

EROSION FOR PFM #2

PROJECT NO. 16134

July 2021

Figure 15

RJH CONSULTANTS, INC.

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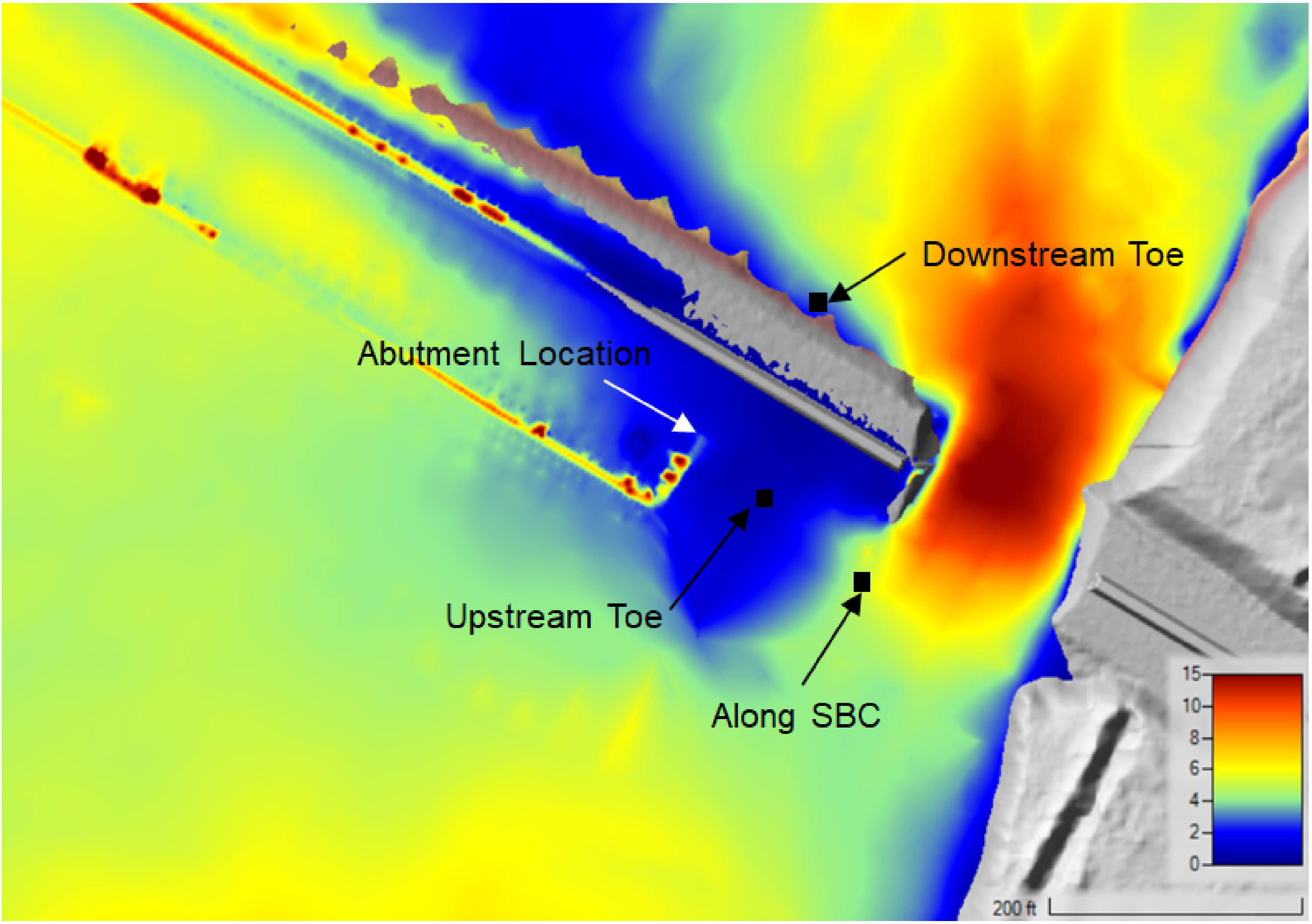

Notes:

1. Locations are approximate.

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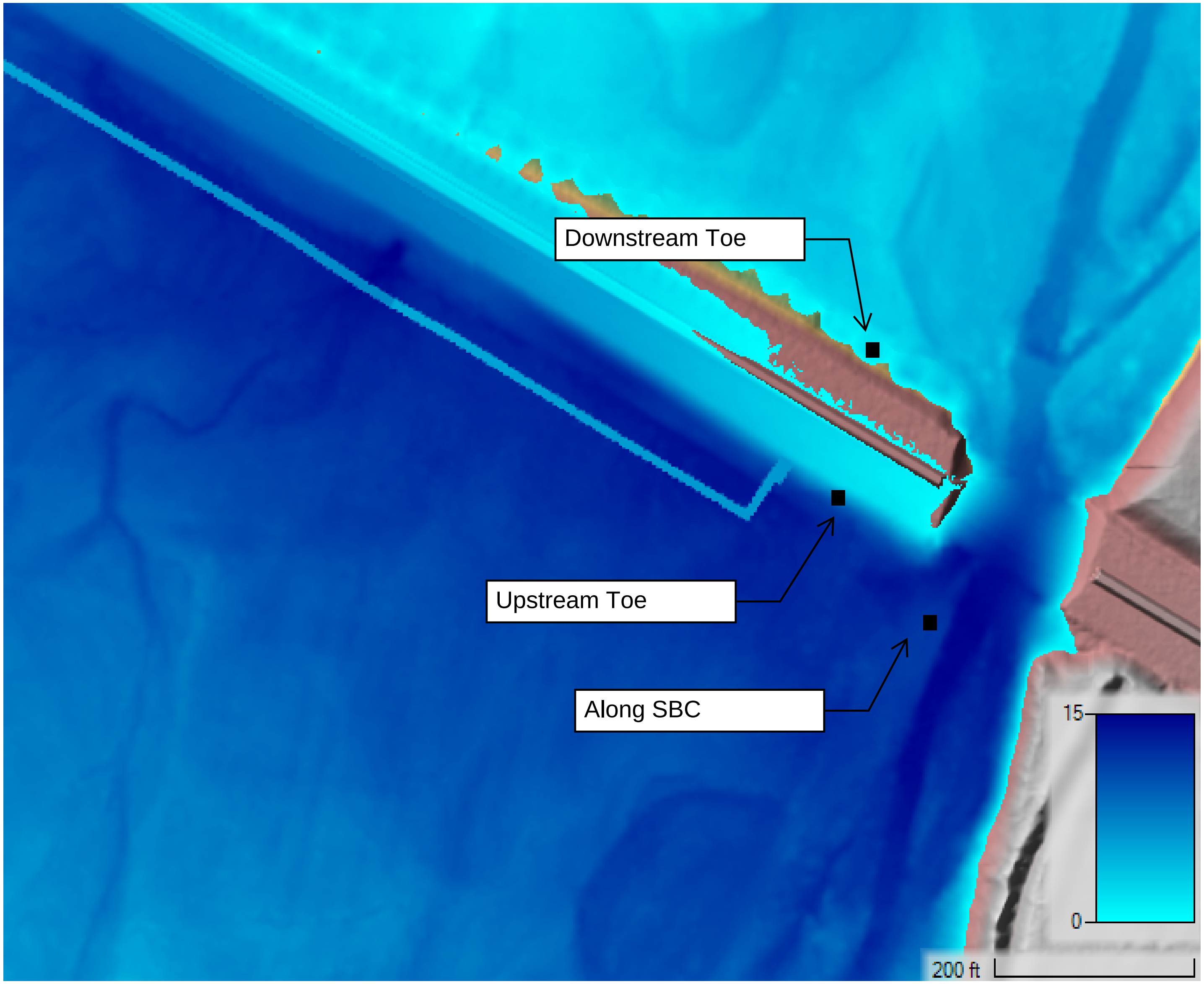
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

HEC-RAS MODEL WITH
MAXIMUM VELOCITIES

PROJECT NO. 16134

December 2021

Figure 16



Notes:
1. Locations are approximate.

REPRODUCE IN COLOR

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SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

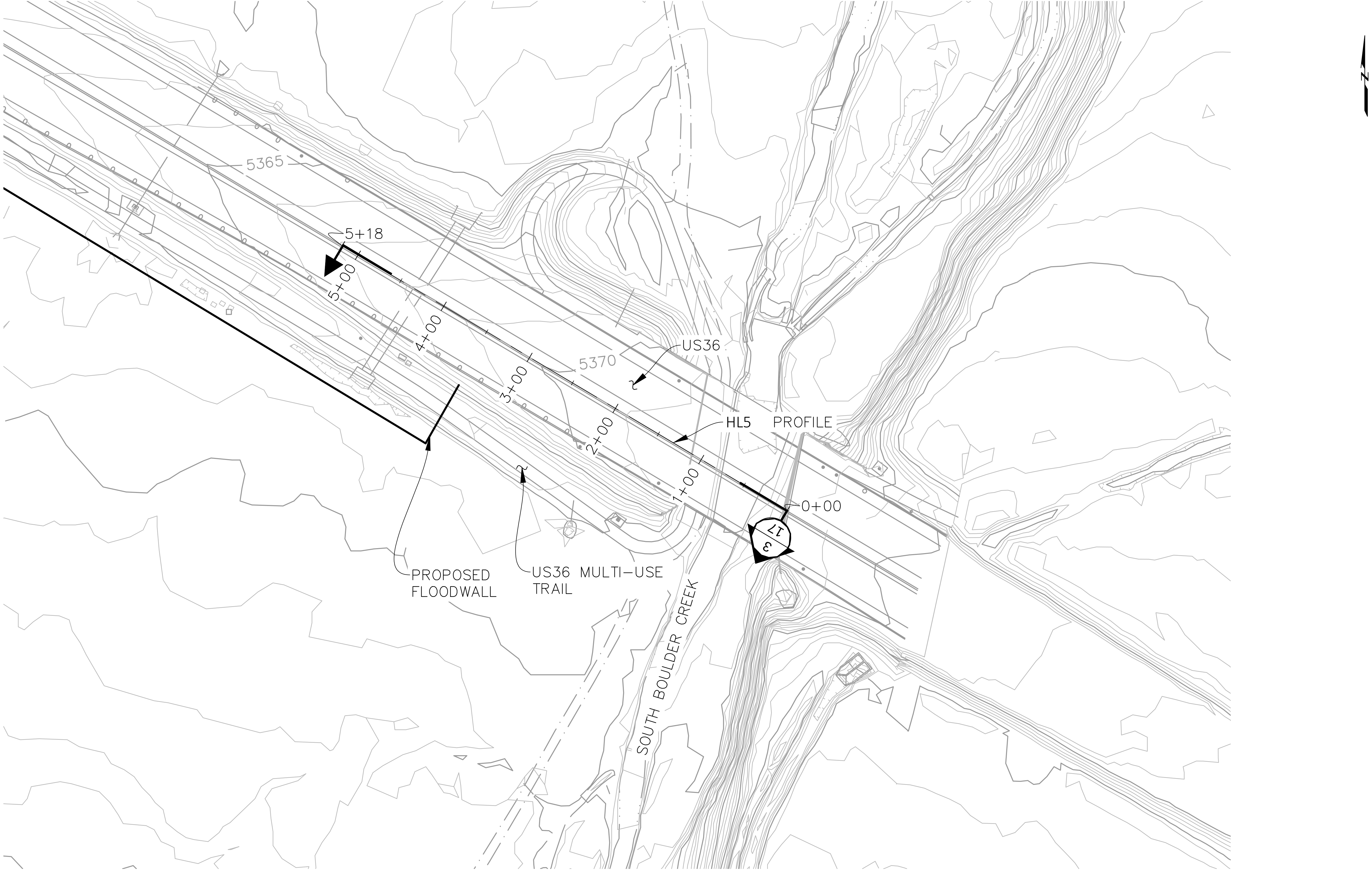
HEC-RAS MODEL WITH MAXIMUM DEPTHS

PROJECT NO. 16134

December 2021

Figure 17

P:\16134 – SOUTH BOULDER CREEK\CAD\FIGURES\2020-12-16_WINDAM_MEMO_FIGS.DWG 6/15/2021 10:31 AM



0 50 100 200
SCALE IN FEET

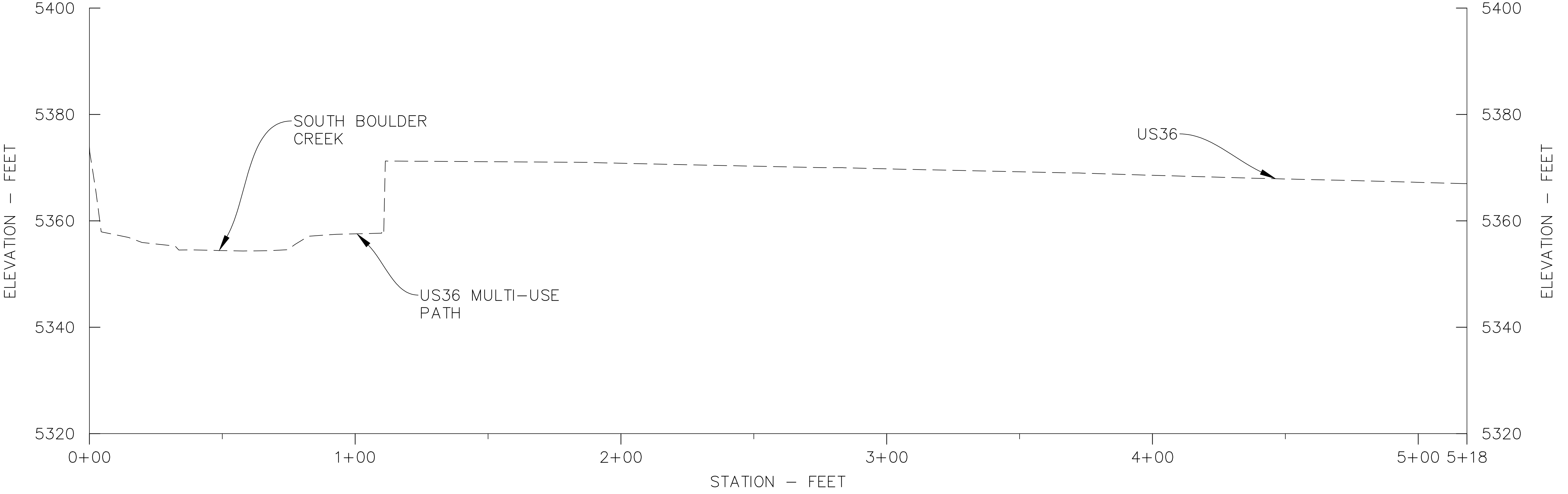
SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PLAN OF US36
EMBANKMENT - HL5

PROJECT NO. 16134

July 2021

Figure 18



SECTION 3/16

NOTE:
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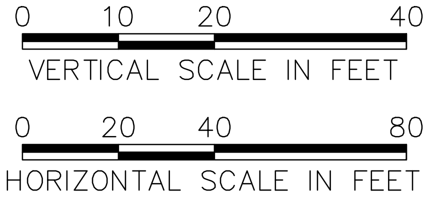

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REGIONAL DETENTION
CONCEPT DESIGN

US36 EMBANKMENT
PROFILE - HL5

PROJECT NO. 16134

July 2021

Figure 19

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SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

EROSION FOR PFM #3

PROJECT NO. 16134

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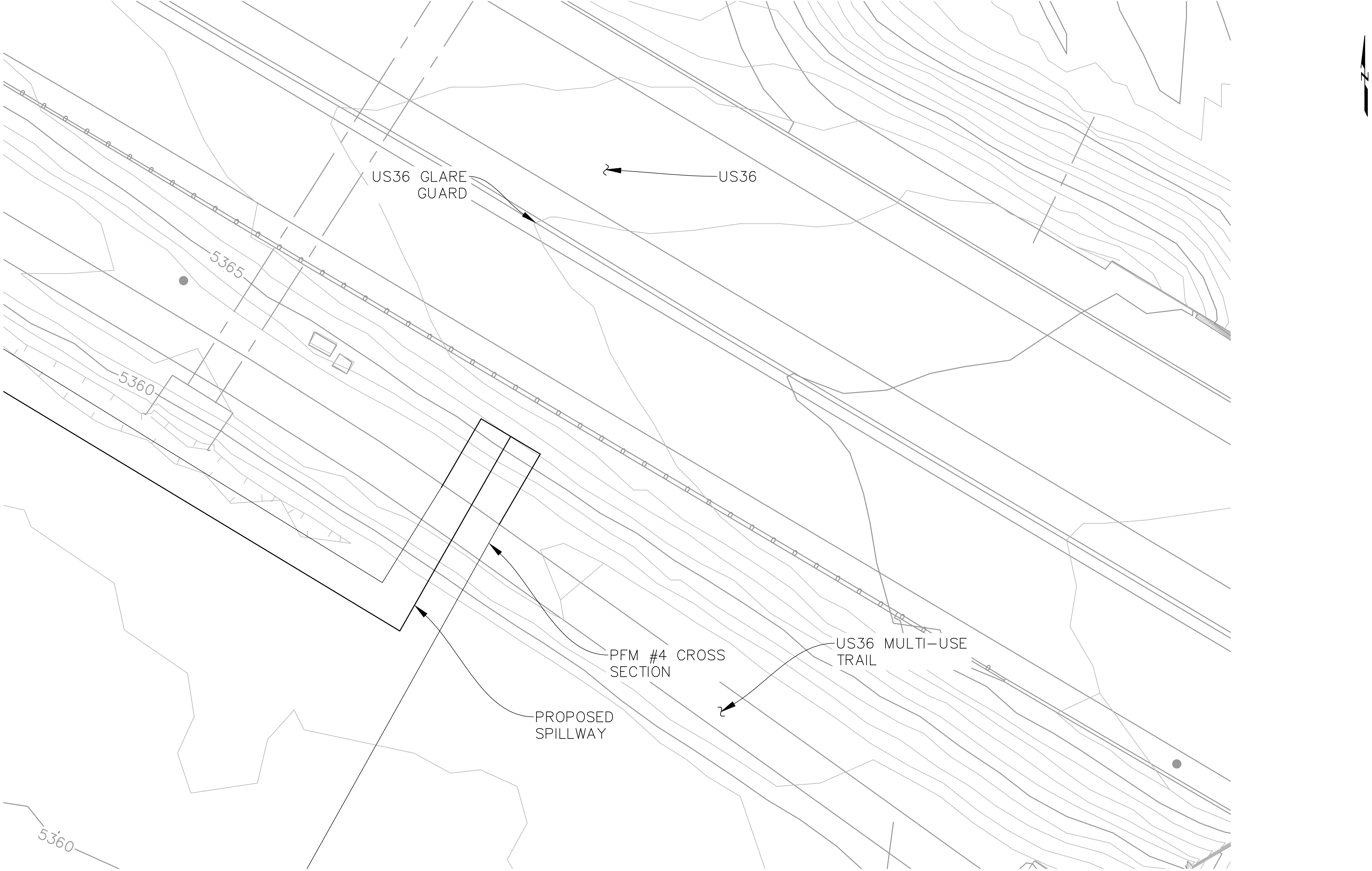
Figure 20

P:\16134 – SOUTH BOULDER CREEK\CAD\FIGURES\2020-12-16_WINDAM_MEMO_FIGS_RECOVER.DWG 7/15/2021 10:19 AM



NOTE:

1. THE NATURAL GROUND SURFACE IN THE SEEPAGE MODEL IS SIMPLIFIED AND DOES NOT MATCH THE EXISTING TOPOGRAPHY PRESENTED.

0 15 30 60
SCALE IN FEET

CITY OF BOULDER

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

PFM #4 CROSS
SECTION

July 2021

Figure 21

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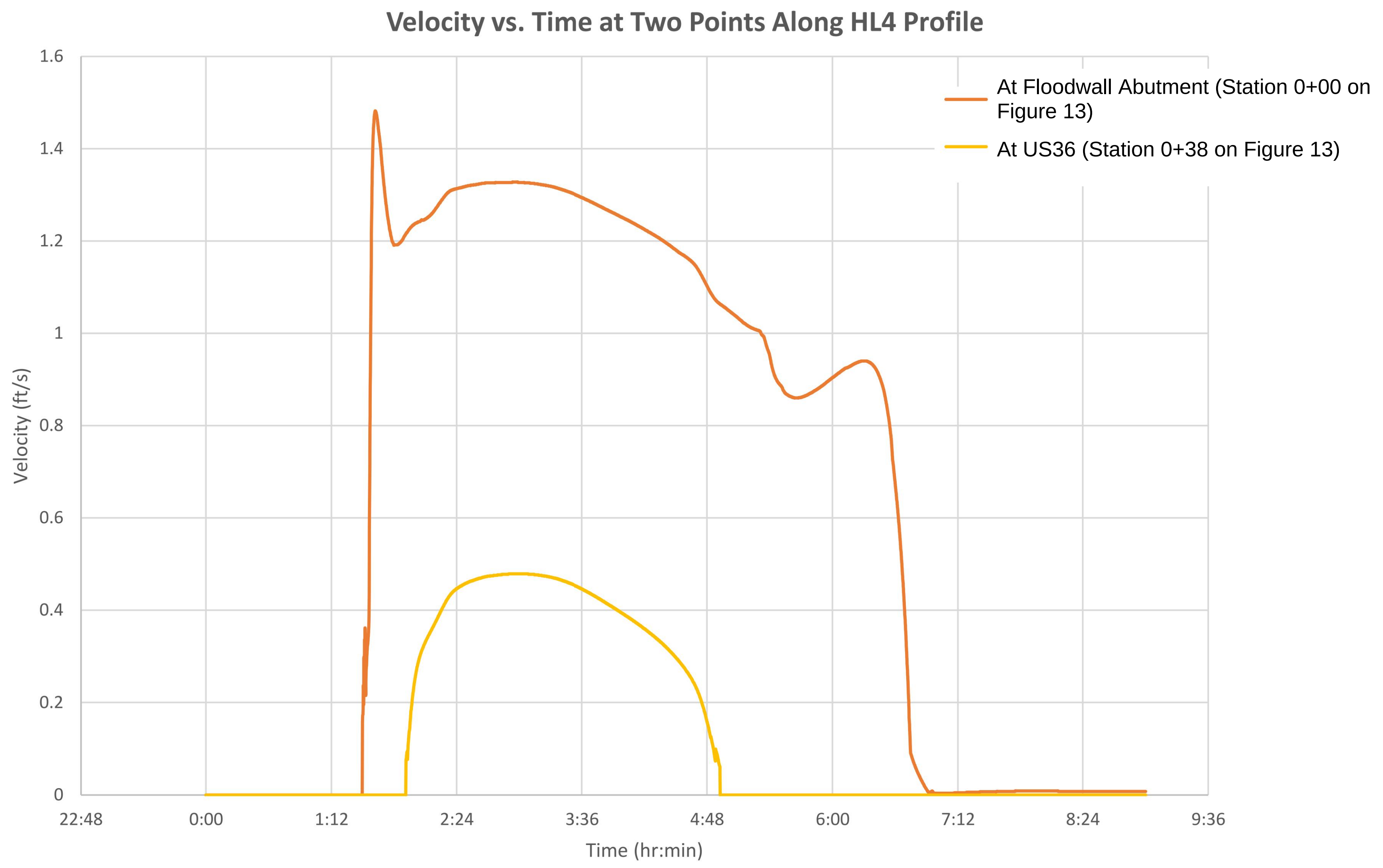

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REGIONAL DETENTION
CONCEPT DESIGN

PROJECT NO. 16134

VELOCITY VS. TIME - HL4

July 2021

Figure 22

APPENDIX E

GROUNDWATER MODELING OF PROPOSED CONDITIONS

ANALYSES MEMORANDUM

Project No. 16134

Performed: **3/31/2022** **By:** **ATMerook**
Checked: 7/7/2022 **By:** JNH
Approved: 7/12/2022 **By:** ABP

CLIENT: **City of Boulder**
SUBJECT: **South Boulder Creek Regional Detention Project**
MODFLOW-USG Groundwater Modeling of Proposed Conditions for Preliminary Design

---

1.0 - Introduction

RJH Consultants, Inc. (RJH) is using the computer program MODFLOW-USG (Panday et al., 2017) to simulate the groundwater system at the South Boulder Creek Regional Detention Facility Site (Site) for existing (i.e., baseline) and proposed (post-construction) conditions. The groundwater model previously developed and calibrated to baseline conditions (Baseline Model) is described in the *Baseline Groundwater Model Report* (Baseline Report) (RJH, 2021). The purpose of this memorandum is to document changes made to the Baseline Model to facilitate Proposed Conditions modeling and to assess simulated impacts of proposed facilities on the existing groundwater system. Preliminary Design recommendations are made where applicable.

Model simulations were run for one year and divided into 12 stress periods. Each stress period represented one month and was subdivided into 30 timesteps. One-year simulations were used instead of the two-year simulations used by the Baseline Model to shorten model run times while still simulating a complete annual cycle of wet and dry seasons.

2.0 - Pre-Project Model Conditions that were Modified from the Baseline Model

RJH used a slightly modified version of the Baseline Model as the benchmark with which to compare the impacts of the proposed facilities. The existing conditions model used for comparisons is herein referred to as the "Pre-Project" model to differentiate it from the previously developed Baseline Model.

A horizontal hydraulic conductivity of about 5.7E-4 feet per day (ft/d) (2E-7 centimeters per second (cm/s)) was used for both weathered and unweathered bedrock materials in the Pre-Project Model, which is about 2 to 3 orders of magnitude lower than the values used in the Baseline Model for ~~weathered and unweathered~~ bedrock, respectively. This modification to the Baseline Model material properties was based on additional field data collected during Phase II investigations, sensitivity analyses performed during calibration of the Baseline Model, and engineering judgement. Additional information about this modification is provided in Attachment A.

unweathered and weathered

All other material properties and inputs used for the Pre-Project model remained consistent with those summarized in the Baseline Report. The Pre-Project scenario used as the existing conditions benchmark for the preliminary design analyses is identical to the "Sens_KpK1" model presented in Appendix K.5 of the Baseline Report (RJH, 2021).

3.0 - Proposed Groundwater Barrier with No Groundwater Conveyance System

RJH developed a groundwater model that simulated proposed groundwater barriers with no groundwater conveyance system (herein referred to as No Conveyance System) to evaluate the effects of proposed seepage barriers and identify whether conveyance facilities were needed.

3.1 - Model Changes

Model components and inputs that were altered from the Pre-Project scenario are described in Section 3.1.

3.1.1 - Horizontal Flow Barriers (HFBs)

Low-permeable horizontal flow barriers (HFBs) were input to simulate proposed seepage barriers along the spillway secant pile wall alignment, the detention excavation perimeter barrier wall, and the embankment centerline barrier wall alignment. The HFB alignments are shown on Figure 3.1. Typical HFB input values include a wall thickness of 3 feet and a hydraulic conductivity of about 2.8E-4 ft/d (1E-7 cm/s). All HFBs were input through all soil layers (layers 1-3) and the top bedrock layer (layer 4). ✓

3.1.2 - Detention Excavation Property Zones and Boundary Conditions

Model topography was not altered prior to modeling the No Conveyance System scenario because the detention excavation preliminary grading plan is still in development. The hydraulic conductivity of soil within the detention excavation was increased to 500 ft/d, which ✓ is identical to the hydraulic conductivity value previously used in the Baseline Model to simulate open bodies of surface water. This modeling approach accounts for the decreased flow resistance that will exist in the detention excavation post-construction and is shown on Figure 3.2.

A new drain boundary condition was added along the interior of the detention excavation HFB to remove water from the model and simulate how seepage that enters the detention excavation will flow out through the uncontrolled outlet works. The alignment of the added drain is shown on Figure 3.3. Input values include an invert elevation of 5,343 feet, which is ✓ equal to the proposed invert elevation of the outlet works conduit, and a hydraulic conductivity of 5,000 ft/d which is sufficiently high to simulate removal of seepage inflows ✓ through the uncontrolled outlet works. We do not expect that model results will be highly sensitive to the hydraulic conductivity of this drain because it is located entirely interior of the detention excavation HFB boundary.

Recharge inputs within the detention excavation were removed from the model to avoid adding additional water to this area. In our opinion, this is reasonable because inflows to the detention excavation area will be isolated from the surrounding groundwater system by the perimeter barrier wall.

Large inflows into the detention excavation will flow out through the outlet works. These flows were not included in the model, which in our opinion is appropriate because the

MODFLOW-USG model is not intended to simulate surface water flows throughout the Site following precipitation events.

3.2 - Simulated Head Results and Preliminary Design Implications

Simulated groundwater elevation contours for the No Conveyance System scenario were compared against the simulated Pre-Project groundwater table for a representative winter month (i.e., November) and a representative summer month (i.e., June). Color-coded maps were developed to illustrate the magnitude of predicted groundwater level change between the Pre-Project conditions and the No Conveyance System scenario.

Simulated head results for the representative winter and summer months are presented on Figure 3.4 and Figure 3.5, respectively. Blue shaded areas represent areas of simulated groundwater mounding (post-Project groundwater level is higher than pre-Project level) and red shaded areas represent areas of simulated groundwater decline (post-Project groundwater level is lower than pre-Project level). Darker colors represent greater mounding or decline. Areas that are within +/-0.10 feet of Pre-Project groundwater levels are not shaded to improve clarity. The numerical magnitudes of mounding or decline are also shown by callouts at selected locations on the plan figures. ✓

Based on judgement and the inherent variability of the hydrogeologic system, we anticipated that mitigation would be required if predicted groundwater levels in sensitive areas away from Project facilities were generally not within +/- 0.5 feet of Pre-Project levels.

On the west side of the Site, simulated mounding west of the embankment cutoff wall HFB was minor (less than 2.8 feet) and generally limited to University of Colorado (CU) property. Simulated decline east of the dam alignment was less than 0.8 feet and was generally limited to areas near the embankment. Although these results generally appear acceptable in our opinion, groundwater mounding along the west edge of the Site will be influenced by the embankment toe drain. The simulated effects of this drain are presented in Section 4. ✓

Along the spillway, up to approximately 9.4 feet of groundwater mounding was simulated south of the spillway secant pile wall HFB during the representative winter month (Figure 3.4) and up to approximately 9.1 feet of groundwater decline was simulated north of the spillway secant pile wall HFB during the representative summer month (Figure 3.5). A relatively large area of groundwater decline that exceeds 0.10 feet below Pre-Project levels is predicted to extend downstream to about Baseline Road. These predicted groundwater effects are not acceptable and demonstrate that some sort of conveyance system is required at the spillway to maintain Pre-Project groundwater conditions. ✓

4.0 - Proposed Conditions with Groundwater Conveyance System

RJH developed a groundwater model that simulated the proposed groundwater conveyance system (herein referred to as Proposed Conditions) to support preliminary design of conveyance system components.

4.1 - Model Changes

Model components and inputs that were altered from the No Conveyance System scenario are described in Section 4.1.

4.1.1 - Groundwater Conveyance System Hydraulic Conductivity Zones

Two new hydraulic conductivity zones were added to the model to simulate the groundwater conveyance system along the spillway. The simulated components of the groundwater conveyance system were:

1. Collector and distributor pipes. These pipes are anticipated to consist of slotted pipes within aggregate-filled trenches upstream and downstream of the spillway alignment.
2. Connector pipes. These pipes are anticipated to be solid pipes that penetrate through the secant pile wall to connect the collector and distributor pipes.

The groundwater conveyance system pipes were simulated using high-permeable cells within the model grid as shown on Figure 4.1. ✓ Hydrostratigraphic Unit (HSU) cell assignments for the connector cells, which are used by MODFLOW-USG for extraction of simulated flow results, are also presented on Figure 4.1. Both collector/distributor pipes and ✓ connector pipes were input in layers 2 and 3 (soil layers) to intercept the groundwater table and provide adequate conveyance capacity.

Input values for collection and distribution cells included a horizontal hydraulic conductivity value of 5,000 ft/d and a vertical hydraulic conductivity value of 1,000 ft/d. ✓ Input values for connector cells include a hydraulic conductivity value in the longitudinal (north/south) direction of 50,000 ft/d and a hydraulic conductivity value of 1 ft/d in the vertical and lateral ✓ (east/west) directions. ✓ A trench hydraulic conductivity value of 5,000 ft/d was selected based on typical published values for clean gravel (Cedergren, 1989; Reclamation, 2014), and is anticipated to be less than the capacity of the open connector pipes (conservatively considered to be 50,000 ft/d). Hydraulic conductivity for connector cells was defined to limit vertical and lateral flow interactions with the surrounding groundwater along the length of the solid pipe.

Collector and distributor cells were input as discrete segments about 300 feet in length with an approximately 300-foot gap between each segment to prevent a continuous preferential flowpath parallel with the spillway, and to promote flow across the HFB. Connector cells were input approximately equidistant (about 600-foot spacing) along the spillway alignment.

The system will likely be designed and constructed with continuous collector/distributor trenches parallel to the spillway secant pile wall with periodic gates and backfill plugs to replicate the performance of the simulated segmented collector and distribution pipes.

4.1.2 - Western Embankment Toe Drain Boundary Condition

A new drain boundary condition was added along the downstream (western) toe of the proposed embankment dam slightly above the simulated seasonally-high Pre-Project groundwater table. The alignment of the added drain is shown on Figure 4.2. A hydraulic conductivity of 5,000 ft/d ✓ was selected for the drain, which was consistent with the hydraulic conductivity value assigned to the collector/distributor cells along the spillway. This drain boundary condition is able to remove water from the model, and simulates how groundwater that mounds along the west side of the Site could be collected by the embankment toe drain pipe.

The toe drain pipe that will be installed along the northern portion of the embankment was not simulated in the model. This is not anticipated to affect model results because mounding downstream of the embankment is only predicted to occur along the western portion of the embankment (Figures 3.4 and 3.5). ✓

4.2 - Simulated Head Results and Preliminary Design Implications

Simulated groundwater contours for the Proposed Conditions scenario were processed using the same techniques and are presented in the same manner as described in Section 3.2. Simulated head results for the proposed conveyance system during representative winter and summer months are presented on Figure 4.3 and Figure 4.4, respectively.

4.2.1 - West Side of the Site

At the west side of the Site, up to approximately 1.7 feet of groundwater mounding was simulated west of the embankment centerline HFB during the representative summer month while up to approximately 0.9 feet of groundwater decline was simulated east of the embankment centerline HFB for both the representative winter and summer months. These simulated areas of greatest mounding and decline generally occur on CU property near the embankment alignment. The simulated toe drain has significantly reduced the extent and magnitude of groundwater mounding shown on Figure 3.4 and Figure 3.5 near the CU property boundary; however, the simulated drain produced negligible changes to the simulated groundwater decline east of the embankment dam centerline barrier wall, which is expected. As shown on Figure 4.5, the simulated toe drain generally limited groundwater elevations along the boundary profile close to the seasonally-high groundwater table simulated for Pre-Project conditions.

The drain pipe will be installed slightly above the historical high groundwater level. This will inevitably result in some minor groundwater mounding during periods where the existing groundwater table is seasonally low; however, this mounding is not anticipated to exceed the seasonal high groundwater levels. In our opinion, this design solution will adequately mitigate groundwater effects on the west side of the Site because:

1. Groundwater levels downstream of the dam will be maintained at or below the seasonally-high levels.
2. Any groundwater effects downstream of the dam are predicted to remain on CU property.
3. Based on publicly available information, the residences downstream of the dam do not appear to have basements and are at very low risk of being impacted by the proposed groundwater effects.
4. The minor groundwater effects that occur upstream of the dam alignment are not anticipated to significantly affect CU's development of this area.

4.2.2 - Spillway

Up to approximately 0.9 foot of groundwater mounding was simulated south of the spillway HFB during the representative summer month while up to approximately 1.7 feet of groundwater decline was simulated north of the spillway for the representative summer month. Simulated areas of greatest mounding and decline generally occur within the U.S. Highway 36 (US36) Right-of-Way (ROW) near the alignments of the proposed spillway apron and pedestrian trail. The simulated conveyance system has nearly eliminated the groundwater mounding shown on Figure 3.4 and Figure 3.5 upstream of the spillway HFB, and a slight (approximately 0.5-foot) groundwater decline is currently predicted to occur upstream of the spillway.

The simulated conveyance system also greatly reduced the large area of groundwater decline that extended north of South Boulder Road as shown on Figure 3.4 and Figure 3.5. The simulated conveyance system is still predicted to produce a small area of groundwater decline north of proposed facilities as shown on Figure 4.3 and Figure 4.4. This area is generally located between the embankment and the Table Mesa Regional Transportation District (RTD) Park-N-Ride. Groundwater decline within this area is predicted to be up to about 2.6 feet near the embankment and reduce to about 0.6 feet near the north side of US36. Land use within this area generally consists of highway interchanges within Colorado Department of Transportation (CDOT) ROW, and groundwater level changes within this area are not anticipated to cause adverse effects. ✓ ✓

The predicted groundwater elevations after construction of the Project are generally within +/-0.5 feet of the predicted Pre-Project groundwater elevations except for localized areas immediately adjacent to the spillway wall. Downstream groundwater effects predominantly occur within the US36 ROW. In our opinion, the observed spatial distributions and magnitudes of groundwater mounding and decline are acceptable for the purposes of preliminary design. Adjusting gates to reduce the flow capacity through the system could likely reduce the size and magnitude of the groundwater decline that is currently being simulated upstream of the spillway. A key location for providing operational flexibility appears to be in the northwest corner of the Open Space & Mountain Parks (OSMP) South field, where the model results are predicted to transition from decline to mounding. ✓

4.3 - Simulated Flow Results and Preliminary Design Implications

Simulated flows were extracted from the model results using either assigned HSU cells or boundary condition cells. Simulated flows are presented for the middle and/or last timestep in each reported model stress period.

4.3.1 - Simulated Flows Beneath US36

Simulated flows beneath US36 were extracted from the Pre-Project and Proposed Conditions scenarios using the same designated US36 HSU zones 6 through 9 that were reported within the Baseline Report (RJH, 2021) and are shown on Figure 4.6. ✓

The simulated incremental and cumulative flows through soil beneath US36 for the Pre-Project and Proposed Conditions scenarios are presented on Figure 4.7. Figure 4.7 shows the total flow combined from HSU zones 6 through 9, and the flow information for each individual HSU is presented in Attachment B. As shown in Attachment B, the proposed conveyance system is predicted to result in some minor redistribution of flows beneath US36. Flows through HSUs 6 through 9 are predicted to decrease by 0.2 to 14 percent on average compared to the Pre-Project scenario. However, as shown on Figure 4.7, this redistribution of flows is predicted to result in less than 2 percent change in the total flows beneath US36, which in our opinion is negligible and within tolerable limits. ✓ ✓

In our opinion, the Proposed Conditions scenario maintains flows beneath US36 similar to Pre-Project conditions and is an acceptable groundwater conveyance system configuration for preliminary design.

4.3.2 - Simulated Connector Pipe Flows

The simulated incremental flows through each connector pipe HSU are presented on Figure 4.8. The configuration of connector pipe HSUs is presented on Figure 4.1. ✓

The majority of flow is passing through HSU 14 (westernmost connector pipe), which is similar to the flow direction of the natural groundwater system. A peak flow of approximately 2,700 cubic feet per day (ft^3/d) (14 gallons per minute [gpm]) was simulated through this ✓

HSU. Based on hydraulic analyses, less than 1 inch of differential head is required to produce this amount of flow through a 10-inch-diameter plastic pipe. In our opinion, the connector pipes will have ample capacity to maintain groundwater levels with negligible headloss across the spillway. Conveyance system sizing will be controlled by constructability and long-term maintenance considerations instead of pipe capacity. The connector pipes will likely require ~~valves~~ gates to regulate seepage through the conveyance system during periods of flood detention.

4.3.3 - Detention Excavation Drain Flows

The simulated incremental drain flow for the Proposed Conditions scenario is shown on Figure 4.9. Seepage into the detention excavation was simulated at approximately 0.8 to 0.9 ft^3/d (less than 0.005 gpm), which is very low and in our opinion is reasonable for a lined pit. We expect that this quantity of seepage will either be removed via evapotranspiration or evacuated through the outlet works conduit.

4.3.4 - Western Embankment Toe Drain Flows

The simulated incremental drain flow for the Proposed Conditions scenario is shown on Figure 4.10. Groundwater collection by the toe drain was simulated at up to 40 ft^3/d (0.2 gpm) and is commonly less than 20 ft^3/d (0.1 gpm), which is very low.

Based on Site topography and our interpretation of subsurface conditions, it appears that a drain pipe can be installed along the western embankment toe between the final grade and the seasonally-high natural groundwater table, and can be installed to slope downward to the north. Water that is collected by this segment of the toe drain pipe can likely be redistributed in the subsurface farther north along the embankment alignment where groundwater decline is predicted to occur near the downstream embankment toe (Figures 4.3 and 4.4).

4.3.5 - Global Water Budget Components and Error

The total predicted inflows and water budget error (the difference between total predicted inflows and outflows within the model) throughout the entire MODFLOW-USG model are shown on Figure 4.11 for the Pre-Project and Proposed Conditions scenarios. Predicted inflows for both simulations were similar in magnitude (within 1 percent of each other). The model convergence and water budget errors for both simulations were very similar and were typically within +/- 0.5 percent, which is within acceptable tolerances based on industry standards for groundwater modeling. In our opinion, the data on Figure 4.11 demonstrates that the simulated proposed conditions are not adversely affecting numerical convergence or flows quantities throughout the model.

flow

5.0 - Sensitivity Testing

RJH performed sensitivity analyses to evaluate how changes in collector/distributor cell hydraulic conductivity would impact results of the Proposed Conditions scenario. This parameter was selected for analysis because it was anticipated to significantly affect model results in the vicinity of the irrigated OSMP fields upstream and downstream of US36 based on RJH's model development process and judgement. The conductivity of the system could also be varied during routine operations by adjusting ~~valves.~~ gates.

The sensitivity analysis evaluated a horizontal hydraulic conductivity of 50,000 feet per day for the collector and distributor trench cells, which was one order of magnitude higher than the value used in the selected Proposed Conditions scenario. The anisotropy ratio for this permeability zone was not changed during the sensitivity simulation. In our opinion, the high hydraulic conductivity value used in the sensitivity simulation represents a practical upper bound for the trench aggregate backfill and is also representative of the conductivity of the drain pipe within each trench.

One-year simulations were used to evaluate a complete cycle of wet and dry seasons similar to the Proposed Conditions scenario.

Results from the sensitivity analysis are presented in Attachment C and are described in the following sections. ✓

5.1 - Sensitivity of Heads

Figures C.1 and C.2 show simulated head results for representative winter and summer months, respectively, using the same contouring pattern described in Section 3.2. Based on ✓ comparison of the results on Figures C.1 and C.2 to those on Figures 4.3 and 4.4, the magnitude and spatial distribution of head results are generally similar regardless of whether the collection and distribution trenches are modeled using a conductivity of 5,000 feet per day or 50,000 feet per day. Groundwater decline is slightly greater on OSMP South fields when a higher system conductivity is used; this illustrates the importance of providing ~~valves~~ gates on the conveyance system so its capacity can be restricted as needed to mimic pre-Project groundwater levels.

5.2 - Sensitivity of Flows

The simulated incremental and cumulative flows through the soil beneath US36 for the Pre-Project and Sensitivity Analysis scenarios are presented on Figure C.3. Flow results for each of the HSUs beneath US36 (HSU 6 through 9) are shown on Figures C.4 through C.7. ✓ The flow patterns shown on these figures for the Sensitivity Analysis are very similar (within less than 1 percent) to the flows for the Proposed Conditions Scenario (Figure 4.7 and ✓ Attachment B). We interpret this to mean that the overall flow through the system is being controlled by the surrounding alluvium instead of by the conveyance system facilities.

5.3 - Sensitivity Summary

Based on the results of the sensitivity analysis presented in Section 5, the Proposed Conditions scenario model is relatively insensitive to increases in collector/distributor trench permeability. Increases in this parameter value appear to cause slightly greater change on OSMP South fields; however, the predicted results are still reasonable for 30 percent design. From these analyses we conclude:

- The groundwater conveyance system is expected to operate adequately under a wide range of system capacity.
- We observed head decreases of 0.2 to 0.4 feet upstream of the spillway alignment after increasing the trench hydraulic conductivity by one order of magnitude. The ✓ conveyance system will likely need to be regulated (e.g., restrict flow through the system) to improve system performance and reduce difference between pre-Project and post-Project groundwater levels. differences

6.0 - Conclusions

We conclude the following from the information presented within this memorandum:

- The groundwater conveyance system simulated within the Proposed Conditions scenario is an acceptable preliminary design solution (Figure 4.3 and 4.4). ✓
    - Negligible groundwater head effects are predicted to occur on OSMP North fields.
    - Groundwater head effects on the OSMP South fields are predicted to be head declines that are generally less than 1.0 foot and predominantly less than 0.5 feet. These effects generally occur in the western portion of OSMP South fields. We suspect that these declines could be mitigated during operation by throttling gates to slightly restrict flow through the conveyance system. ✓
    - Groundwater head effects up to about 1 to 2 feet are predicted to exist near the spillway alignment; however, these are mostly limited to areas within the CDOT ROW. ✓
    - Groundwater decline of about 0.6 to 2.6 feet is predicted to exist in a localized area between the embankment and the Table Mesa RTD Park-N-Ride. This area consists of highway interchanges within the CDOT ROW, and the predicted groundwater deficiencies are not anticipated to cause adverse effects in this area. ✓
    - Groundwater head effects elsewhere throughout the Site are relatively minor and are typically limited to the CU South campus and adjacent CDOT ROWs.
    - The simulated preliminary design solution maintains similar flows beneath US36 compared to simulated Baseline Model conditions. Some redistribution of flows beneath US36 is predicted to occur; however, the preliminary design solutions provide total flows beneath US36 that are within 2 percent of the Proposed Conditions model. ✓
    - Simulated seepage into the detention excavation is minor (less than 1 ft^3/d). ✓
    - Groundwater head mounding of about 0.5 to 1.7 feet are predicted to exist below the western portion of the embankment. Simulated groundwater collection by the western embankment toe drain is minor (up to 40 ft^3/d [0.2 gpm]). ✓
- The Proposed Conditions scenario supports the preliminary design of features to mitigate groundwater impacts.
    - Collector and distributor pipes parallel to the spillway secant pile wall will be constructed as continuous trenches with periodic pipe gates and backfill plugs to control the distribution of flow through the system.
    - Connector pipes through the spillway secant pile wall will be constructed with gates to regulate seepage through the conveyance system during periods of flood detention.
    - Seepage into the detention excavation will be removed either via evapotranspiration or through the outlet works conduit.
    - A drain pipe will be installed along the downstream toe of the embankment dam to manage seepage through the dam. On the west end of the Site, the pipe will be installed near the seasonally-high groundwater table observed in nearby monitoring wells to prevent routine collection of groundwater, which will result in some inevitable mounding during winter when the groundwater table is naturally low. The mounding is estimated to be up to approximately 1.7 feet compared to Pre-Project levels and is predicted to remain on CU South Campus property. We expect that the water collected by this portion of the toe drain pipe can be conveyed by gravity and redistributed within the subsurface farther north where groundwater declines are predicted to exist near the downstream embankment toe. ✓
    - The Proposed Conditions scenario results are generally stable and relatively insensitive to a reasonable range of conveyance system hydraulic conductivity.

Flow through the conveyance system will likely need to be regulated to achieve desired performance.

  - During future phases of design, the groundwater model will be updated to more closely match the configurations of proposed facilities and to confirm that the groundwater conveyance systems operate appropriately under a range of expected site conditions and hydraulic loading events.

- The conveyance system should include multiple gates to provide redundancy and flexibility of operations.
  - Gates should be positioned to maintain the desired non-flood groundwater levels without allowing excessive seepage during flood loads.
  - Long-term groundwater monitoring will be required after construction. Iterative gate adjustments will likely be required to initially obtain the desired post-construction groundwater patterns.

7.0 - References

Cedergren, H.R. (1989). *Seepage, Drainage, and Flow Nets*. Third Edition.

Panday et al. (2017). *MODFLOW-USG version 1.4.00: An unstructured grid version of MODFLOW for simulating groundwater flow and tightly coupled processes using a control volume finite-difference formulation*. U.S. Geological Survey Software Release. October.

RJH Consultants, Inc. (2021). *Baseline Groundwater Model Report.* South Boulder Creek Regional Detention Project. July.

U.S. Bureau of Reclamation (Reclamation) (2014). *Design Standards No. 13: Embankment Dams. Chapter 8: Seepage.*

Attachments:

Figures 3.1 through 4.11

Attachment A	Bedrock Hydraulic Conductivity for MODFLOW Groundwater Modeling of Proposed Conditions
Attachment B	Flows Beneath US36 by HSU
Attachment C	Conveyance System Permeability (K) Sensitivity

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Figure 3.1
Horizontal Flow Barrier (HFB) Alignments

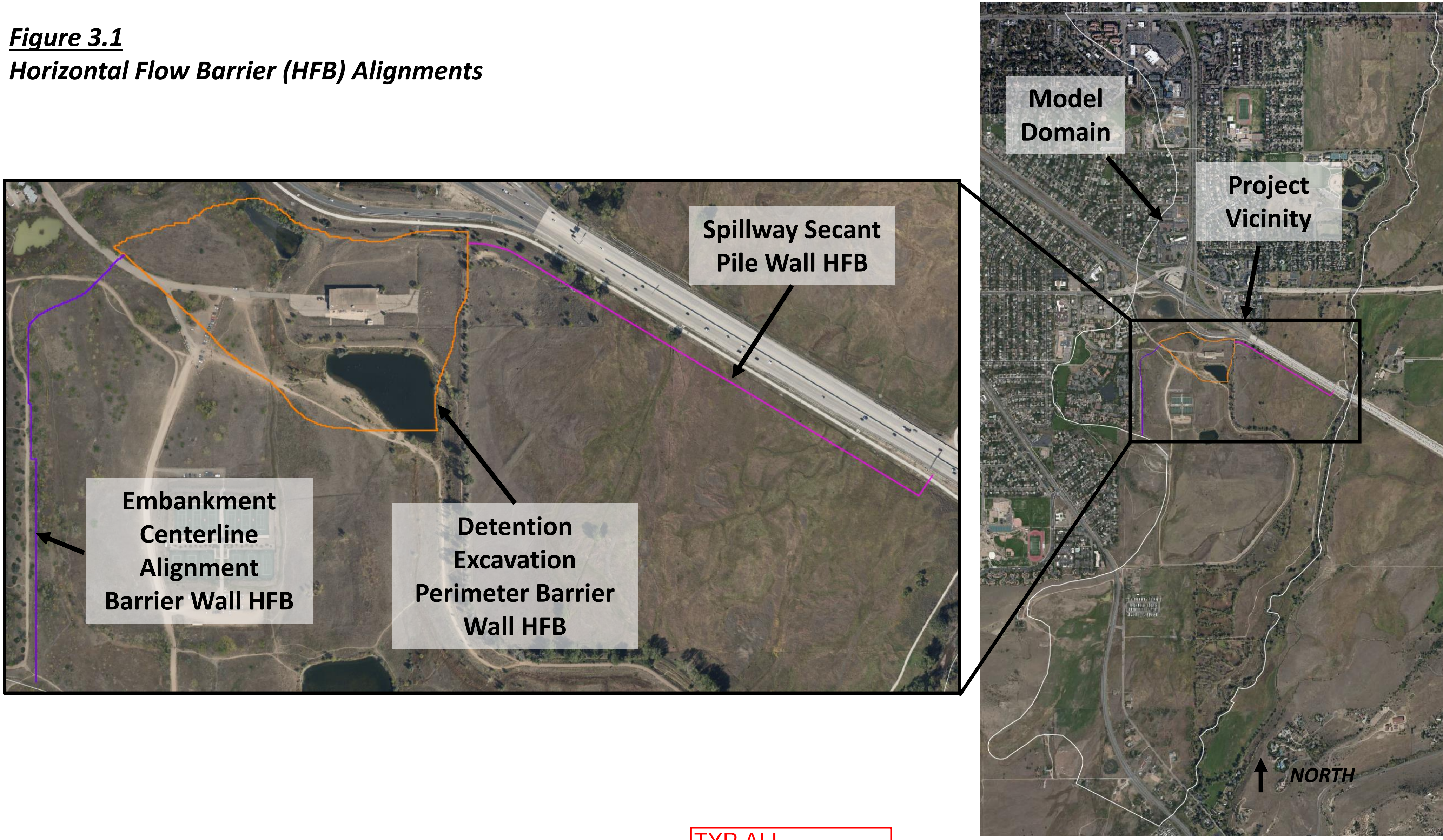

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Figure 3.2

Detention Excavation Hydraulic Conductivity Zonation

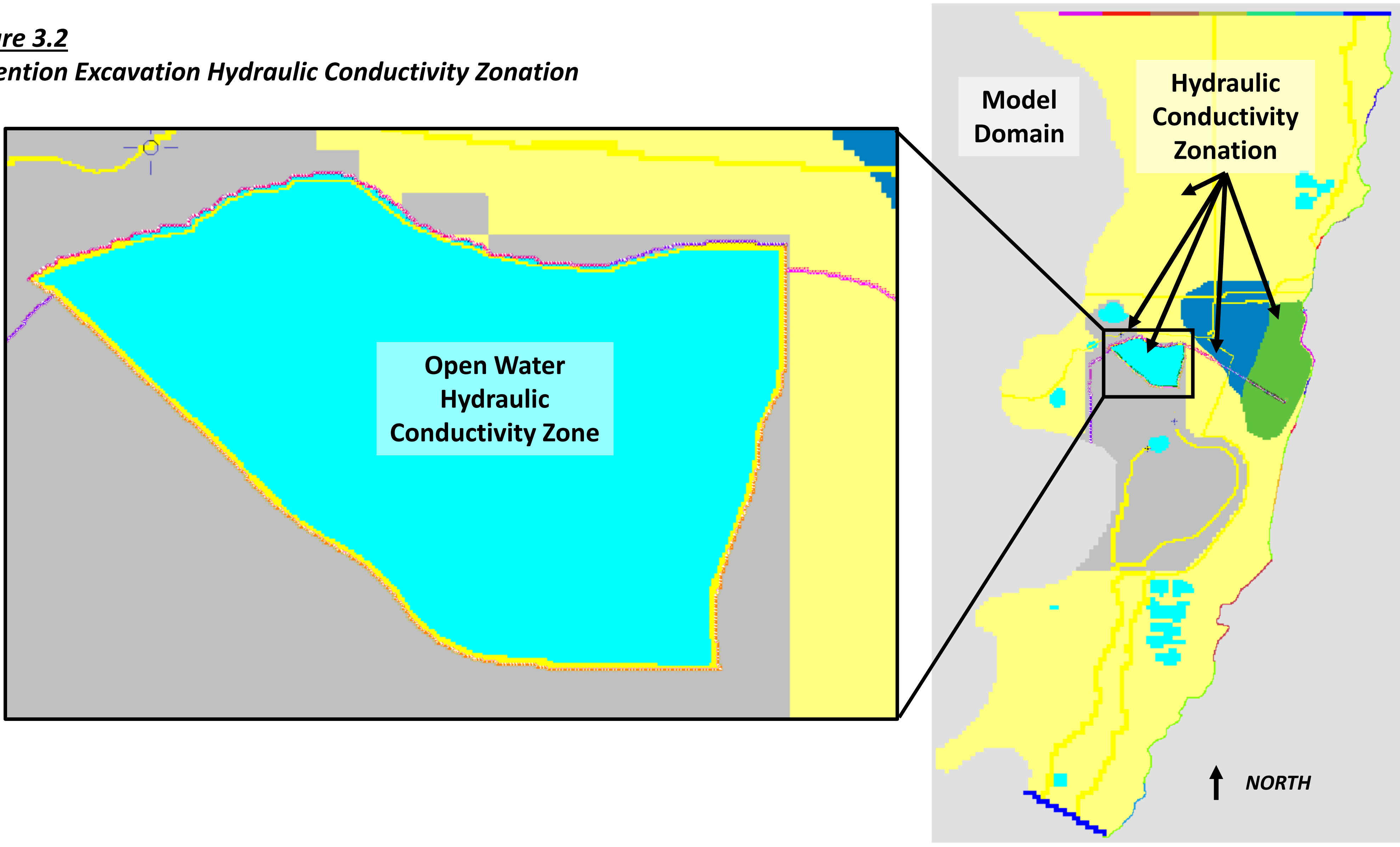

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Figure 3.3

Detention Excavation Interior Drain BC

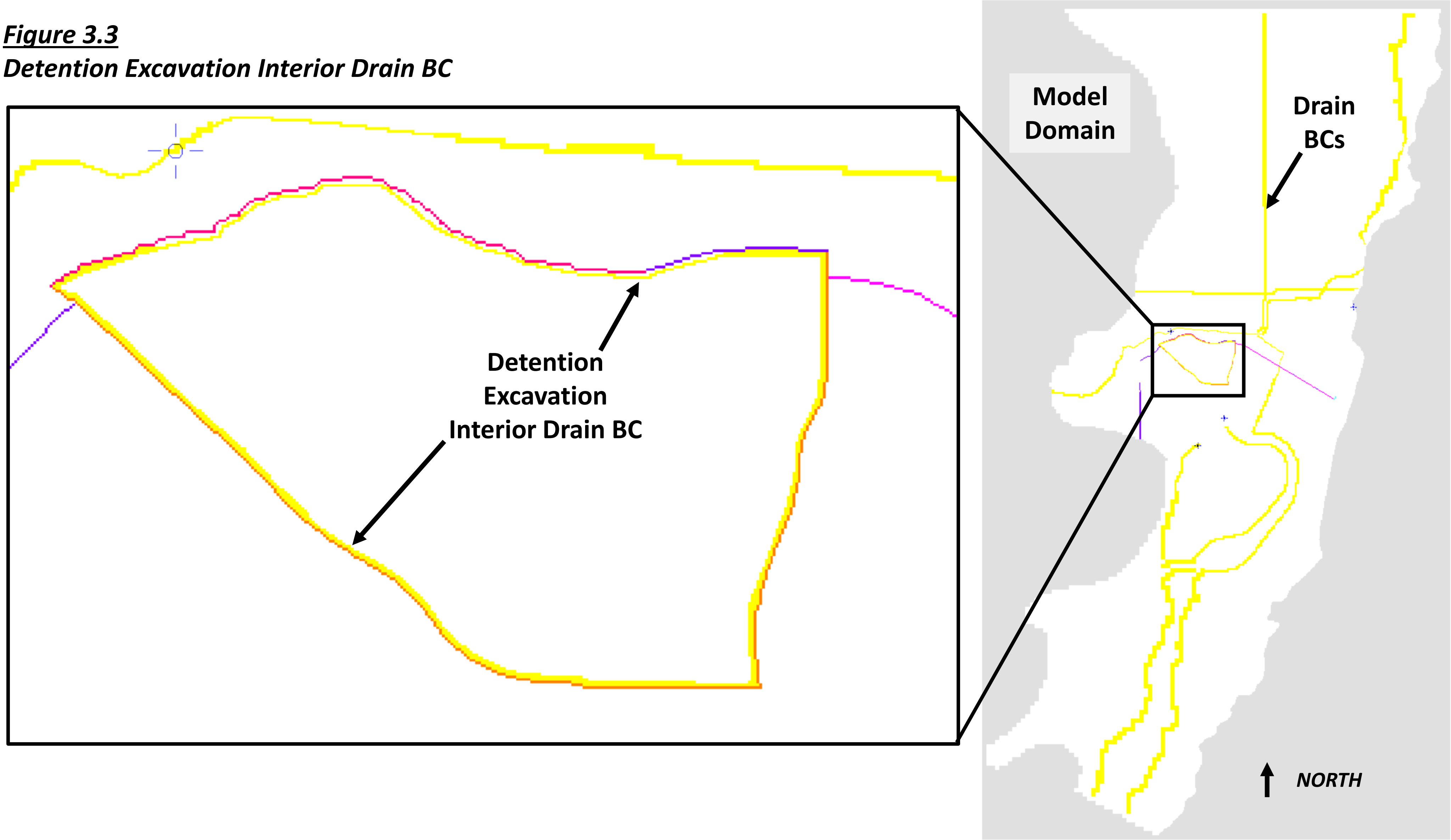

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Figure 3.4

'No Conveyance System' Scenario – Winter Head Results

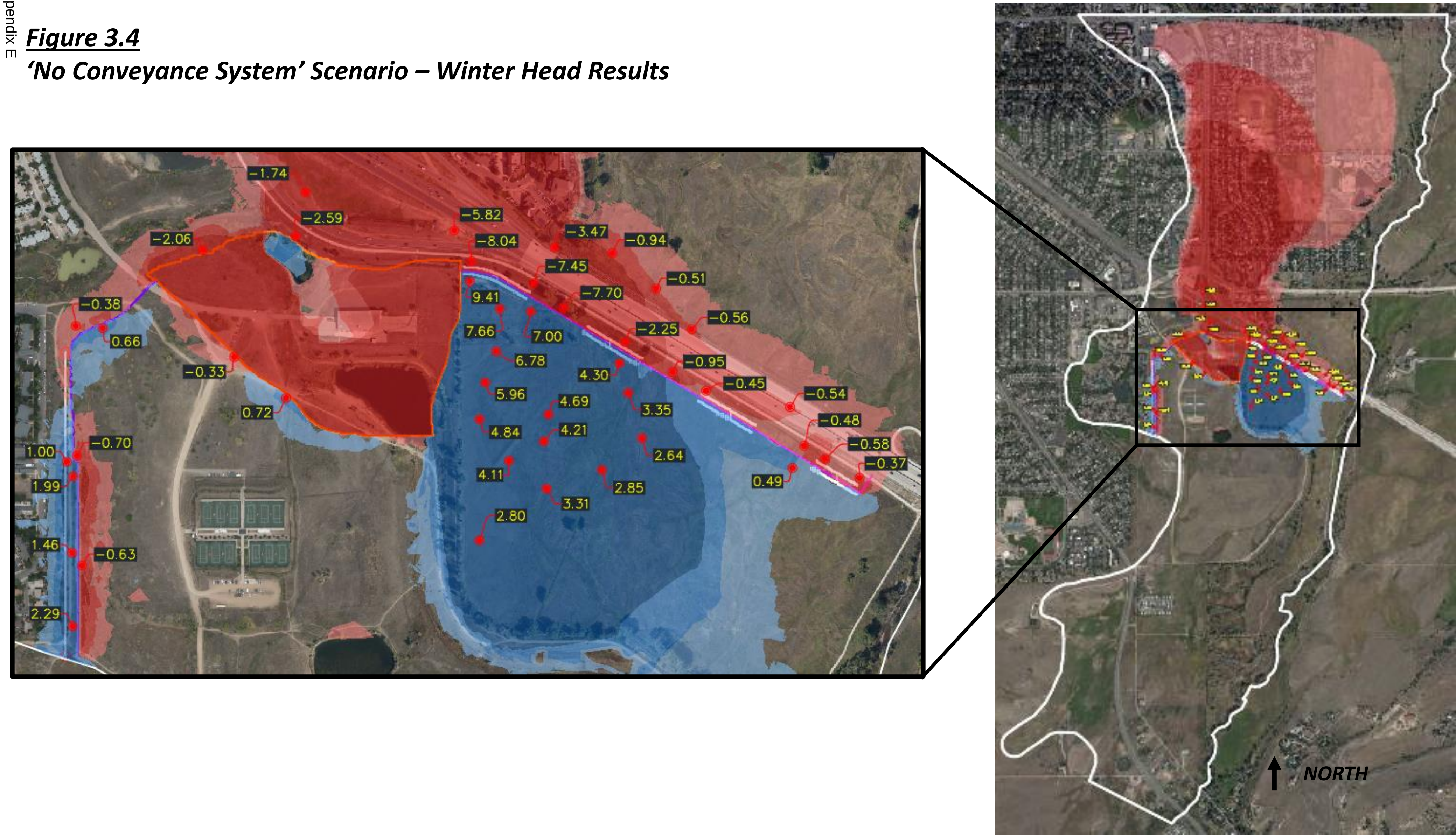

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Figure 3.5
‘No Conveyance System’ Scenario – Summer Head Results

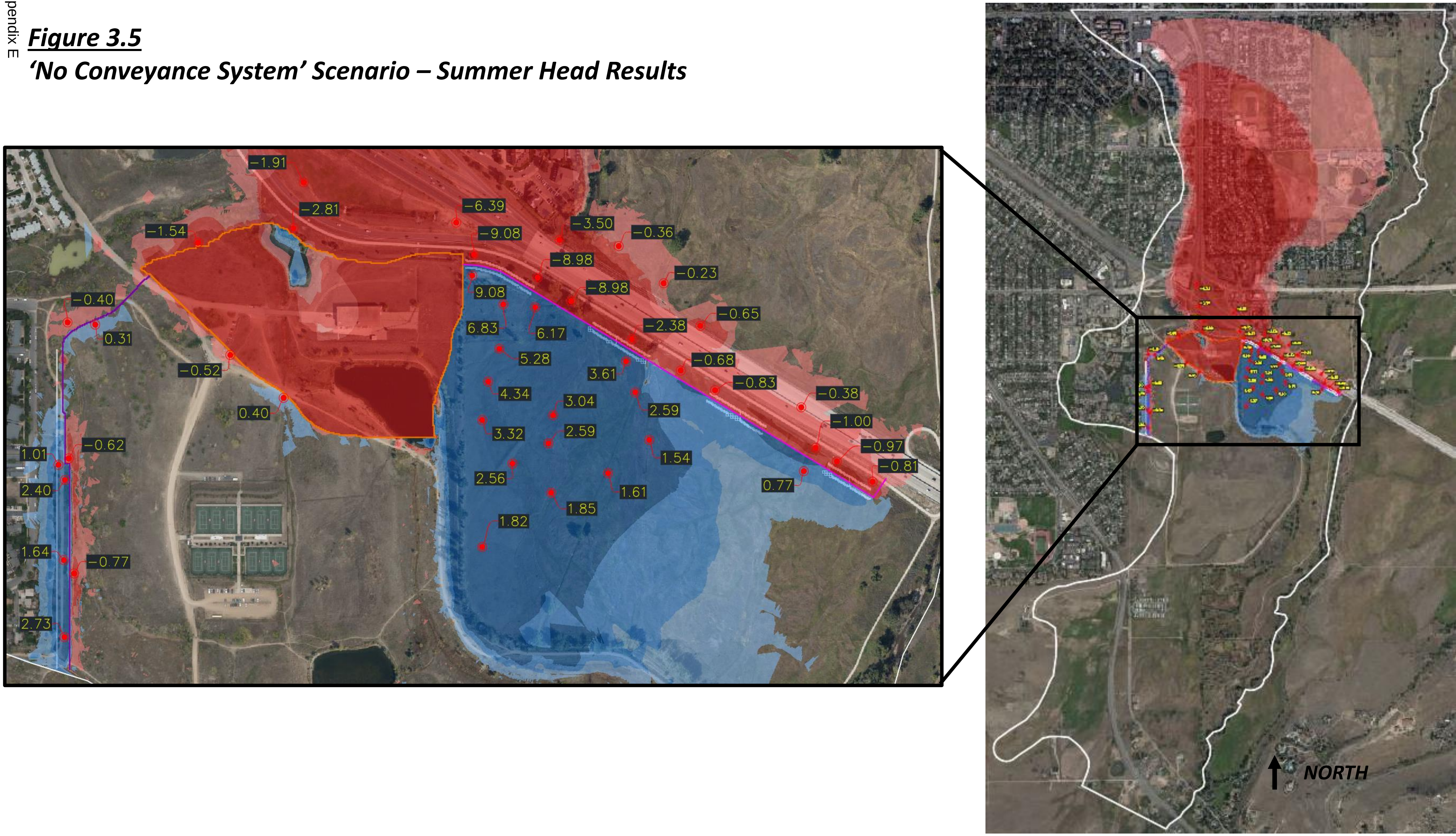

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Figure 4.1
Groundwater Conveyance System Hydraulic Conductivity Zonation

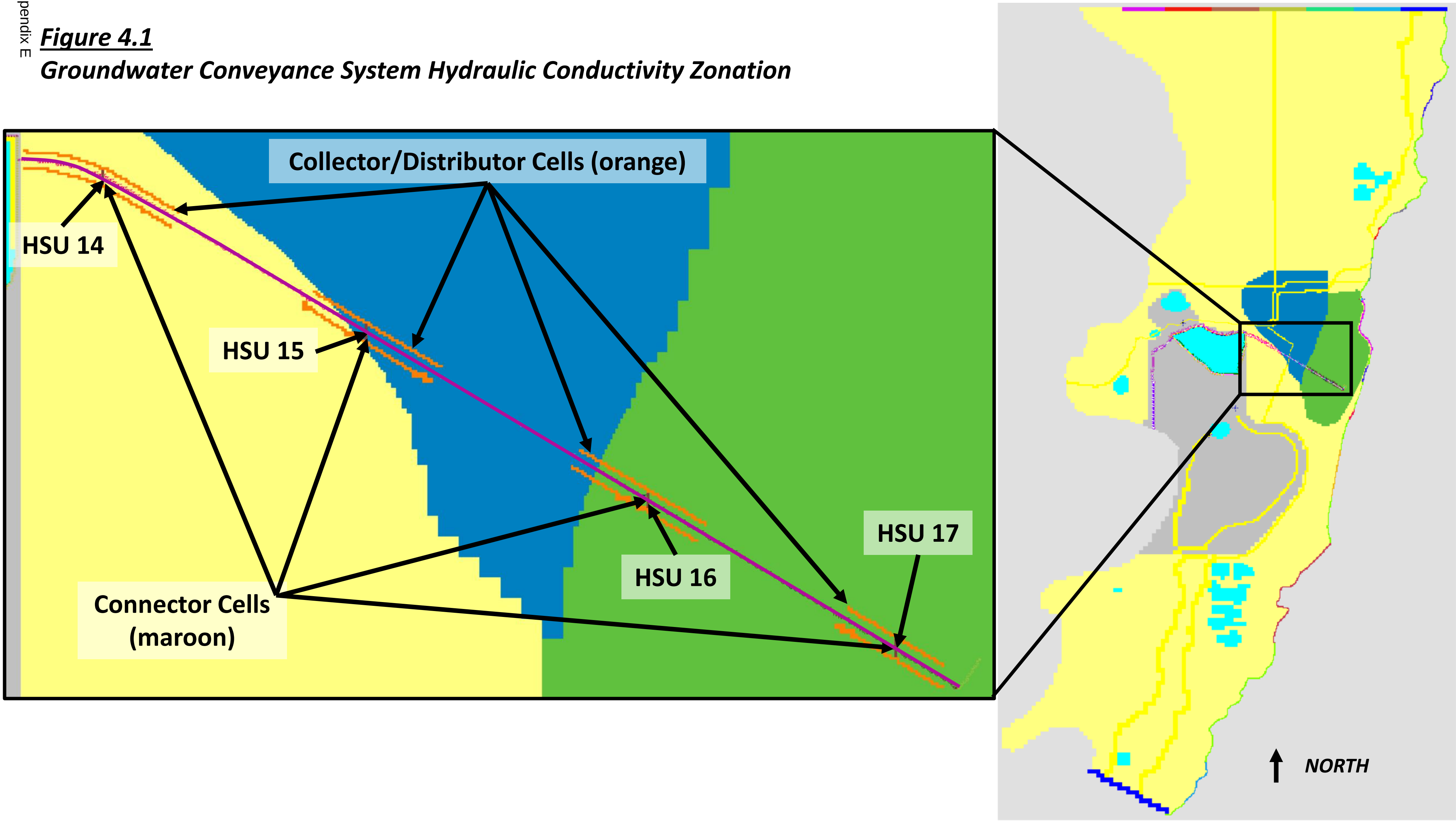

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Figure 4.2

Western Embankment Toe Drain BC

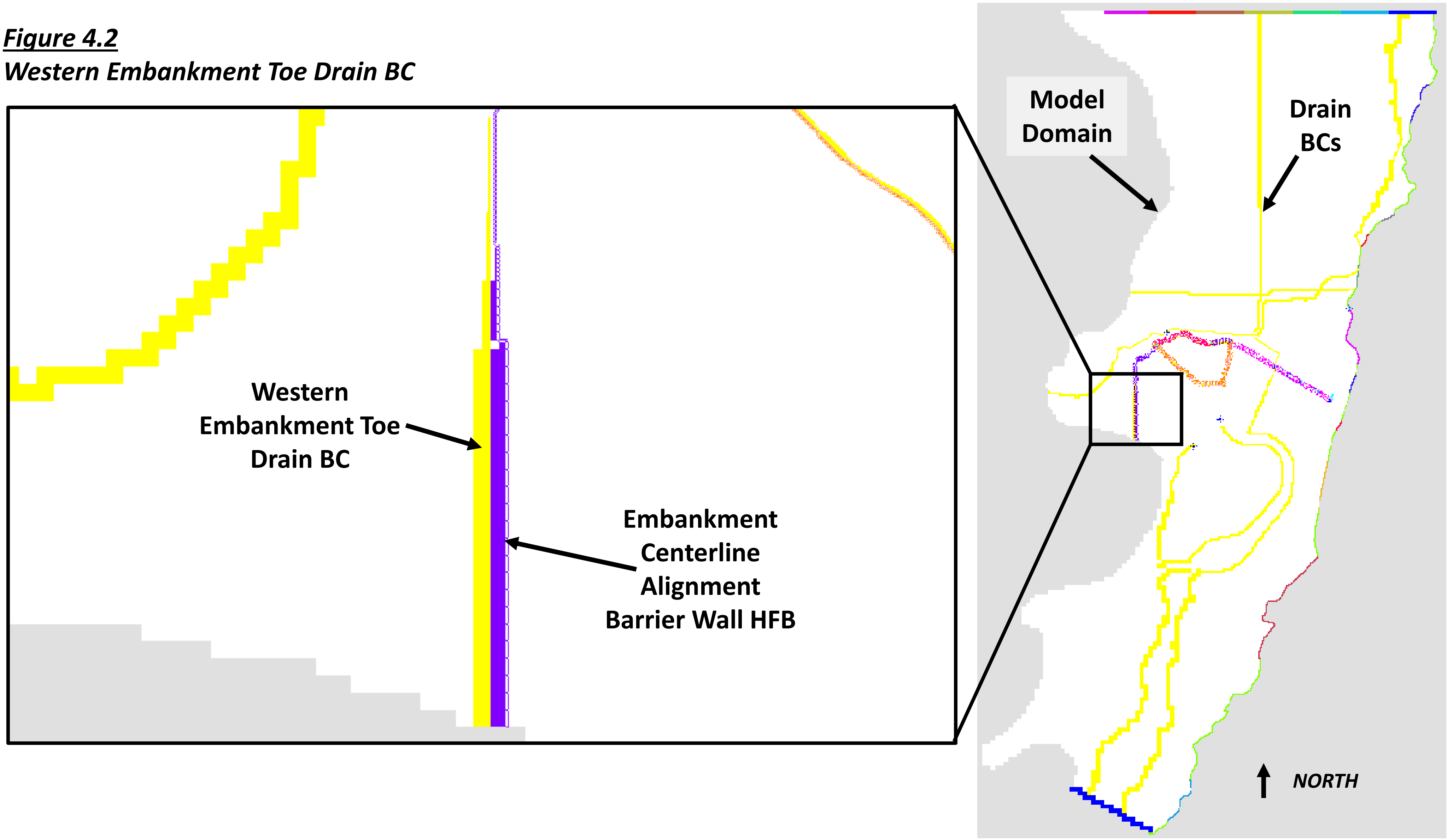

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Figure 4.3

‘Proposed Conditions’ Scenario – Winter Head Results

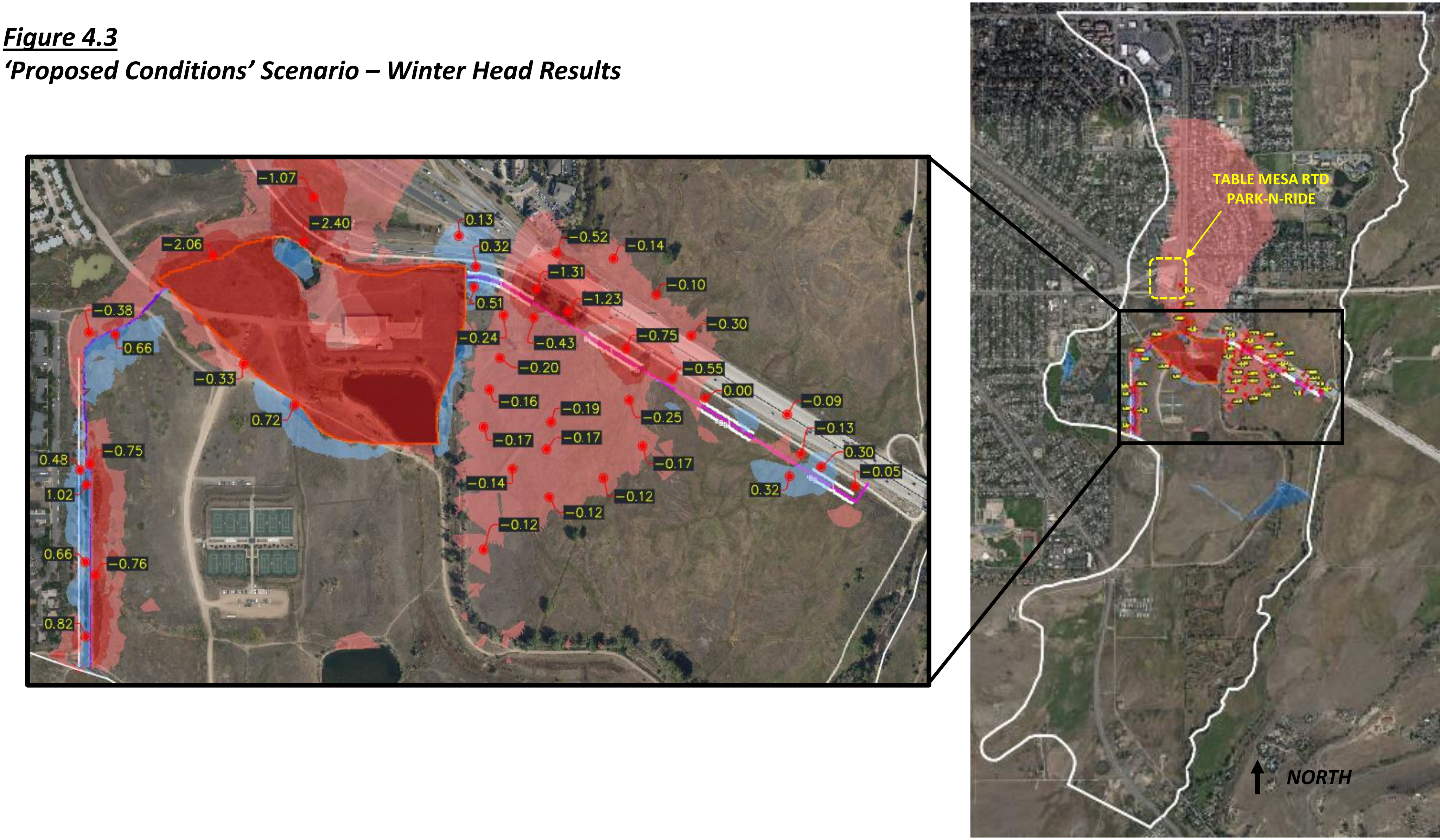

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Figure 4.4

'Proposed Conditions' Scenario – Summer Head Results

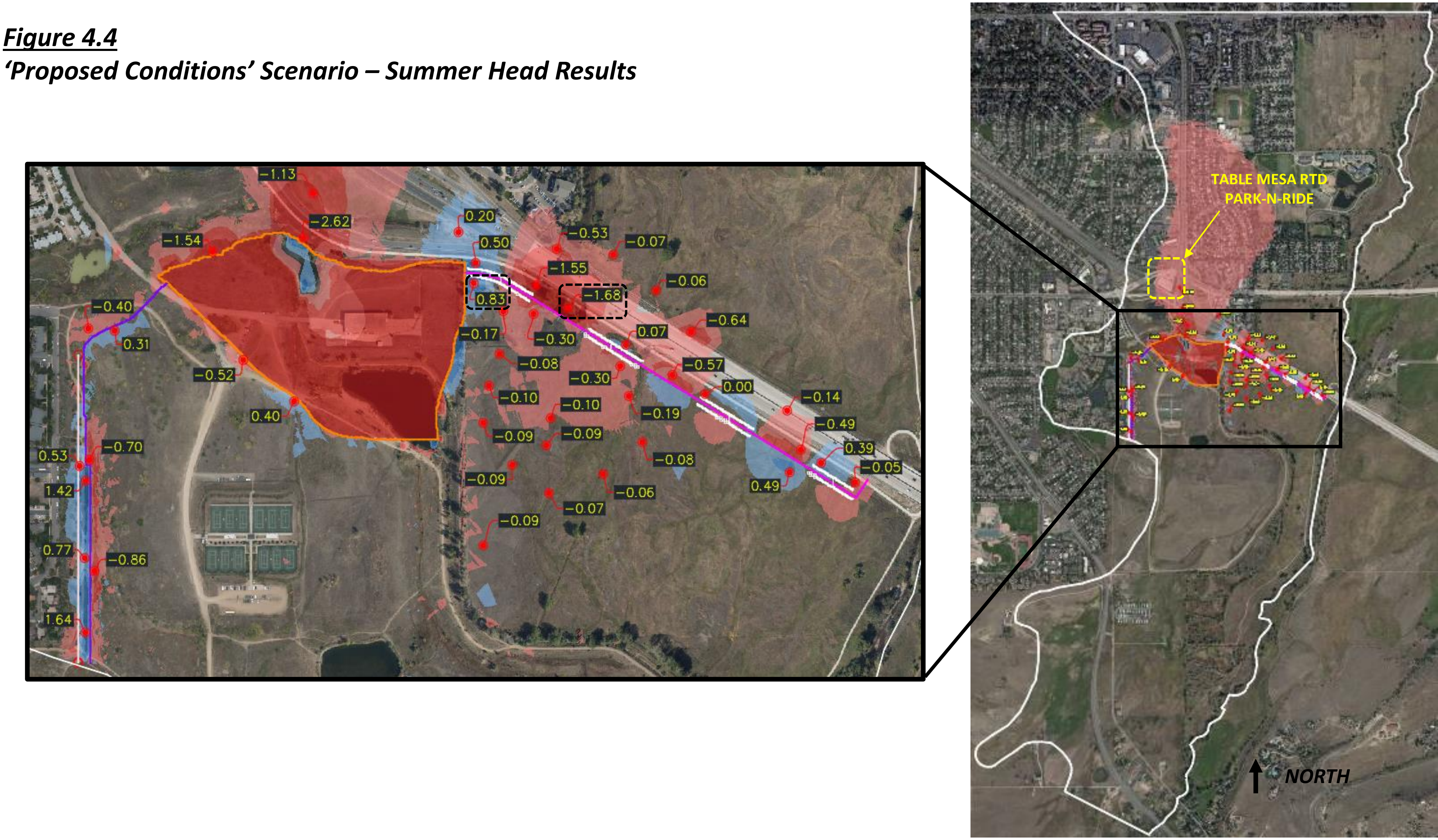

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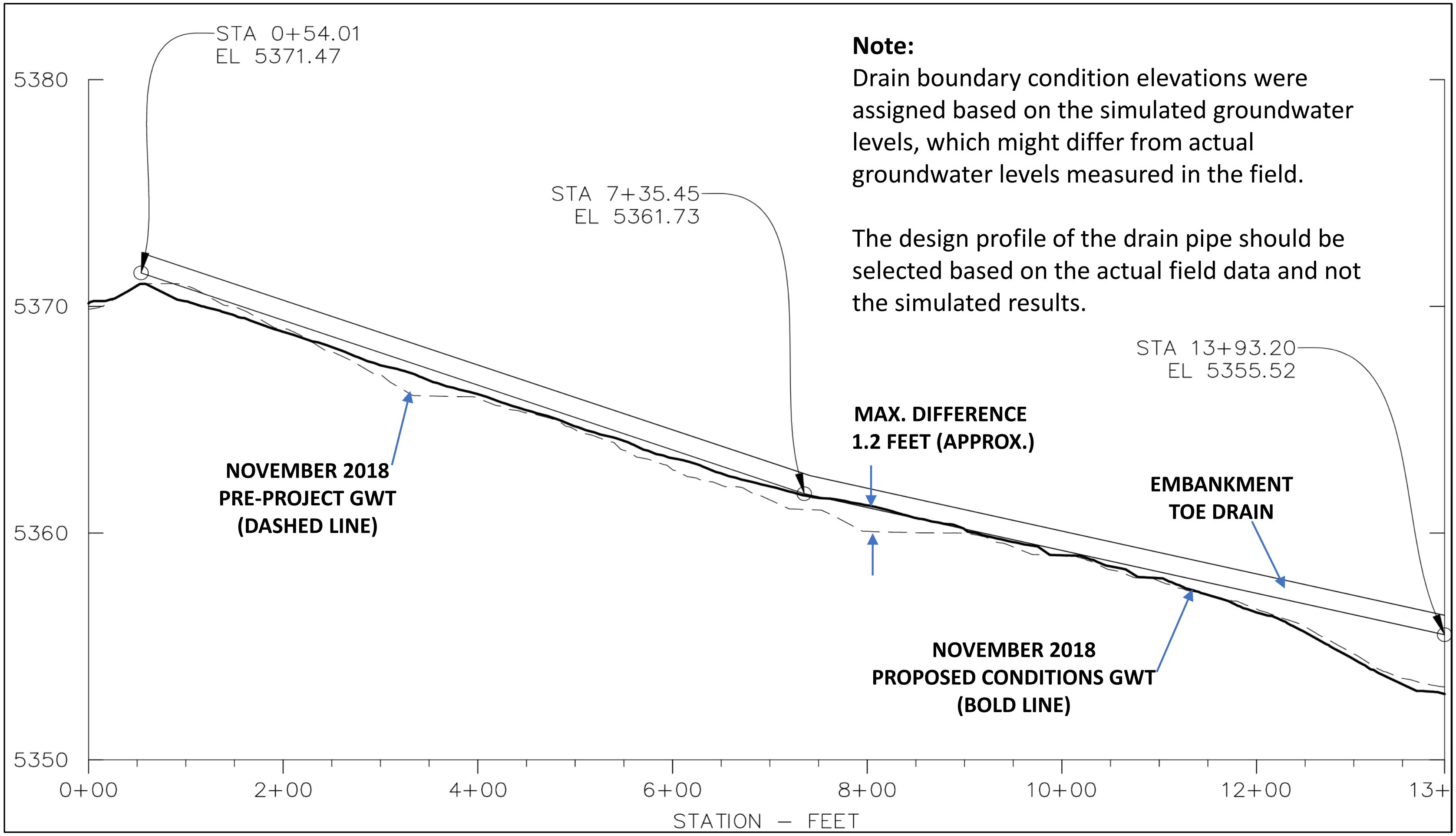
Figure 4.5

Western Embankment Toe Drain BC – Winter Head Results Profile

DRAIN ALIGNMENT PLAN



Note:
Drain boundary condition elevations were assigned based on the simulated groundwater levels, which might differ from actual groundwater levels measured in the field.

The design profile of the drain pipe should be selected based on the actual field data and not the simulated results.

- Stress Period 2 (November 2018) generally yielded highest GWT elevations along the drain profile

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Figure 4.6
Groundwater Conveyance Features and HSU Locations

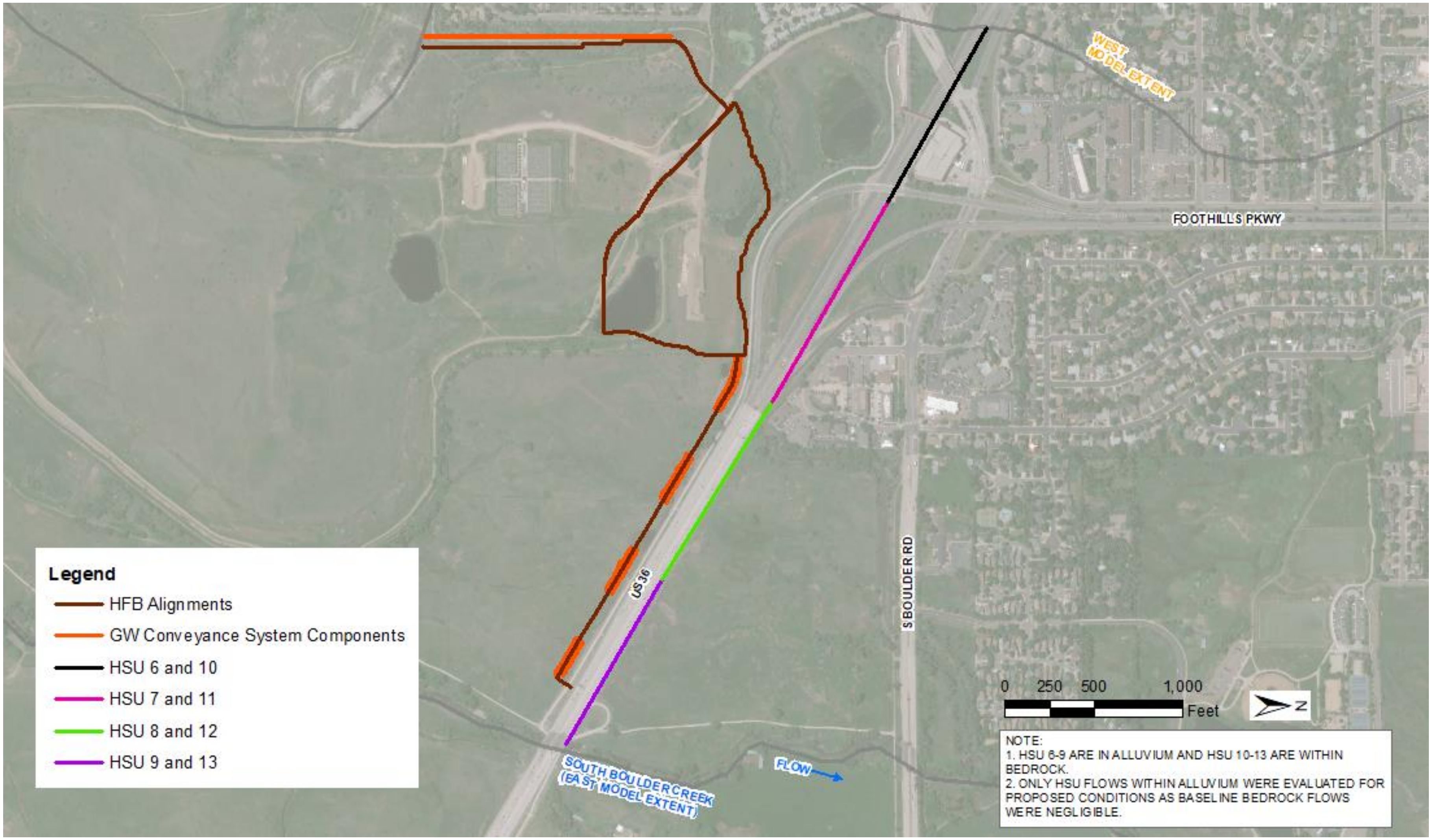

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Figure 4.7
Simulated Flows Beneath US36 (Combined)

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through the alluvium HSUs are predicted to decrease by about 117 cu ft per day (less than 2 percent of Pre-Project flow rates).

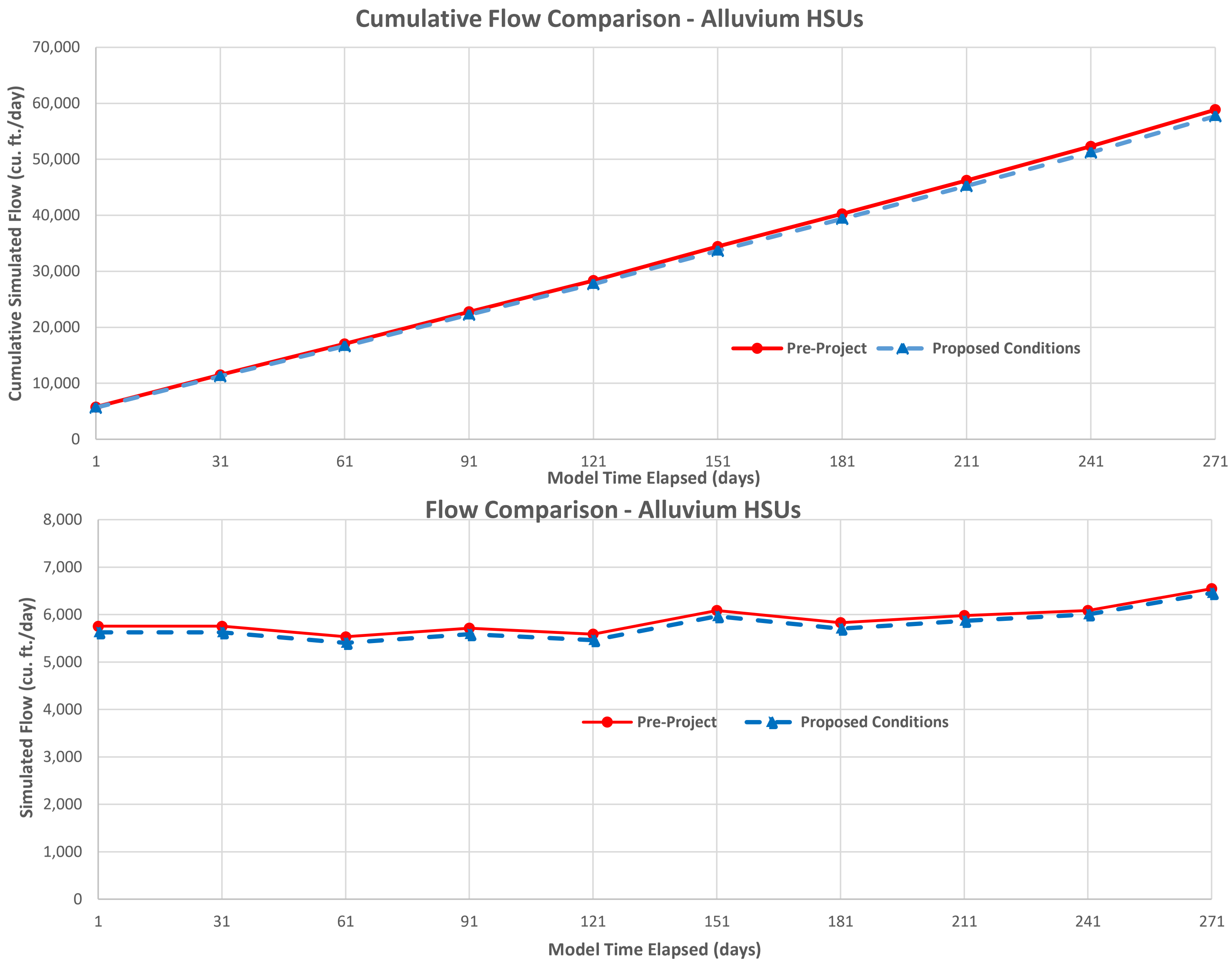

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Figure 4.8
Simulated Flows through Connector Pipe HSUs

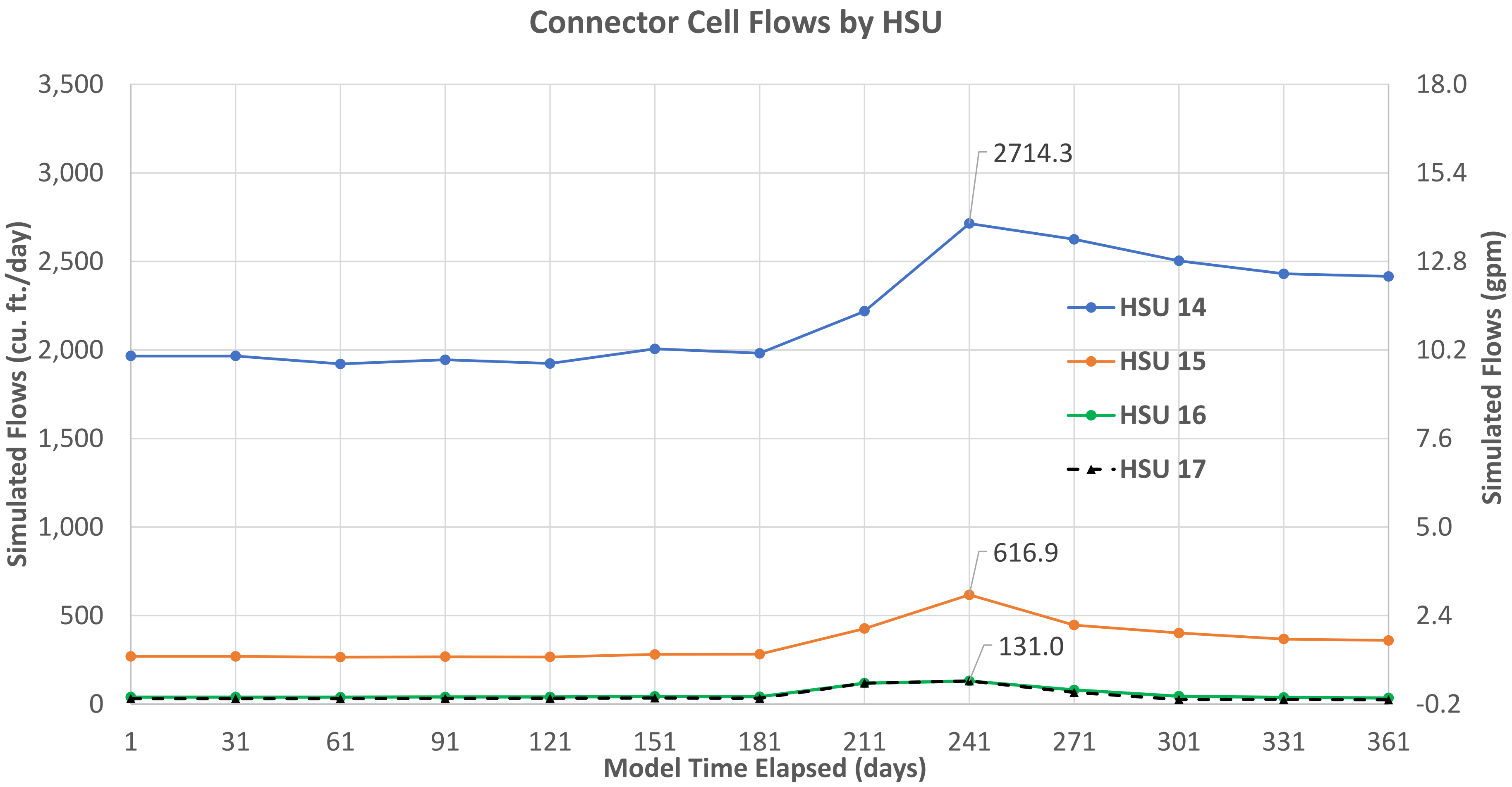

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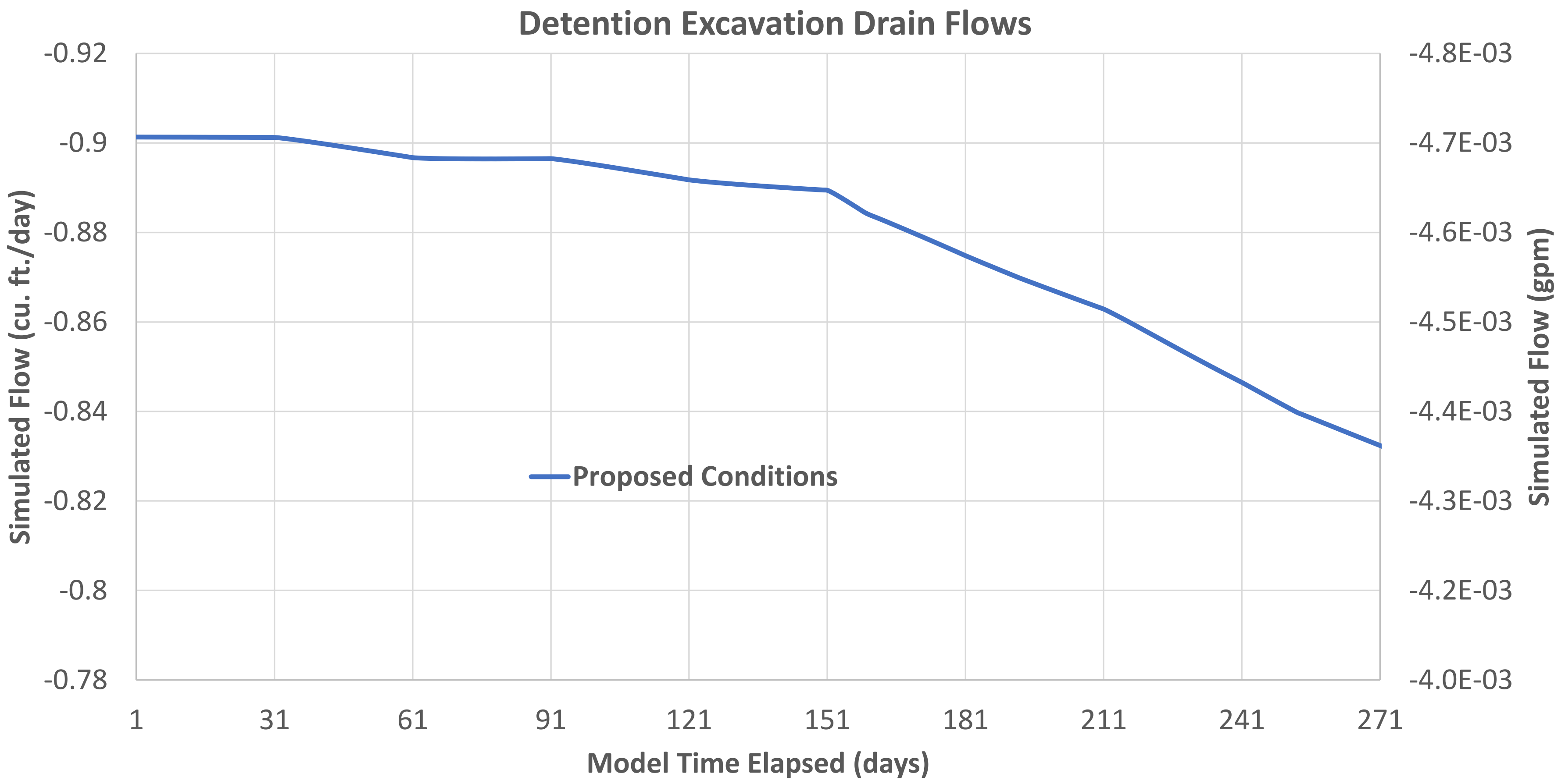
Figure 4.9

Simulated Flows through Detention Excavation Drain BC



Note:

Negative flow values mean that model flows are exiting the domain through the Drain boundary.

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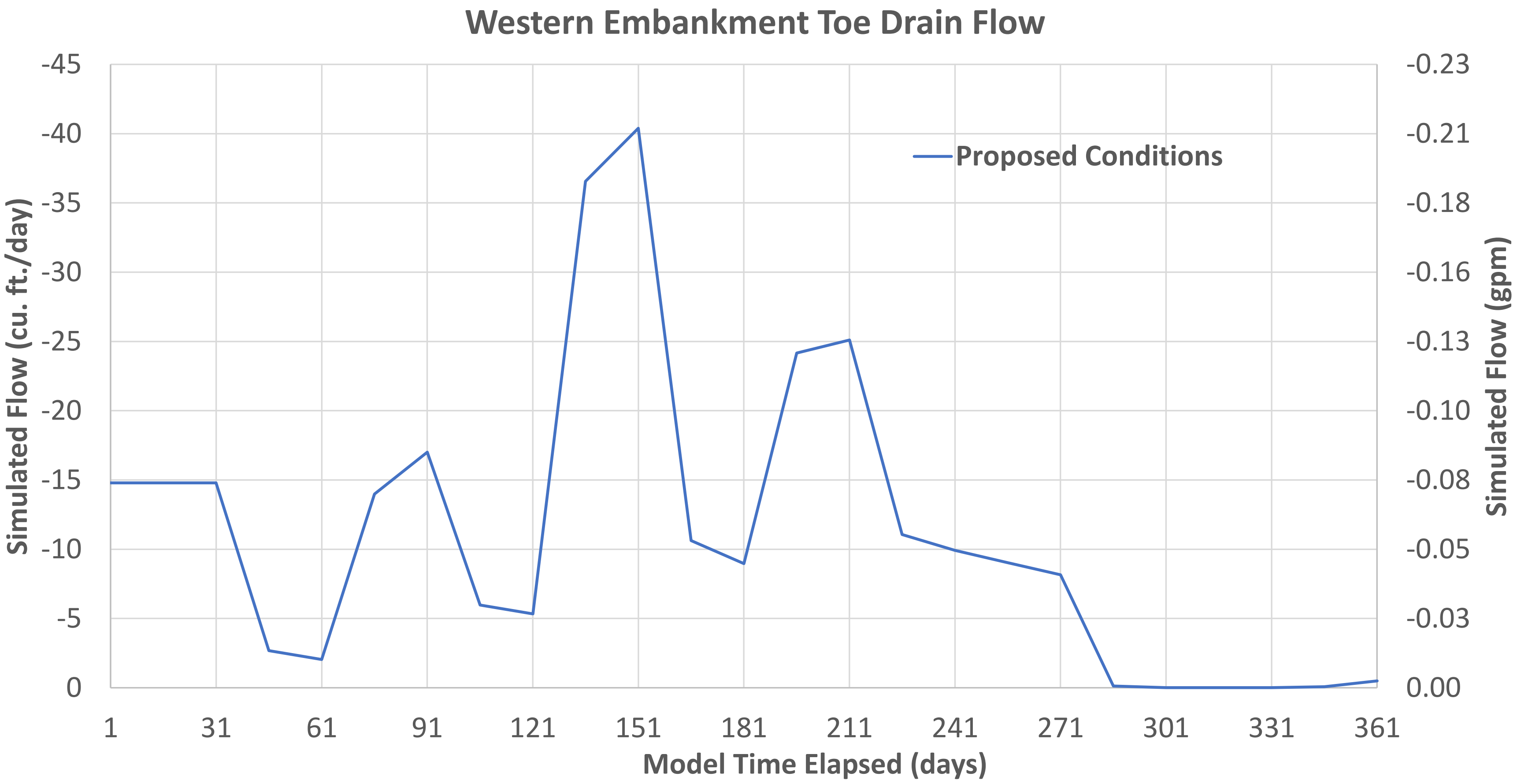
Figure 4.10

Simulated Flow through Western Embankment Toe Drain BC



Note:

Negative flow values mean that model flows are exiting the domain through the Drain boundary.

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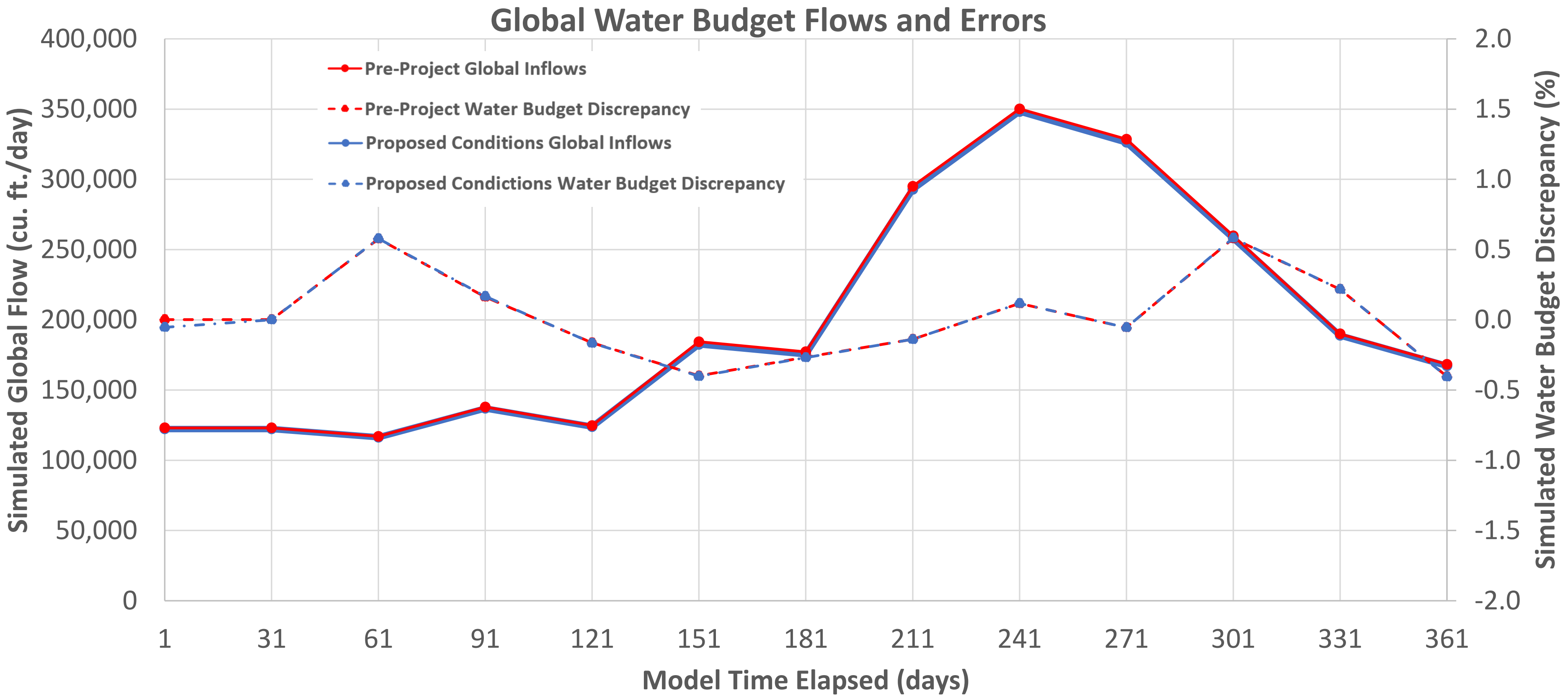
7/7/22 JNH

Figure 4.11
Global Water Budget and Errors



Note:
- Water budget errors are presented incrementally for each stress period.
- Inflows represent the average daily flow rate throughout each stress period. Positive differences and percent discrepancies mean inflow exceeds outflow.
- Negative differences and percent discrepancies mean outflow exceeds inflow.

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7/7/22 JNH

ATTACHMENT A

BEDROCK HYDRAULIC CONDUCTIVITY FOR MODFLOW GROUNDWATER MODELING OF PROPOSED CONDITIONS

7/7/22 JNH

ANALYSES MEMORANDUM

Project No. 16134

Performed: 7/20/2021 **By:** ABP

Checked: 7/22/2021 **By:** ATM

Approved: ______ **By:** ______

CLIENT: **City of Boulder**

SUBJECT: **South Boulder Creek Regional Detention Project**

Bedrock Hydraulic Conductivity for MODFLOW Groundwater Modeling of Proposed Conditions

1.0 - Introduction

RJH is using the computer program MODFLOW to simulate the groundwater system at the South Boulder Creek Regional Detention Facility Site for existing (i.e. Baseline) and proposed (post-construction) conditions. The purpose of this memorandum is to document differences in bedrock hydraulic conductivity that exist between the Baseline groundwater models and the proposed conditions models.

2.0 – Baseline Modeling

Material properties used in the Baseline model were developed based on data collected during the Phase I geotechnical investigation (RJH, 2019). Data obtained from nine Packer tests performed in three borings during the Phase I investigation were used to develop hydraulic conductivity properties for bedrock and are summarized on Figure 5.4 of the Baseline Report (RJH, 2021) (Exhibit 1). The Packer test results used to develop the Baseline model material properties were performed in B-110(P), B-111(P), and B-112(P) at the locations shown on Exhibit 2. These borings were generally located near and north of the proposed embankment and spillway alignments.

Based on the distribution of Packer test results and samples recovered during Phase I investigation, we divided the bedrock into weathered and unweathered zones during development of the Baseline groundwater model. The horizontal hydraulic conductivities assigned to the weathered and unweathered bedrock zones in the calibrated Baseline model were 1.4E-4 and 2.5E-5 cm/s respectively. As shown on Exhibit 1, these selected values were near the upper end of the Phase I Packer test results and in our opinion are relatively high for the Pierre Shale.

Sensitivity analyses were performed during development of the Baseline model and it was identified that the calibrated model was relatively insensitive to decreases in hydraulic conductivity of bedrock. During sensitivity analyses, both the weathered and unweathered bedrock zones were assigned a horizontal hydraulic conductivity of about 2E-7 cm/s, which in our opinion is reasonable for an aquitard such as the Pierre Shale. Additional information about the sensitivity analyses is presented in the Baseline Report (RJH, 2021).

3.0 – Proposed Conditions Modeling

Forty-nine additional Packer tests were performed in fifteen borings during the Phase II geotechnical investigation after calibration of the Baseline model had begun. These Packer tests were performed in the 200-series borings shown on Exhibit 2, and were generally located along the alignments of the proposed spillway, embankment, and barrier wall. Two of the borings (B-214(P) and B-215(P)) were located within the zone of bedrock previously modeled as weathered Pierre Shale. Each test result was less than 3E-7 cm/s and most tests exhibited no flow (nominally 1E-7 cm/s). These results were generally lower than those from the Phase I investigation and in our opinion are reasonable for the Pierre Shale. Additional information is presented in the Phase II Geotechnical Report (in progress).

For groundwater modeling of the proposed Project facilities, we selected to use a horizontal hydraulic conductivity of 2E-7 cm/s for both the weathered and unweathered bedrock zones, which differs from the values used in the calibrated Baseline model. In our opinion this adjustment is appropriate because:

1. Significantly more Packer tests were performed during Phase II than in Phase I, and all of the Phase II Packer test results were 3E-7 cm/s or lower. In our opinion a horizontal hydraulic conductivity of 2E-7 cm/s is reasonable for an aquitard such as the Pierre Shale.
2. Packer tests were performed in significantly more borings during Phase II than during Phase I, which provides additional information about the spatial variability of hydraulic conductivity throughout the Site. The general hydraulic conductivity of the overall rock formation is expected to be relatively low, although higher hydraulic conductivities could exist in localized fractured zones.
3. Sensitivity analyses performed during development of the Baseline model showed that reducing the bedrock hydraulic conductivity to 2E-7 cm/s did not significantly affect model calibration.

4.0 – References

RJH Consultants, Inc. (2019). *Phase I Geotechnical Report.* South Boulder Creek Regional Detention Project. August.

RJH Consultants, Inc. (2021). *Baseline Groundwater Model Report.* South Boulder Creek Regional Detention Project. July.

5.0 – Attachments

Exhibit 1 – Figure 5.4 from the Baseline Groundwater Model Report

Exhibit 2 – Exploration Location Map

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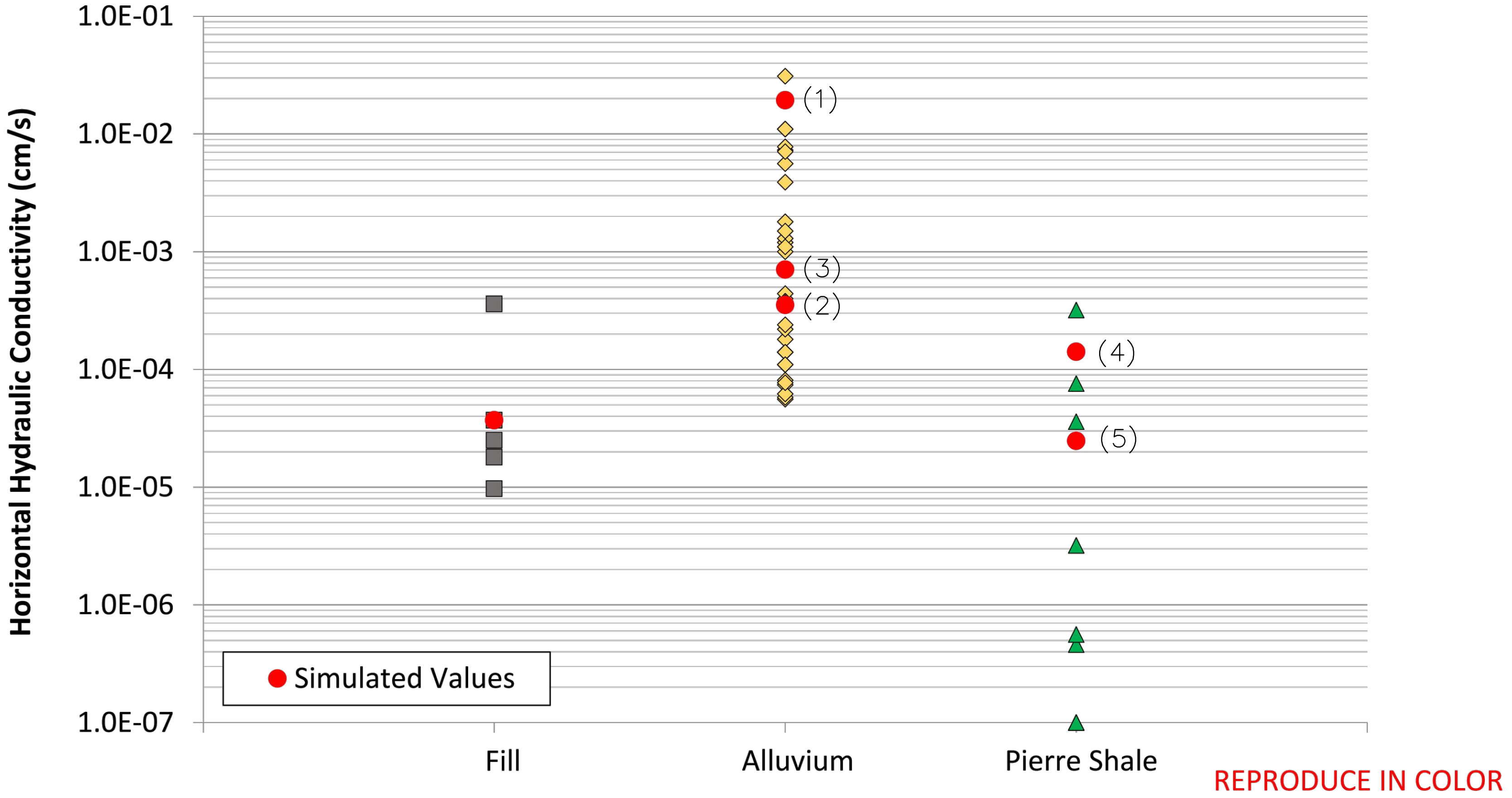

REPRODUCE IN COLOR

NOTES:

1. ALLUVIUM ZONE A
2. ALLUVIUM ZONE B
3. ALLUVIUM ZONE C
4. WEATHERED PIERRE SHALE
5. UNWEATHERED PIERRE SHALE
6. INDIVIDUAL DATA POINTS REPRESENT TEST RESULTS FROM IN SITU HYDRAULIC CONDUCTIVITY TESTS.

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION -
BASELINE GROUNDWATER
MODEL REPORT

PROJECT NO. 16134

GEOLOGIC UNIT
HYDRAULIC CONDUCTIVITY

July 2021

Figure 5.4

P:\16134 – SOUTH BOULDER CREEK\CAD\FIGURES\GROUNDWATER\BASELINE_REPORT\16134_20-07-28_FIGURES.DWG 7/8/2021 2:39 PM

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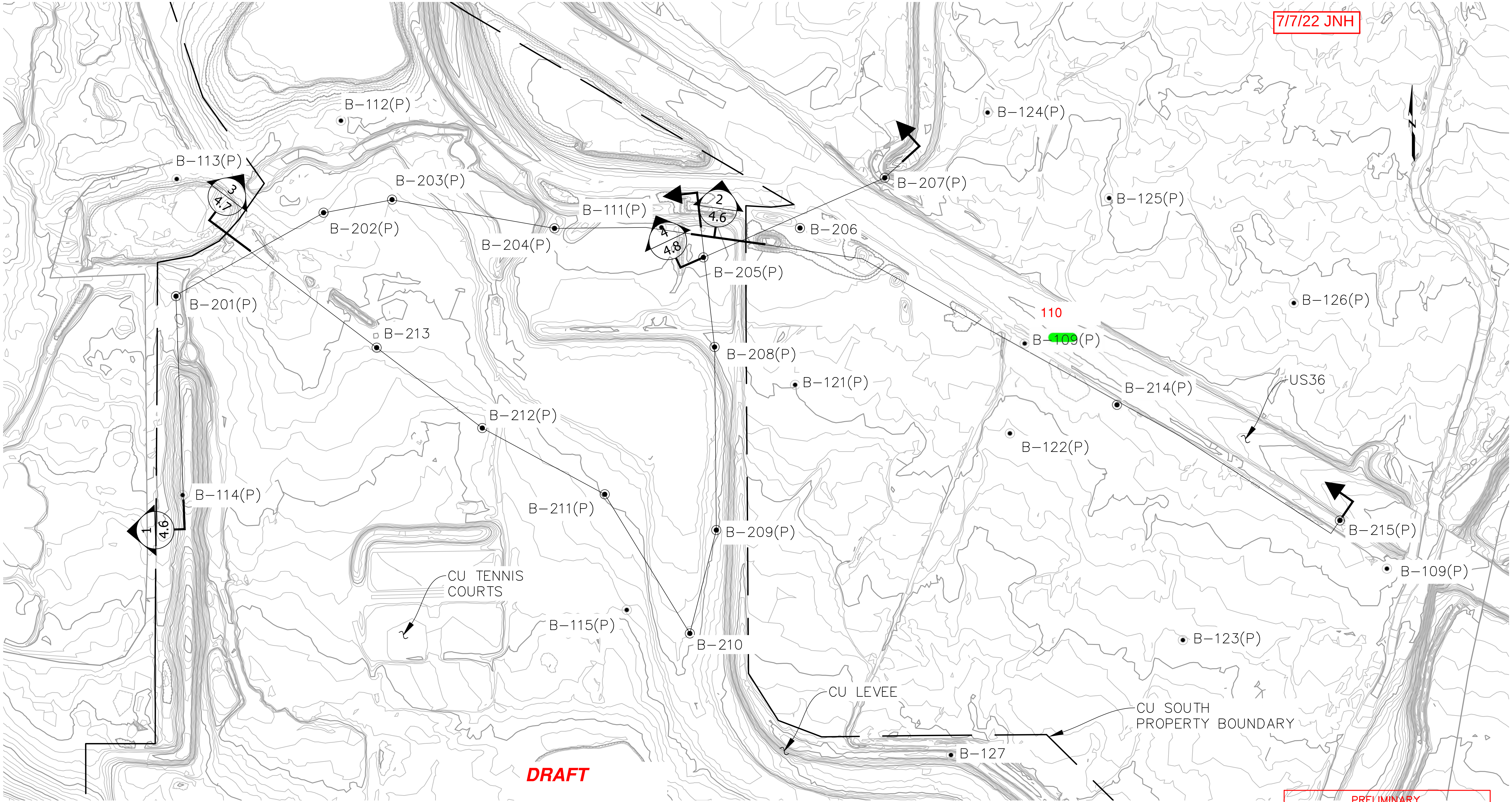

NOTE:
1. LIDAR DATA PROVIDED BY THE CITY AND COUNTY OF BOULDER AND FIELD SURVEY DATA PROVIDED BY FLATIRONS SURVEYING, INC.
2. FIGURE LEGEND AND GENERAL NOTES ARE ON FIGURE 2.2.

0 150 300 600
SCALE IN FEET

CITY OF BOULDER

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION - PHASE II
GEOTECHNICAL DATA REPORT

EXPLORATION LOCATION MAP

PROJECT NO. 16134

October 2018

Figure 2.1

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ATTACHMENT B

FLOWS BENEATH US36 BY HSU

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Attachment B
Simulated Flows Beneath US36

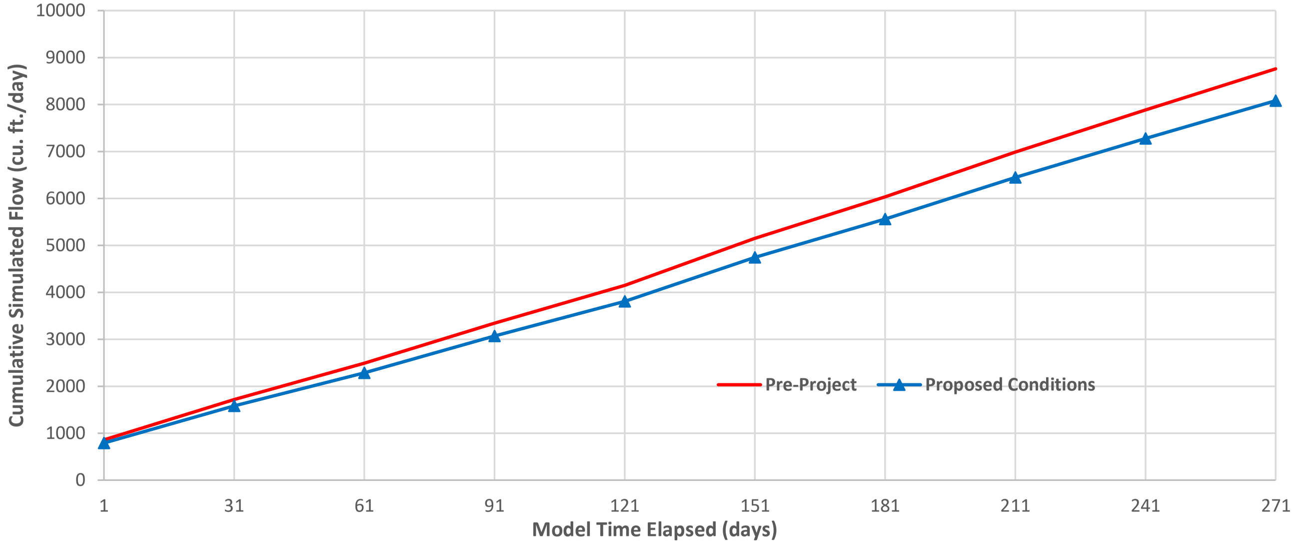
HSU 6

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

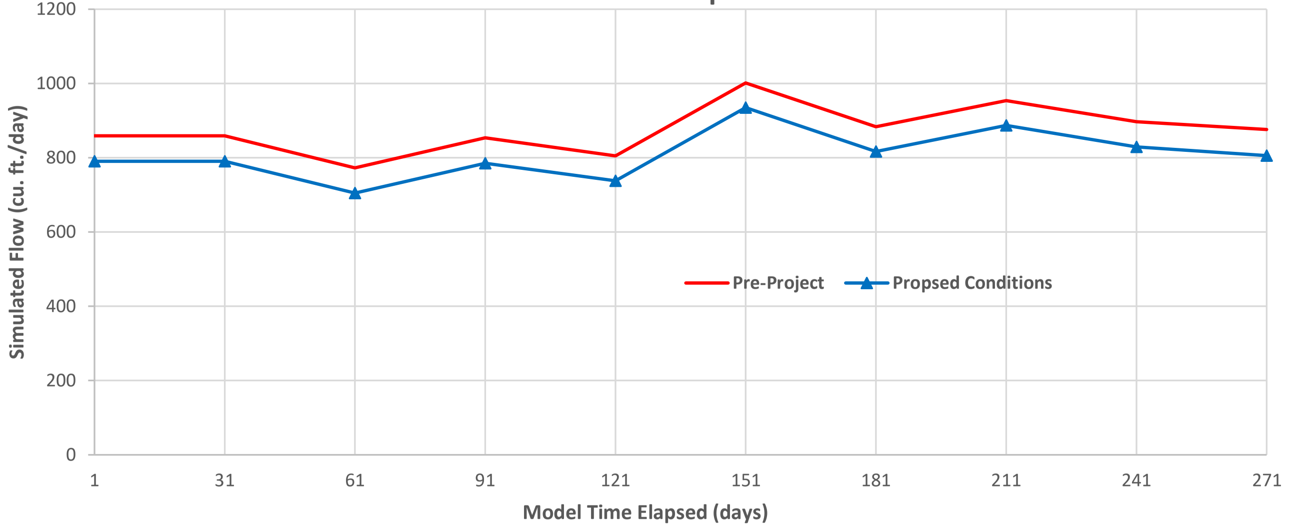
Proposed Conditions flows through HSU 6 are predicted to decrease by about 68 cu ft per day on average (about 8 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR

HSU 6 Cumulative Flow Comparison



HSU 6 Flow Comparison



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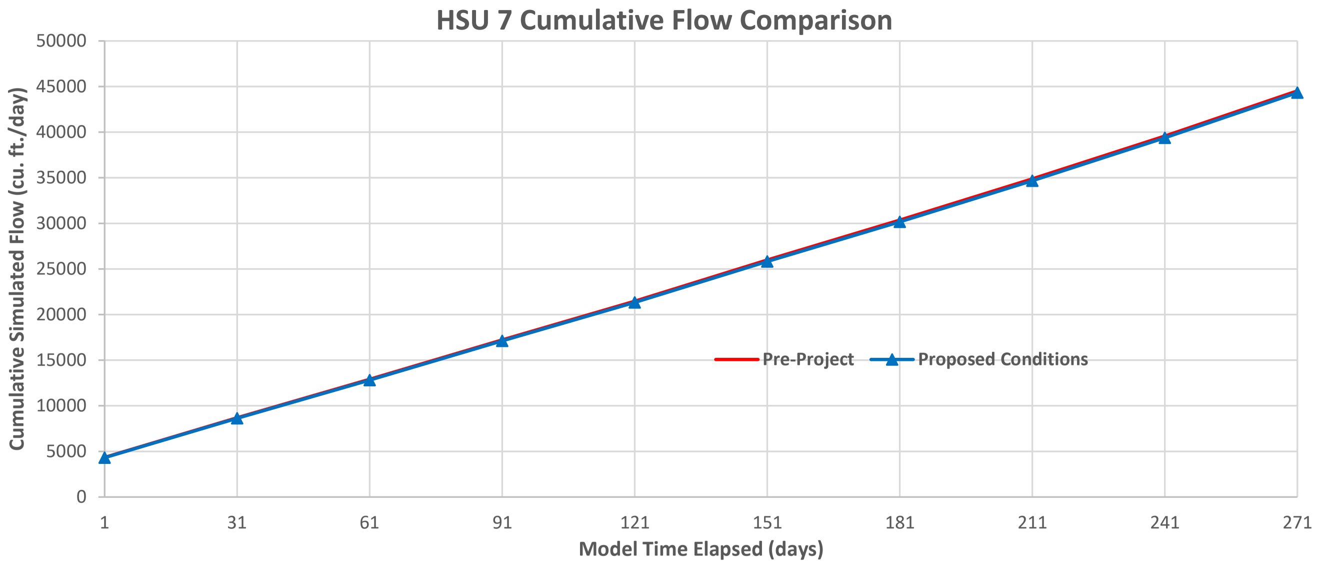
Attachment B
Simulated Flows Beneath US36

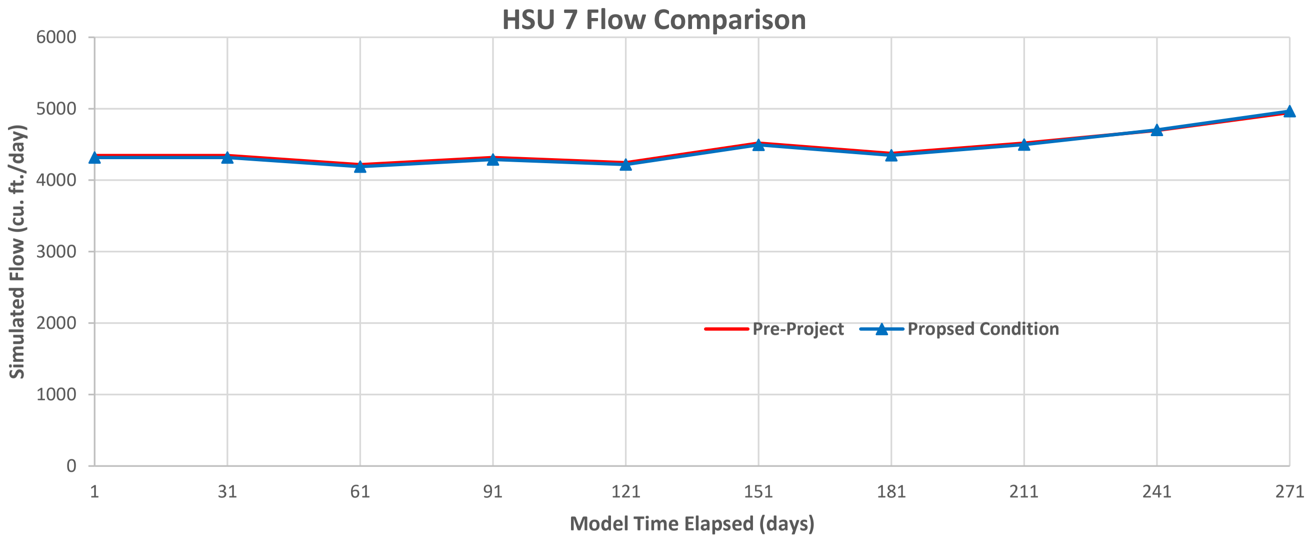
HSU 7

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 7 are predicted to decrease by about 18 cu ft per day on average (about 0.4 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR





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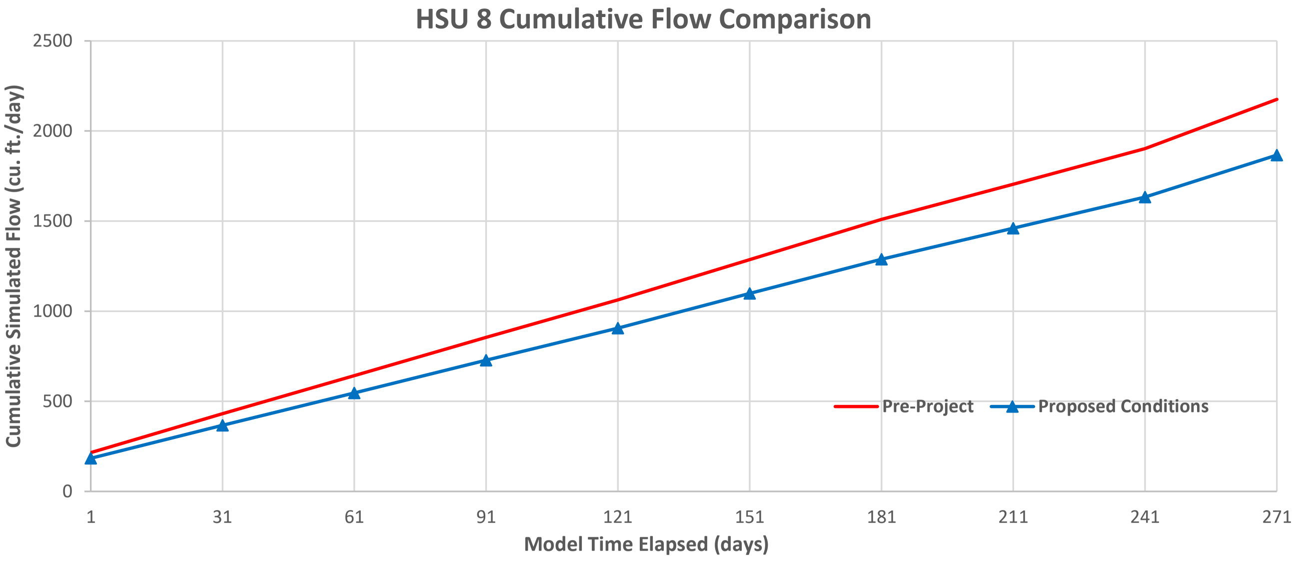
Attachment B *Simulated Flows Beneath US36*

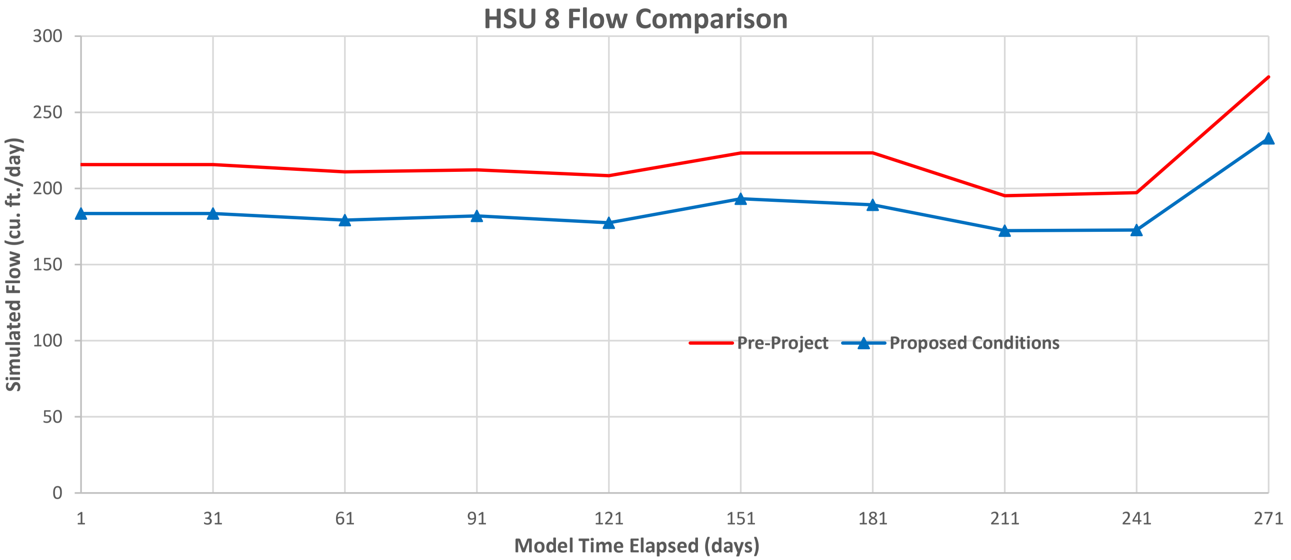
HSU 8

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 8 are predicted to decrease by about 31 cu ft per day on average (about 14 percent of average Pre-Project flow rates).

REDUCE IN COLOR


HSU 8 Cumulative Flow Comparison


HSU 8 Flow Comparison

7/7/22 JNH

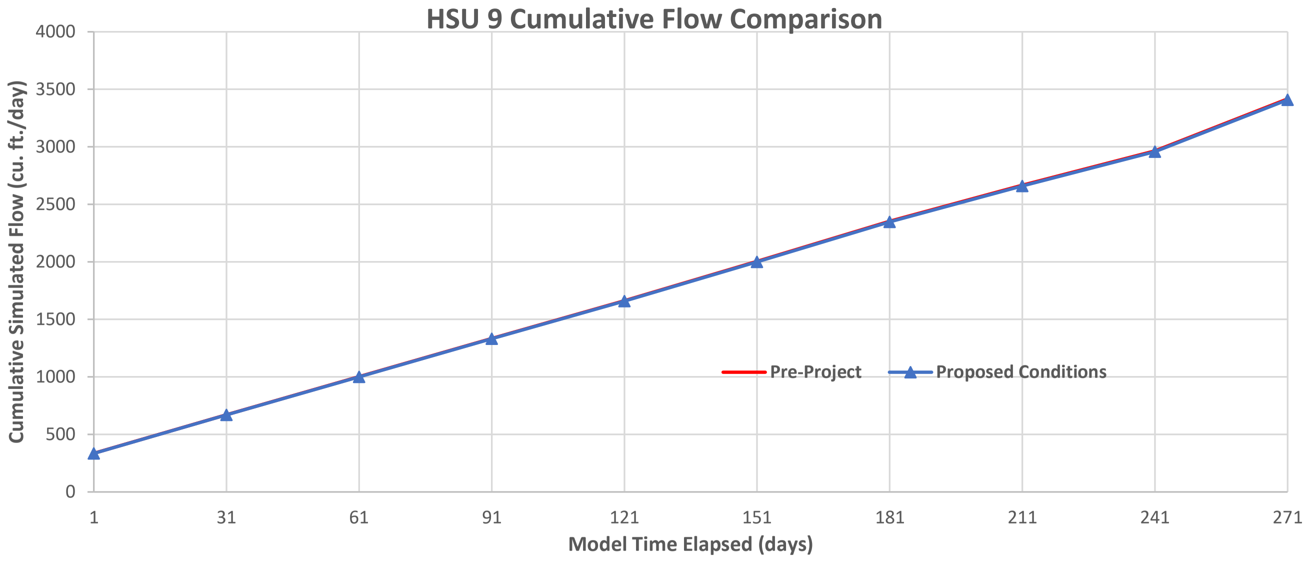
Attachment B
Simulated Flows Beneath US36

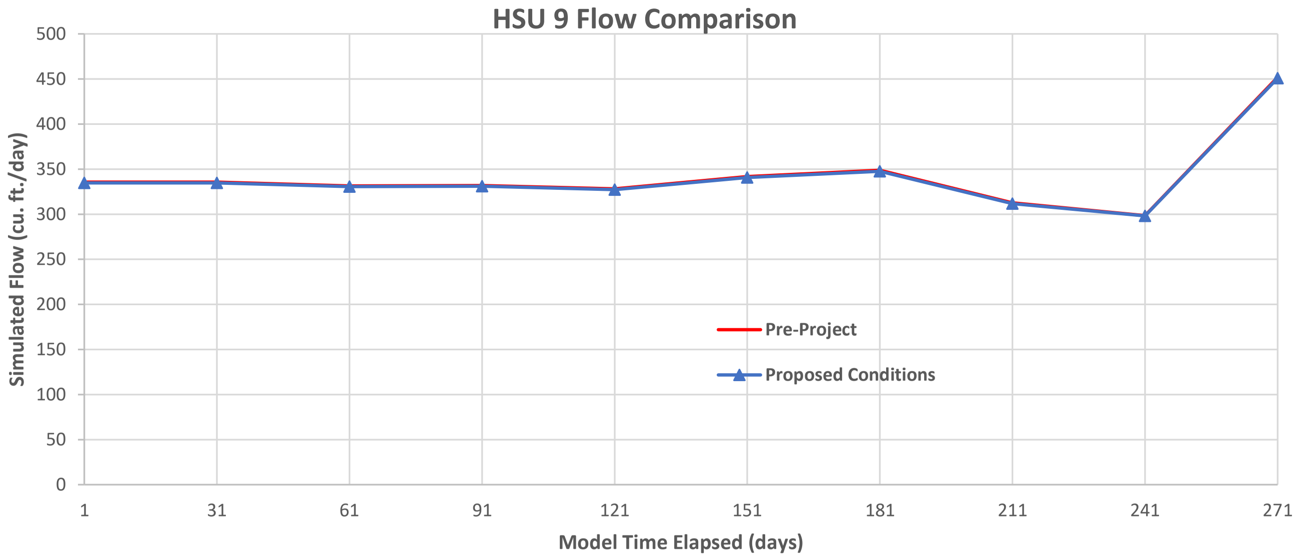
HSU 9

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 9 are predicted to decrease by about 0.8 cu ft per day on average (about 0.2 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR


HSU 9 Cumulative Flow Comparison


HSU 9 Flow Comparison

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ATTACHMENT C

CONVEYANCE SYSTEM PERMEABILITY (K) SENSITIVITY

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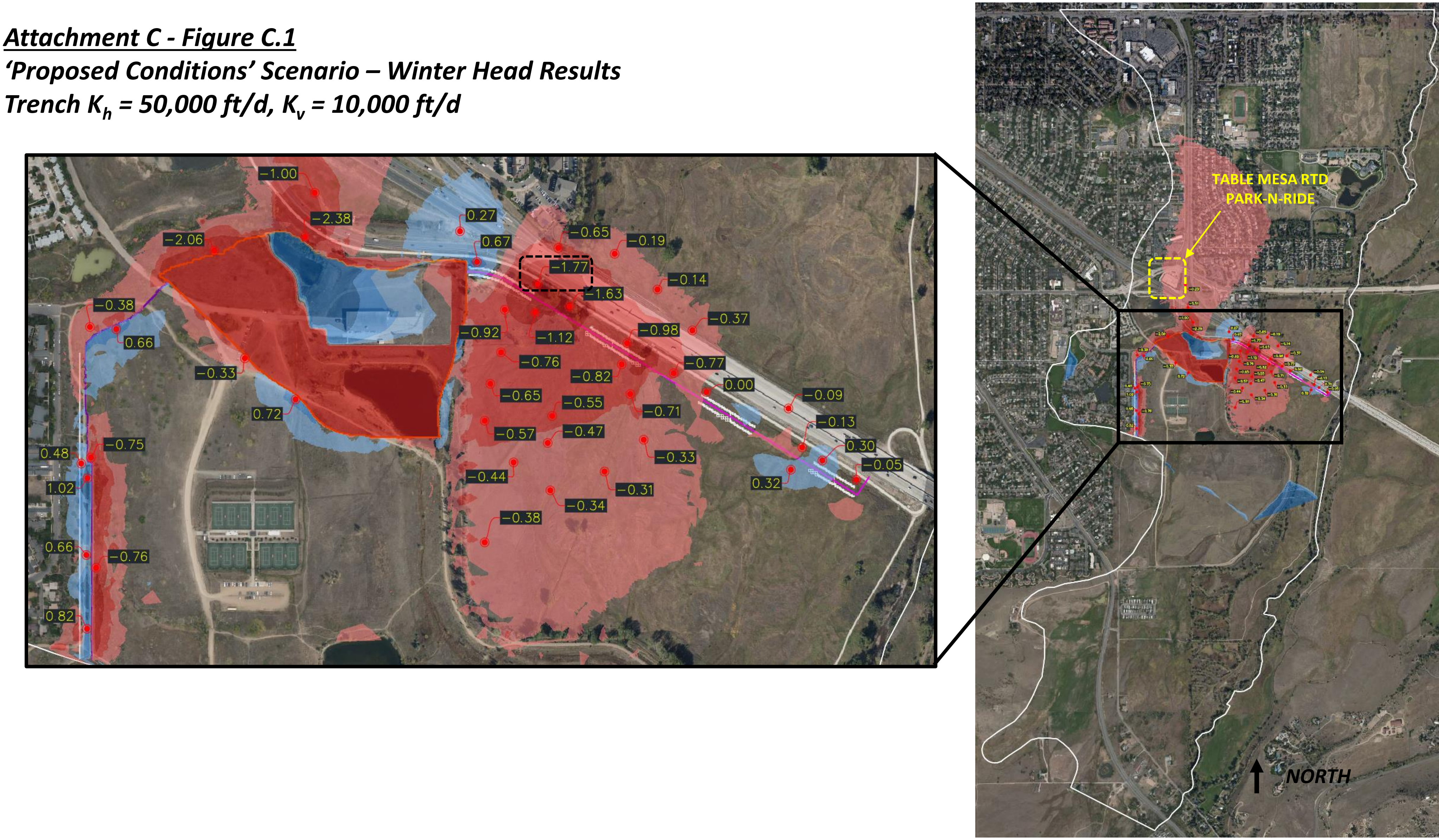
Attachment C - Figure C.1
'Proposed Conditions' Scenario – Winter Head Results
Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d



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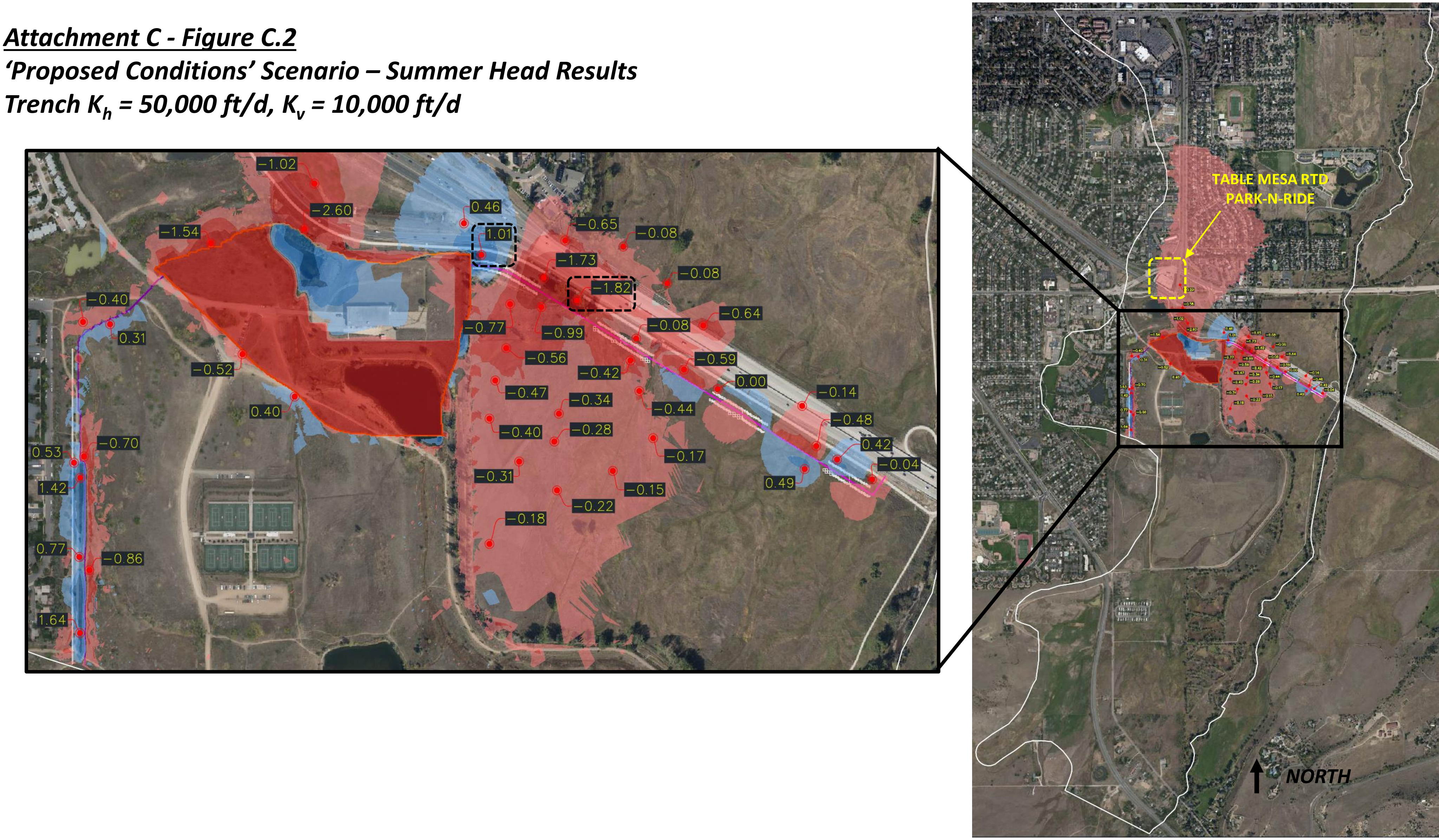
Attachment C - Figure C.2

‘Proposed Conditions’ Scenario – Summer Head Results

Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d



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Attachment C - Figure C.3

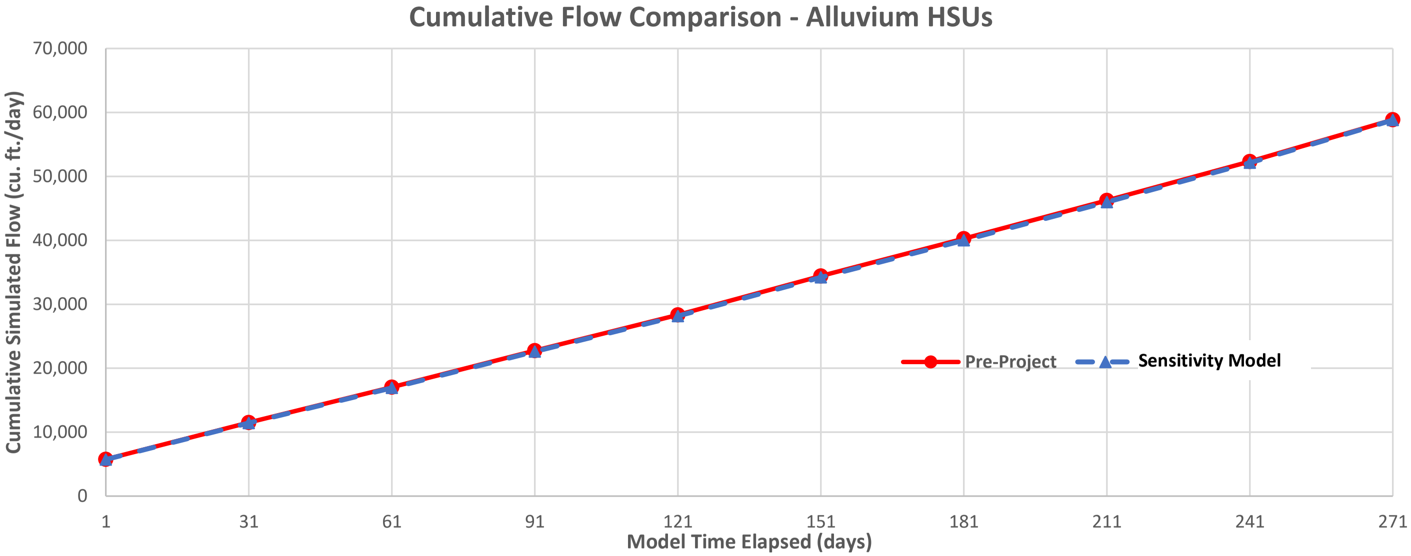
Simulated Flows Beneath US36 (Combined)

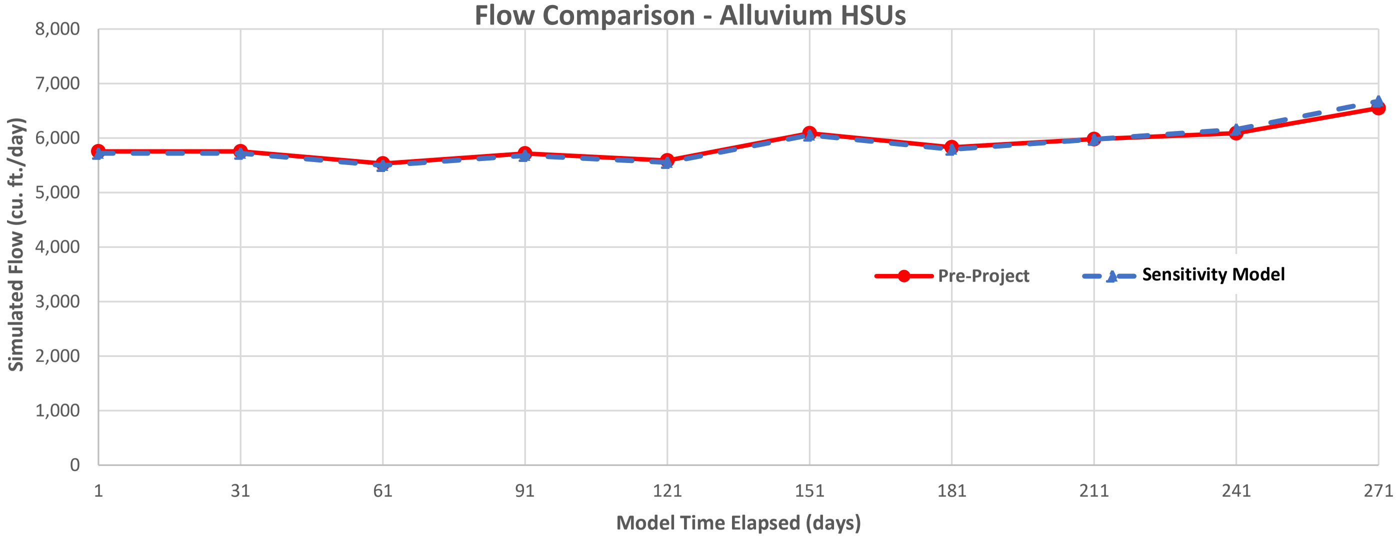
Trench K_h = 50,000 ft/d,
K_v = 10,000 ft/d

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through the alluvium HSUs are predicted to decrease by about 5 cu ft per day compared to Pre-Project flows (less than 1 percent of Pre-Project flow rates).

REPRODUCE IN COLOR





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7/7/22 JNH

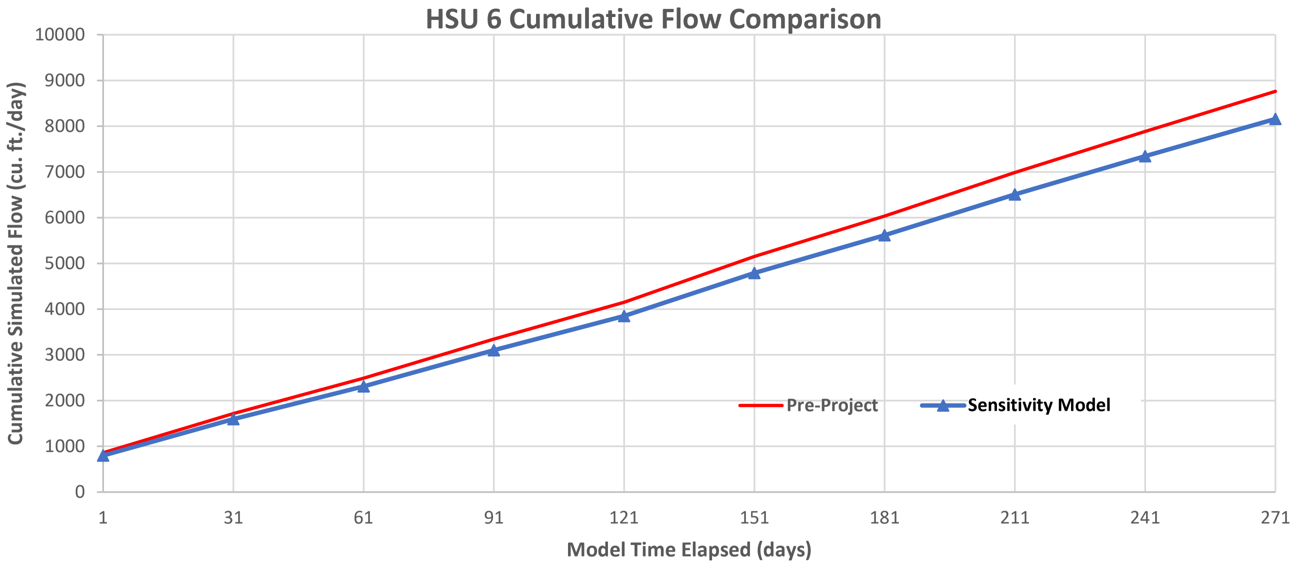
Attachment C - Figure C.4
Simulated Flows Beneath US36 Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d

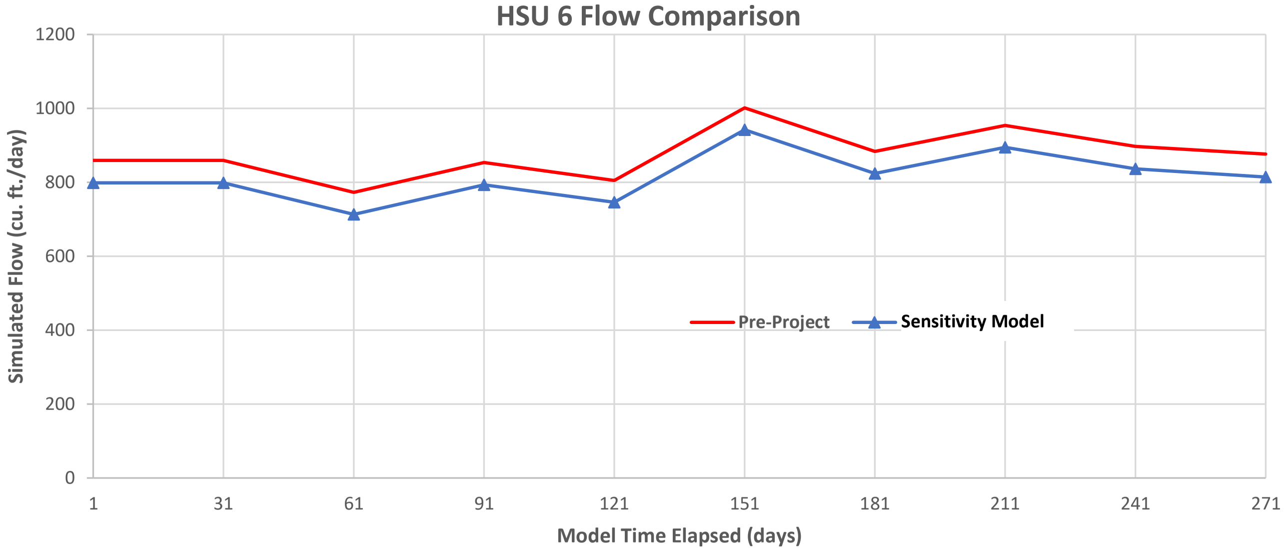
HSU 6

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 6 are predicted to decrease by about 60 cu ft per day on average compared to Pre-Project flows (about 7 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR


HSU 6 Cumulative Flow Comparison


HSU 6 Flow Comparison

7/7/22 JNH

Attachment C - Figure C.5

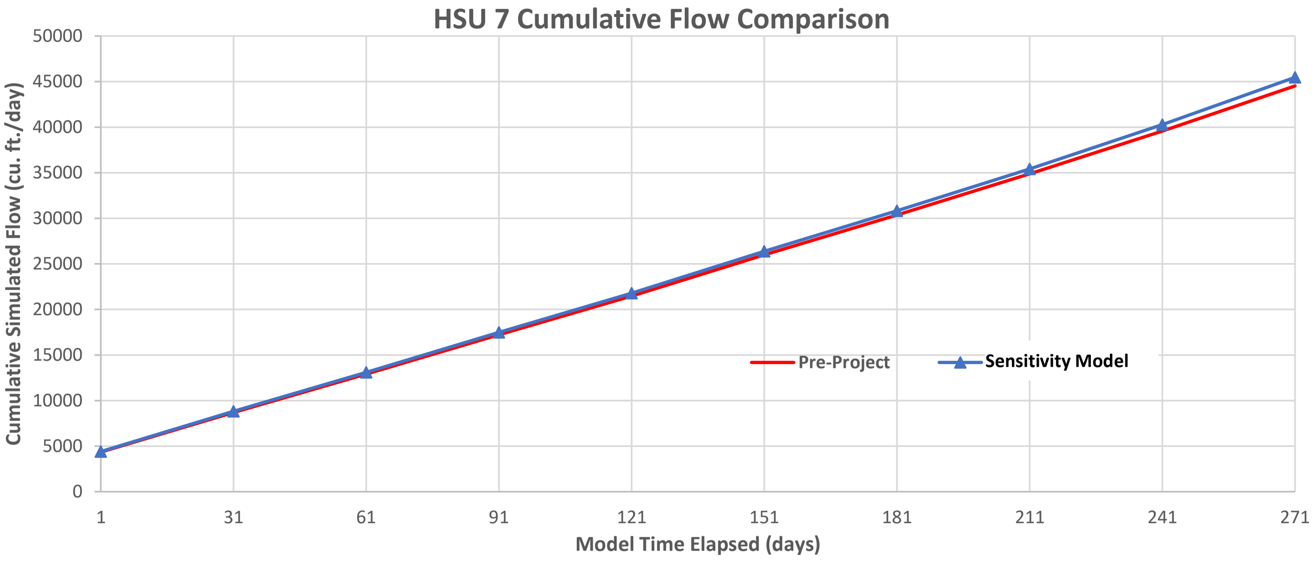
Simulated Flows Beneath US36 Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d

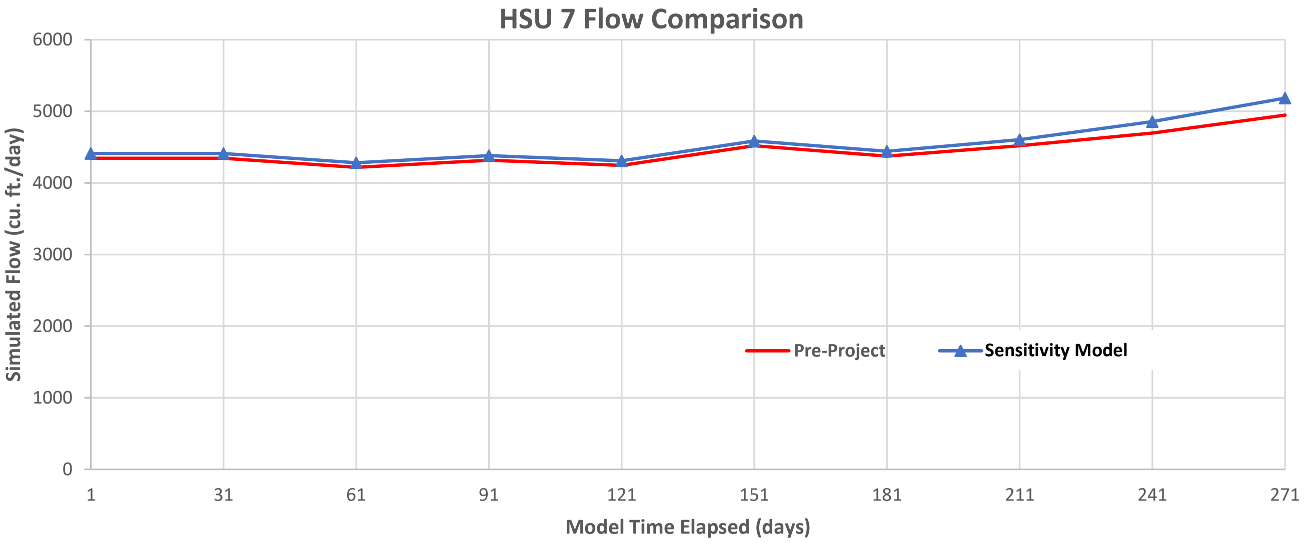
HSU 7

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 7 are predicted to increase by about 95 cu ft per day on average compared to Pre-Project flows (about 2 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR





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Attachment C - Figure C.6

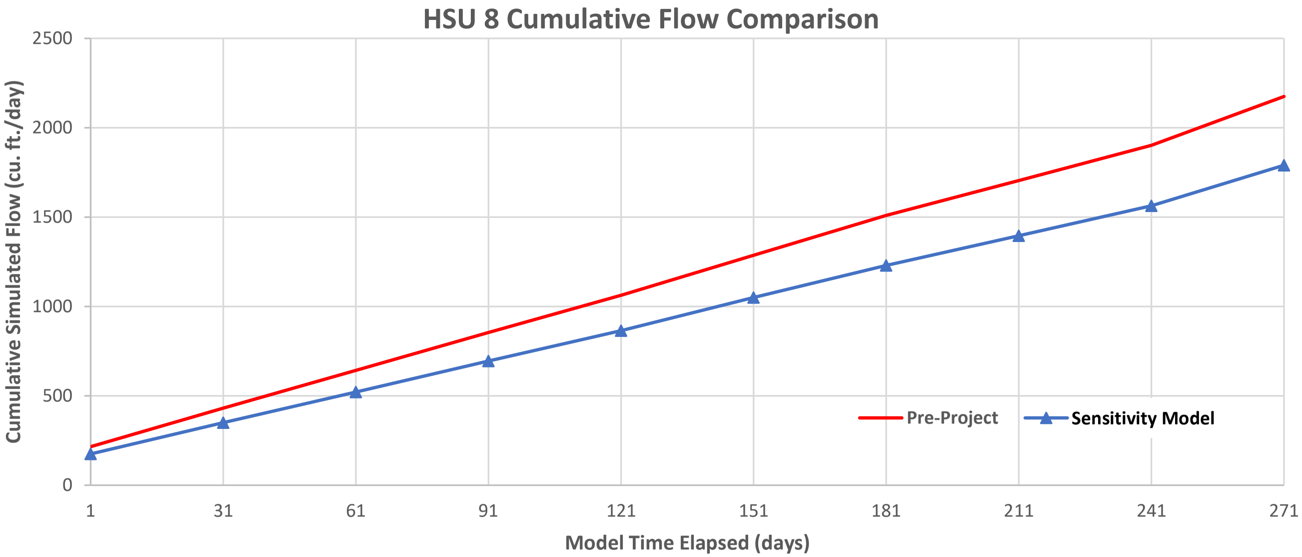
Simulated Flows Beneath US36 Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d

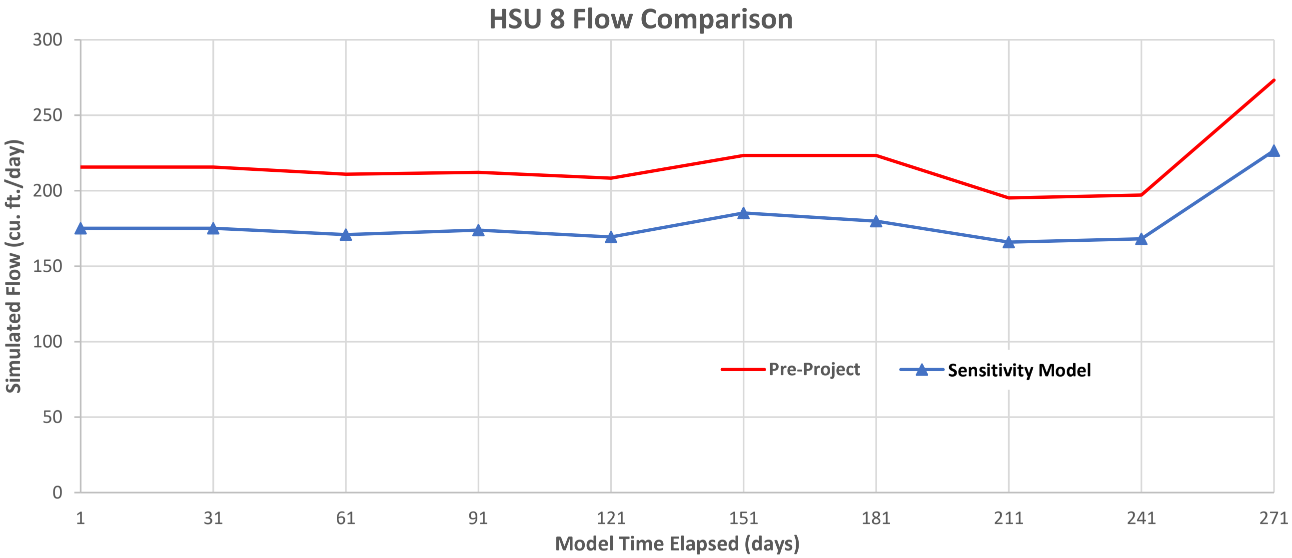
HSU 8

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 8 are predicted to decrease by about 38 cu ft per day on average compared to Pre-Project flows (about 18 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR





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Attachment C - Figure C.7

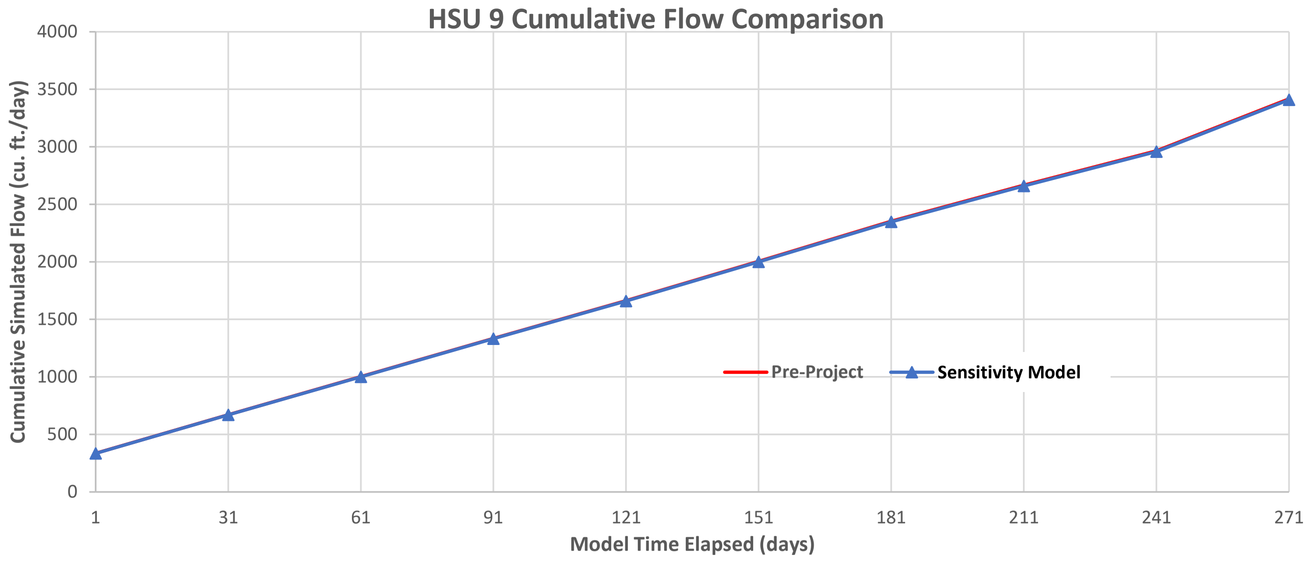
Simulated Flows Beneath US36 Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d

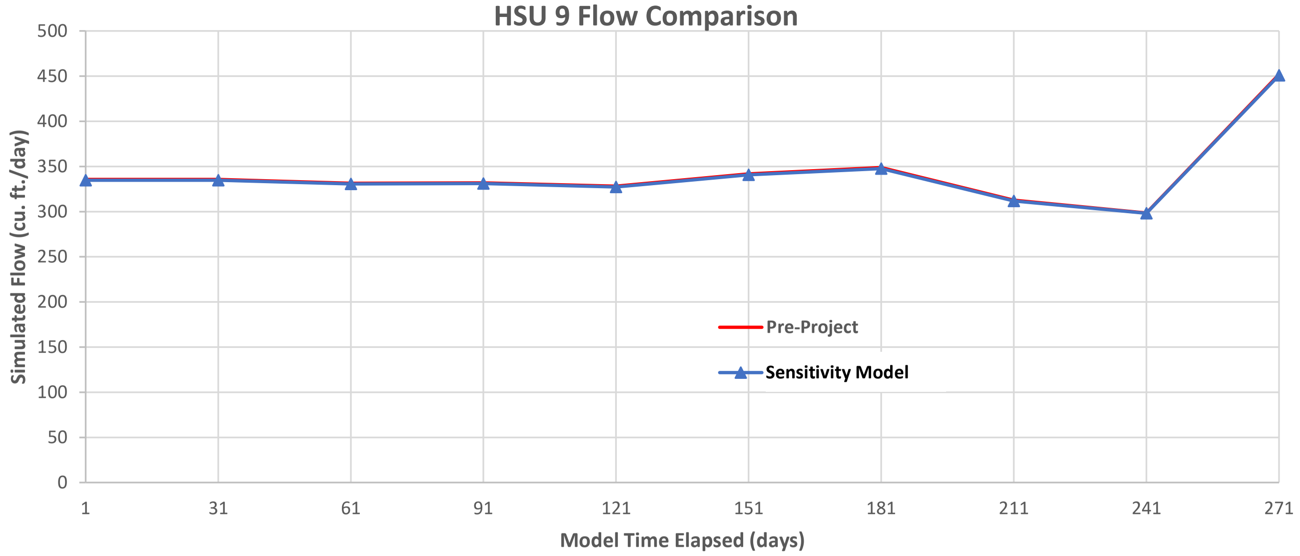
HSU 9

Note:
Flow values for Pre-Project and Proposed Conditions scenarios were extracted at the final timestep from each stress period and then linearly interpolated to provide a continuous timeseries.

Proposed Conditions flows through HSU 9 are predicted to decrease by about 0.8 cu ft per day on average compared to Pre-Project flows (about 0.2 percent of average Pre-Project flow rates).

REPRODUCE IN COLOR



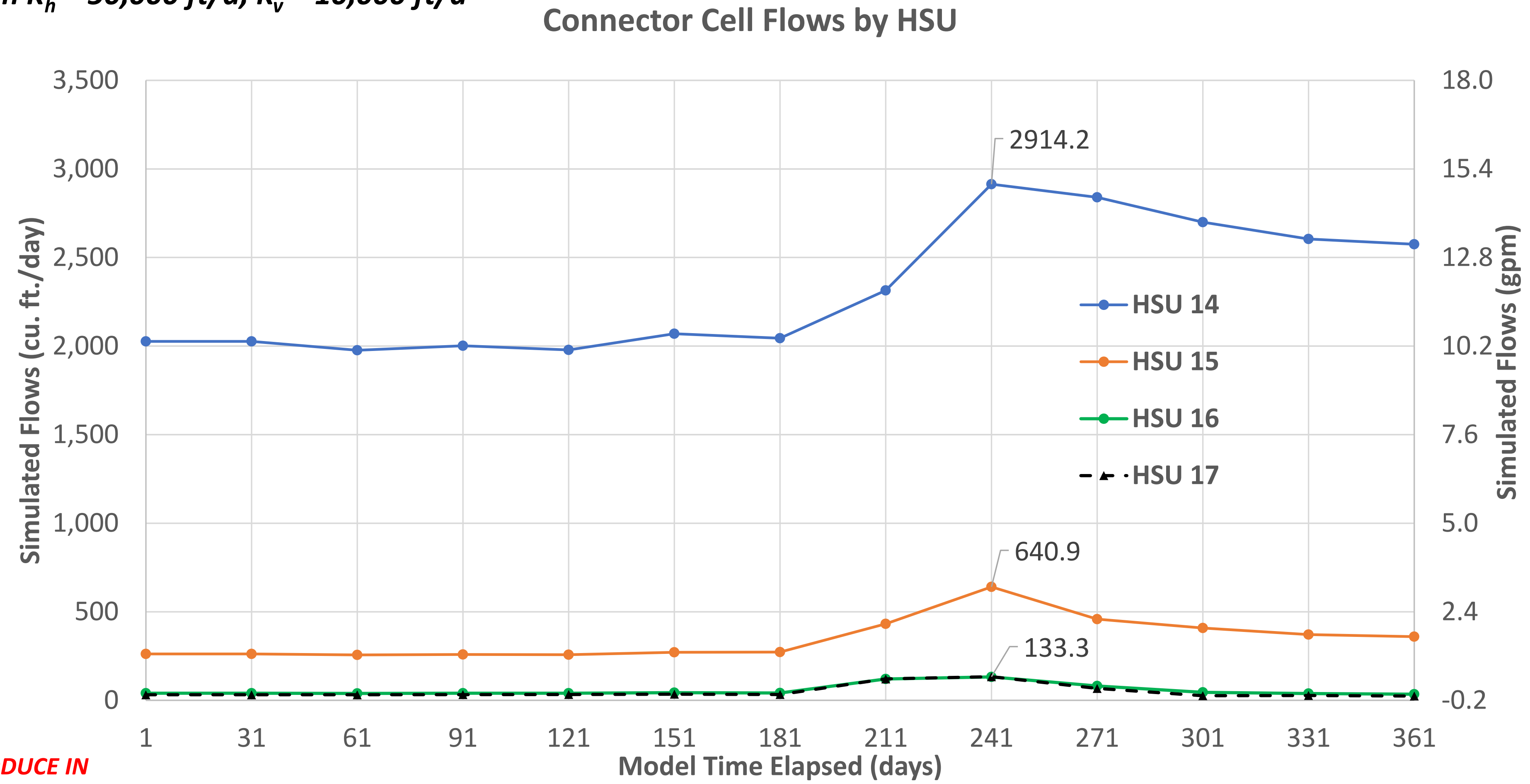


7/7/22 JNH

Attachment C - Figure C.8

Simulated Flows through Connector Pipe HSUs

Trench K_h = 50,000 ft/d, K_v = 10,000 ft/d



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APPENDIX F

TUNNEL FEASIBILITY EVALUATION

March 4, 2022
Project No. 16025

LITHOS ENGINEERING
2750 S. Wadsworth Blvd., Suite D-200
Denver, Colorado 80227
303.625.9502
www.LithosEng.com

RJH Consultants, Inc.
9800 Mt. Pyramid Court, Suite 330
Englewood, CO 80112

Attention: Mr. Eric Hahn, PE
Project Manager

Regarding: South Boulder Creek Regional Detention Project
Tunnel Feasibility Evaluation and Cost Estimate
Boulder, Colorado

Mr. Hahn,

This memorandum presents our opinion of tunneling for the subject project, including appropriate tunneling techniques, order of magnitude construction costs, and construction risks for conducting a crossing of US-36 in Boulder, Colorado. We understand that RJH Consultants, Inc. (RJH) is providing design services for the City of Boulder (City) to mitigate flooding in communities along South Boulder Creek. The tunnel is intended to drain floodwater retention ponds. The Project area is shown in Figure 1 below and in the attached annotated figures.

Figure 1: South Boulder Creek in the vicinity of US-36 and South Boulder Creek

Based on conversations with RJH and the City, Lithos understands the project design is likely to change pending City input and further civil, geotechnical, and hydraulic design. This memorandum is based on a review of the available geotechnical data from the geotechnical investigation conducted by RJH, subsurface conditions, preliminary project layout, and third-party impacts. Based on these parameters, this memorandum summarizes tunnel feasibility at the proposed crossing, as well as appropriate tunnel techniques and methods to reduce construction risk.

Plan and profile have not yet been developed beyond the conceptual level. Key tunnel assumptions include:

- Tunnel length of 630 LF from planned outlet work portals
- Inside tunnel diameter of 60 inches
- Approximate grade of 0.3% down from west-southwest to east-northeast
- Single-pass steel pipe installation, possibly lined post installation

This memorandum includes a discussion of the following subjects:

1. Summary of ground conditions
2. Tunnel construction considerations
3. Utilities / Layout considerations
4. Third party impact considerations
5. Feasibility of tunnel construction methods and associated risks
6. Preliminary opinion of probable construction cost

Memorandum Summary

A summary of our opinions is provided below:

1. There are no fatal design flaws to tunnel construction at the anticipated alignment using appropriate design/construction methodology; however, the presence of cobbles and boulders below the groundwater table present major risks to construction.
2. Ground conditions at the anticipated alignment are anticipated to consist of sand, gravel, cobble, and boulders. Tunneling will likely occur below the groundwater table.
3. Key risks for tunnel construction include over excavation resulting in settlement/sinkholes affecting US-36, and obstructions, including nested cobbles and boulders, which would prevent tunneling advance.
4. Recommendations to manage tunnel-related risks posed to third parties and overlying infrastructure include:
    a. Lower the tunnels at least one pipe diameter below top of bedrock. Bedrock provides a consistent excavation medium and thereby significantly reduces construction-related risks. We understand this recommendation may not be feasible due to a fixed downstream elevation.
    b. Reduce the tunnel drive lengths. Shorter tunnel drives allow for greater control over the tunnel excavation, thereby reducing construction risks. Drive lengths less than approximately 300 feet allow for additional tunnel techniques better suited to maintaining stability within the anticipated ground conditions. Additional shaft costs will be offset by more economical tunneling techniques and by risk reduction.
    c. Specify tunnel techniques with lesser construction risks, including:

        i. Pipe Ramming if tunnel drives are less than approximately 300 feet. Pipe ramming can displace cobbles and small boulders and can be utilized with an artificial plug to provide face stability. If line and grade requirements are critical with minimal deviation allowed, pipe ramming can be guided with specialized pilot rods equipped for ground conditions.
        ii. Shielded tunneling combined with dewatering and prescriptive specifications regarding face control and risk mitigation strategies. Lowering groundwater levels beneath the tunnel elevation may be difficult given the presence of shallow bedrock. Vertical dewatering wells may require close spacing, which is difficult to gain approval in the CDOT right of way. Challenging horizontal directional drains will likely be required with this technique, with strict qualifications required for the directional drain installer to ensure the drains are functional.
        iii. Excavation with a Microtunnel Boring Machine (MTBM) sized and equipped appropriately for cobbles and boulders. An appropriately sized MTBM would have an excavated diameter of approximately 96 inches or greater and be equipped with a cone crusher. This technique has high risk of oversized boulders halting tunnel advance.
    d. Implement a Geotechnical Baseline Report (GBR) for Tunnel Construction. A GBR is standard practice for managing and allocating risks associated with underground construction. Further information regarding a GBR will be including with future deliverables.
5. The proposed alignment intersects a secant pile wall and a soil-bentonite wall at a skew. While concepts will be furthered as design progresses, conventional approaches for these include the following.
    a. Secant Pile wall: Due to added complications for tunneling through concrete, we recommend preparing a portal at the secant pile wall prior to tunneling by cutting a hole in the secant piles. This would require a dewatered excavation or shaft at this location. Backfilling the excavation with a controlled low strength material can restore the impermeability and integrity of the secant pile wall. Secant pile structural design should seek to redistribute loads around the tunnel/ secant pile intersection.
    b. Soil-bentonite wall: all tunneling methods proposed can excavate through a soil bentonite wall with ease. A tunnel-based contact grouting program can restore the impermeability of the soil bentonite wall.
6. Third party requirements will require additional project coordination. Specifically, the Colorado Department of Transportation generally prefers steel initial support and crossing perpendicular to the roadway. Ditch operators generally require additional tunnel-based grouting. Both require instrumentation and monitoring for tunnel construction.
7. Preliminary opinions of probable construction cost for tunneling utilizing these techniques range between $2.1 million to $2.7 million.

1. Summary of Ground Conditions

Lithos Engineering (Lithos) performed a review of both the Phase I and Phase II site investigations completed by RJH and attended test pit excavation. Phase I data is presented in the “Phase I Geotechnical Report, South Boulder Creek Regional Detention Project” (RJH, 2019). The report presents findings from publicly available data including United States Geological Survey (USGS) maps, Colorado Division of Reclamation, Mining, and Safety (DRMS) records, Natural Resources Conservation Service (NRCS) Data, CDOT boring logs, and Boulder Open Space and Mountain Parks (OSMP) monitoring wells. The report also summarizes the site and subsurface conditions identified through the findings of a geotechnical site investigation including geotechnical borings, monitoring wells, field testing, and corresponding lab testing. The Phase II site investigation targeted boring locations to be in the immediate vicinity of the proposed tunnel and other critical structures, with three borings along the proposed alignment. In addition to the phased investigations, a test pit was excavated on December 15, 2021 in the vicinity of the proposed tunnel.

This memorandum primarily focuses on information derived from the Phase II site investigation borings and test pit excavation, while utilizing the Phase I report for general site context.

Project Setting

The Project is in Boulder, Colorado. The proposed alignment crosses US-36, approximately 1,000 feet southeast of the intersection of US-36 and S. Boulder Road, through OSMP property and a CDOT right of way surrounding US-36. The OSMP property contains high quality wetlands and federally listed threatened and endangered species habitat for the Preble’s meadow jumping mouse and the Ute Ladies’ tresses orchid. South Boulder Creek is approximately 1,500 feet east of the proposed alignment. The proposed alignment also crosses the 100- and 500-year floodplains, though a goal of the project is to modify these floodplains.

Ground Conditions Summary

The Phase II site investigation included three geotechnical borings near the proposed tunnels which ranged from 34 to 38 feet deep: B-205, B-206, and B-207. These borings generally encountered coarse alluvium between the ground surface and extending to depths between 12.5 and 17.5 feet below the ground surface. The coarse alluvium observed included:

- Poorly Graded Gravel with Clay and Sand (GP-GC)
- Poorly Graded Gravel with Silt and Sand (GP-GM)
- Silty Gravel with Sand (GM)
- Poorly Graded Sand with Clay and Gravel (SP-SC)
- Poorly Graded Sand with Silt and Gravel (SP-SM)

The maximum particle size observed in a sampler was 1.5 inches, however there were multiple instances of reported auger grinding and chatter, slow drilling rates, and the rig rocking back and forth as a result of ground material resistance. Cobbles were observed in the cuttings of B-205 in the first foot of drilling. The drill rig response to advancing through the coarse alluvium suggests larger gravels, cobbles, and boulders are present, as confirmed during test pit excavation. Bedrock of the Pierre Shale formation was encountered in all three borings at depths between 12.5 and 17.5 feet below the ground surface, to the maximum extent of drilling. Groundwater was encountered in all three borings at depths between 4 and 8 feet. Generalized ground conditions are tabulated below.

Table 1: Subsurface Conditions Encountered in Tunnel Borings

Boring	Depth (ft)	Material	Groundwater Depth (March 2020)
B-205 (West)	0 - 2	Poorly Graded Sand with Silt and Gravel (SP-SM)	8.0
	2 - 4.5	Poorly Graded Gravel with Silt and Sand (GP-GM)	
	4.5 - 12	Poorly Graded Sand with Silt and Gravel (SP-SM)	
	12 - 14	Poorly Graded Gravel with Silt and Sand (GP-GM)	
	14 - 17.5	Poorly Graded Gravel with Clay and Sand (GP-GC)	
	17.5 - 38	Pierre Shale Bedrock	
B-206 (Central)	0 - 12.5	Poorly Graded Sand with Silt and Gravel (SP-SM)	4.5
	12.5 - 34	Pierre Shale	
B-207 (East)	0 - 2	Poorly Graded Sand with Clay and Gravel (SP-SC)	8.0
	2 - 11.5	Poorly Graded Sand with Silt and Gravel (SP-SM)	
	11.5 - 14.5	Silty Gravel with Sand (GM)	
	14.5 - 35	Pierre Shale Bedrock	

Cobble and boulder frequency, distribution, size range, composition, and abrasiveness were estimated during test pit excavation. Throughout the test pit, a significant proportion of cobbles were observed with a maximum dimension between 3 inches to 4 inches. Lesser quantities were observed with dimensions up to 12 inches. Boulders measuring greater than 12 inches were also noted, with the largest boulder encountered measuring 22 inches. Cobbles were frequently clast supported (“nested”) whereas boulders were isolated within a silty sand matrix. Boulders were primarily well rounded, strong with unconfined compressive strengths likely to range between 15,000 psi to 35,000 psi, and highly abrasive to tunneling machinery. Cobble and boulder properties affecting tunneling will be further quantified in future deliverables.

For preliminary constructability purposes, the tunnel alignment will encounter wet sand, gravel, cobbles, and boulders. Near the center of the alignment, the tunnel may skim weathered shale bedrock. We anticipate the ground within the tunnel profile will behave as fast raveling ground above the groundwater table and as flowing ground below the groundwater table in accordance with general tunneling classifications for ground behavior. In general, these classifications require full contact support.

The assumed Project alignment is approximately 1,500 linear feet from the present South Boulder Creek channel. Creeks have historically meandered, changed course, and flooded throughout history, incising the underlying shale and depositing both fine (clay and silt) and coarse (sand, gravel, cobble, and boulder) materials within the Project area. Ground conditions adjacent to fluvial systems and near mountains can be highly variable, therefore care should be taken to verify the range of anticipated conditions as well as to minimize risk to the Project’s success.

2. Tunnel Construction Considerations

The following are common tunnel construction methods for this type of installation. Possible methods for the current alignment include, 1) MTBM and 2) shielded tunneling combined with dewatering. If the alignment is split into two drives, pipe ramming is a viable technique with less risk and cost than the previous options. Allowable and unallowable tunneling techniques are generally listed in the specifications and a GBR. Further discussion of each in association with Project specific geotechnical risk, utilities / layout considerations and impact to third parties are discussed in Section 5.

Suitable Techniques with Current Alignment:

1. Microtunnel Boring Machine (MTBM)
    a. This technique uses a pressurized rotating cutting head to excavate ground. Ground support is provided by a jacked pipe or with erected support, and with a pressurized bentonite-water slurry at the excavation face to counter earth pressures.
    b. This technique is a conservative approach applicable for many ground conditions (including wet conditions), tunnel diameters between 24 – 120 inches.
    c. The practical maximum length of this technique between shafts is approximately 2,000 feet.
    d. This technique provides limited access to the excavation face. Obstructions (with nominal dimensions approximately 1/3 the diameter of the machine, or steel), utilities, and cemented ground represent significant construction risk. It would be prudent to upsize the MTBM diameter appropriately for boulders and specify equipment that can crush cobbles and boulders. The MTBM has some ability to displace or break apart larger boulders at the cutting head; however, nested boulders and cobbles can still prove challenging. If the machine fails to do so, a rescue shaft or dewatered tunnel may be required to salvage the machine.
    e. This technique requires an extensive temporary staging area up to 1/4 acre for the launch shaft and 1/8 acre for the receiving shaft.
    f. This technique is often significantly more expensive and schedule intensive than other techniques.
2. Shielded Tunneling / Hand mining with Full Alignment Dewatering
    a. Excavation occurs by hand-tools and/or with a hydraulic excavator arm at the head of the machine. Ground support is provided by a jacked pipe or with erected support in addition to removable mechanical barriers at the face.
    b. This technique is technically viable; however, full alignment dewatering may be difficult to achieve with shallow bedrock inhibiting groundwater drawdown and CDOT constraints. Horizontal directional drains may be considered to avoid CDOT right of way.
    c. Applicable for the anticipated tunnel diameters between 48 to 168 inches when ground conditions are dry or dewatered, contain fines, within bedrock, or the technique is combined with surface-based dewatering and ground improvement to promote stability.
    d. The practical maximum length of this technique between shafts is approximately 2,000 feet unless erected/stationary ground support is chosen.
    e. This technique provides full access to the excavation face. Obstructions, utilities, and cemented ground are easily accessed for removal, provided a stable excavation face.
    f. This technique requires a larger staging area than auger bore or pipe ram. However, staging area requirements are significantly less than with microtunneling.
    g. Sand shelves, breasting boards, sandbags, injected grout or foam, other mechanical barriers, and shotcrete are often applied at the face to prevent excess material inflow.

Suitable Techniques with Adjusted Alignment:

1. Pipe Ram
    a. This technique involves ramming a steel casing across the alignment prior to excavating material from within the casing. As such, it is a favored technique for third parties as there is lessened risk of overexcavation and settlement.
    b. This technique may see line and grade deviation up to 2% when unguided. A guided pilot tube installation, and upsizing(s) can be required by project specifications to reduce deviation if required for hydraulic performance; however, the specifications will require specialized pilot tubes ground containing boulders.
    c. Applicable for diameters between 12 to 120 inches, when ground conditions have less than approximately 10 feet of groundwater head above the alignment.
    d. The practical maximum length of this technique at these diameters is approximately 300 feet.
    e. This technique is challenging to advance through a full face of bedrock or concentrated boulders.
    f. Technique requires minimal layout requirements and equipment.
    g. Technique has possible vibratory impacts and concerns for third parties.
    h. If significant groundwater is encountered, a loss of face stability may cause over excavation that could result in immediate or delayed settlement of above features such as utilities and roadway. Specifications may require for the contractor to establish artificial plugs within the casing to promote face stability.

Unsuited Common Techniques:

1. Guided Auger Bore
    a. This technique uses a rotating auger flight with a cutting head to excavate ground. Ground support is provided by a jacked pipe.
    b. Applicable for the anticipated diameters between 8 to 72 inches when ground conditions are dry or have a significant fines content with less than approximately 5 feet of groundwater head above the alignment.
    c. The practical maximum length of this technique is approximately 450 feet.
    d. It is challenging to advance augers through strong bedrock, cobbles, and boulders without special tooling (Robbins Small Boring unit or similar).
    e. A steel casing, guided pilot tube installation, and multiple upsizing(s) can be required.
    f. Technique requires minimal layout requirements and equipment.
    g. If significant groundwater and sand/gravel are encountered, a loss of face stability may cause over excavation that could result in immediate or delayed settlement of above features such as utilities and roadway.
2. Tunnel Boring Machine (TBM)
    a. This technique uses a rotating cutting head to excavate ground. Ground support is provided by a jacked pipe or with erected support.
    b. Applicable for the anticipated tunnel diameters between 24 to 120 inches, when ground conditions are dry, contain fines, within bedrock, or the technique is combined with surface-based dewatering and ground improvement.

c. The practical maximum length of this technique between shafts is approximately 2,000 feet unless a liner-plate support is chosen.
d. This technique provides good access to the excavation face. Obstructions, utilities, and cemented ground are easier to remove, provided a stable excavation face is present. Nested cobbles, boulders, and other obstructions can create instability as the rotating cutterhead attempts to ingest materials.
e. This technique requires a larger staging area than auger bore or pipe ram. However, staging area requirements are significantly less than with microtunneling.

3. Utilities / Layout Considerations

When utilizing tunnel construction methods, launch and receiving shafts or portals are required on either side of the crossing. Shafts, if used in lieu of portals, are sized based on the casing, final pipe and construction method as discussed within each method in Section 2 above. For the Project, ample space is available on the southwest upstream end and limited on the northeast downstream end. A shaft is feasible on the west side of US-36 and either a shallow shaft or portal is viable on the east side. Tunneling favors mining uphill to allow for easier muck removal and water drainage; as such, it is likely tunnel contractors will choose to launch mining equipment from the east side and receive at the west side. If project constraints dictate tunneling downhill, tunneling cost may rise between 10% and 30% due to lowered efficiency and operational adjustments. The launch location is a high-traffic area with significant equipment, trucking, and noise throughout the duration of tunnel construction; whereas the receiving location is generally active only for tunnel equipment recovery and final outlet structures.

A gravity feed system will require careful line and grade control to ensure proper hydraulic function. As such, the tunneling method used must be capable of accurate steering to ensure specified line and grade requirements. With the exception of pipe ramming without a guidance system, the methods proposed above are capable of steering provided precautions are taken for potential bedrock and boulders. Precautions may include requiring steering equipment near the head of the excavating equipment.

Except for the secant pile wall discussed herein, the proximity to other utilities, structures, avoidance zones, and nearby work do not appear to conflict with feasibility of a tunnel crossing of US-36. It is assumed that staging area availability for tunneling equipment and materials – e.g. pipe storage, spoils, dewatering discharge, etc. can be provided near launch and receiving sites. Utility considerations will be revisited upon creation of a project plan and profile by RJH. Appropriate recommendations to monitor and protect existing utilities and infrastructure will be provided in future specifications and drawing details.

4. Third party Impacts

Third parties impacting the location of the tunnel include the Regional Transportation District (future construction), University of Colorado – Boulder, CDOT, Dry Creek Ditch No. 2, and OSMP, among others. Presently, RJH is addressing third party constraints to situate not only the tunnel, but the detention pond(s) and other auxiliary features.

Beyond obtaining landowner permissions and easements, other tunneling impacts may include the use of a steel casing versus other types of materials (e.g. fiberglass pipe), crossing the CDOT alignment perpendicular, additional contact grouting near the tunnels and stringent instrumentation and monitoring plan. Future project specifications and drawing details will include minimum instrumentation and monitoring plan requirements.

The alignment also crosses beneath Dry Creek Ditch No. 2. Ditch crossings often require negotiation during design and permitting with the ditch's engineering advocate. Some key points to expect with negotiation:

- Risks to the ditch include settlement, overexcavation, and, if a slurry type system is used, inadvertent fluid returns (frac out).
- Risks are lessened for launching tunnel equipment uphill as there is better excavation control closer to the launch portal.
- Risks can be mitigated proactively by performing a pre-excavation surface-based permeation grouting program. In general, this involves using angled holes to inject sodium silicate or similar chemical grout.
- Ditch operators often request seepage cutoff barriers and flowfill collars on either side of the ditch. This is sometimes accomplished with additional tunnel-based contact grouting near the ditch.
- Ditch operators may require full time construction observation and monitoring and instrumentation. These are common components to tunnel construction.
- Ditch operators often prefer construction occurs when the ditch is not running water. This is preferred by tunnel contractors as well because there is less seepage into the tunnel. In addition, should damages occur, there are more economical repair options.
- Ditch operators often request ditch improvements in the guise of repairs, including lining the ditch with an impermeable lining.

5. Feasibility of Tunnel Methods

Table 2 looks at the tunnel construction methods discussed in Section 2, while assigning a risk associated with ground conditions, utilities/layouts and third-party impacts.

Table 2: Preliminary Risk and Feasibility of Tunnel Construction Methods for US-36 Crossings:

Tunnel Method	**Suitability 1 = low 10 = high**	**Project Specific Risk (High, Moderate, Low)**			**Project Feasibility**
		Ground Conditions, i.e. Groundwater, Nested Cobbles and Sand	Utilities / Layout	Third party impacts	
Pipe Ram (Split Drive)	8	Moderate	Low	Low	Drives under 300 LF are viable in ground conditions. Prescriptive specifications will help manage alignment and settlement risks. If line and grade deviation up to

Tunnel Method	Suitability 1 = low 10 = high	Project Specific Risk (High, Moderate, Low)			Project Feasibility
		Ground Conditions, i.e. Groundwater, Nested Cobbles and Sand	Utilities / Layout	Third party impacts	
					2% cannot be allowed, then specialized tooling will be required to set alignment.
MTBM	5	High	Low	High	May be feasible with large upsize. Design will need to incorporate elements to manage cobble/ boulder risks.
Tunneling Shield / Hand Mining*	5	High	Low	High	May be feasible, but requires a large diameter casing pipe, face control, and dewatering.
TBM*	3	High	Low	High	May be feasible with dewatering, but higher risk than tunneling shield / hand mining as a rotating cutterhead makes it difficult to remove nested cobbles boulders in a safe fashion
Pipe Ram (Single Drive)	1	High	Potential Fatal Flaw	High	Not feasible as one 670 LF drive, high risk with the anticipated ground conditions and potential impact to CDOT, possible heave of roadway.
Guided Auger Bore*	1	High	Potential Fatal Flaw	High	Not feasible due to ground conditions, length, and diameter.

**Requires dewatering*

Colors denote recommended approach: suitable (green); marginal (yellow); poor/fatal flaws (orange)

Microtunneling

It is our opinion at this initial phase and with the documents provided by others that excavation by MTBM with a jacked steel casing is the most viable tunneling approach for the current alignment. However, nested cobbles and boulders present a high risk for tunnel excavation. A MTBM should be sized to at least 3x the largest boulder diameter and equipped to crush/ingest cobbles/boulders. Design and contractual documents should clearly baseline the volumetric percentage, distribution, and other properties of the cobbles and boulders to manage risks of a differing site condition claim or damage to the MTBM during mining. In underground construction, these aspects are often included in a project tunnel GBR.

Design considerations for microtunneling include the following. These will be addressed in tunnel specifications and details.

- Jacking loads imposed on the casing pipe. The casing pipe should be sufficiently thick to accommodate imposed jacking loads. An uneven distribution of loads or using a thinner pipe than necessary can result in a skewed alignment and/or damaged pipe. In severe cases, damaged pipe can lead to a total tunnel loss and emergency procedures – e.g. rescue shafts - to recover equipment. Often rescue shafts are not permitted within right-of-way, which can lead to the MTBM having to be abandoned, or rescued with a dewatered hand tunnel from the other direction.
- MTBM cutterheads are often oversized relative to the casing pipe to reduce frictional forces on the casing pipe and allow the MTBM to advance with less jacking thrust. The overcut creates an annulus between the ground and the outer surface of the casing pipe. In a tunnel of the sizes required for this project, the annulus should be limited to a maximum of 0.75 inches radially. It is recommended to contact grout the annulus to reduce the potential for subsequent settlement of the ground surface above the crossing.
- Over-excavation and uncontrolled ground are most common at the entry and exit points, or eyes, of the shafts. These failures can propagate along the casing pipe and create voids well behind the excavation face. Entry and exit eye seals, properly positioning the machine against the face, and continuous monitoring of machine parameters by an experienced and qualified MTBM operator will help to contain ground and slurry, and prevent over-excavation.
- MTBMs require significantly more ancillary equipment than most other trenchless techniques and a larger construction work area footprint. The layout for a guided pipe ram project is generally large enough for a single crane or excavator with haul attachments, an equipment trailer, pipe laydown, lubricating materials, and generators. An MTBM project will need to accommodate these items as well as additional equipment, including a slurry separation plant, an operator's control box, slurry lines, generators, and additional equipment trailers.

Tunneling Shield with Dewatering

A tunneling shield with hand mining, combined with a carefully implemented dewatering program, is also a technically viable approach. However, dewatering may be difficult to achieve with shallow bedrock and US-36 right-of-way access restrictions. Ground improvement or reinforcement may be needed to prevent sinkholes if flowing sands or nested boulders are present in high quantities below the highway. Ground reinforcement may be implemented from the surface, if access is available, or from within the casing (e.g. probe holes and grouting) with restricted injection pressures so as not to heave overlying infrastructure.

Design considerations for a tunneling shield include:

- Jacking loads imposed on the casing pipe. As with MTBM and further described above, the casing pipe should be sufficiently thick to accommodate imposed jacking loads.

- It is difficult to draw down groundwater in an unconfined aquifer adequate for construction when shallow bedrock is near the tunnel invert. Closely spaced well-points, deep wells within bedrock, and horizontal wells may prove marginally effective at lowering the groundwater table to create a stable excavation face. Any dewatering plan should be thoroughly vetted prior to construction to address suitability for tunnel construction and the potential for roadway settlement.
- The shield excavation face should have a means to close off and stabilize the face should over excavation occur. These may include mechanical barriers, spiling and grout injection at the face, shotcrete, sand bags, and other means. Breasting boards and sand shelves should be utilized during mining to limit material inflow.
- As with MTBM mining, the annulus should be limited to a maximum of 0.75 inches radially. It is recommended to contact grout the annulus to reduce the potential for subsequent settlement of the ground surface above the crossing.

Pipe Ramming

If the design is modified to accommodate tunnel drive lengths under approximately 300 feet, guided pipe ramming is the preferred tunneling technique with least overall project risk. Traditional pipe ramming does not use guidance systems and is preferred for the encountered ground conditions if 2% line and grade deviation is acceptable with hydraulics. Guided pipe ramming utilizes a 4-inch pilot tube to set line and grade prior to ramming across progressively larger steel casings. The pilot tube would have to be specialized to excavate difficult ground and provides insight into ground conditions along the alignment prior to tunnel excavation. For instance, if the pilot tube cannot advance due to a large boulder, then it is unlikely the tunnel casing will advance. At this point, the tunnel might be re-positioned, and the pilot tubes reattempted with minimal third party risk. After the pilot tube in installed, the steel casing is rammed across at the same line and grade and without an overcut annulus. To prevent a larger steel casing from breaking the connection with the pilot tube, an intermediate steel casing sized between 16 inches and 24 inches may be driven part way or the full length of the tunnel. After the steel casing is across, the excavated material within the casing is removed with internal augers or hand mining. Overall, this system presents the lowest risk to third parties since the tunnel support is advanced prior to ground excavation.

Design considerations for a guided pipe ram include:

- Specifying a pilot tube (if required) and casing pipe cutting shoe appropriate for coarse alluvium material.
- Evaluating dynamic loads on the steel casing pipe. Lithos will perform an analysis to specify the minimum steel casing thickness for ramming loads and long-term performance. A pipe ram with sufficient capabilities for cobbles or boulders is likely to be of the Grundoram Apollo class.
- Specifying establishment of a plug within the casing at the start of tunneling. The start of tunneling is where settlement is most likely to occur as not enough material has collected within the casing to counteract face pressure. An artificial plug can be created using sand bags or low-strength materials. In addition, shaft dewatering is likely to alleviate groundwater pressure.
- Splitting the drive such that the pipe ram does not have to tunnel through the secant piles. The secant piles can be incorporated into the launch shaft design, prepared with a launch portal, and the construction joints sealed at the conclusion of tunneling to prevent seepage. A bi-directional

launch shaft can be created at the secant pile wall, allowing for a 300-foot drive under US-36 and a 330-foot drive under OSMP property.

- Specifying vibration monitoring and infrastructure pre-condition assessments along with other conventional tunnel instrumentation and monitoring to manage or prevent claims associated with construction.

Other tunneling techniques are considered unviable with project fatal flaws due to ground conditions, and tunnel length restrictions which would necessitate an intermediate shaft in the highway right of way.

Other Considerations

Lubrication

Dewatering within the launch and receiving pits is anticipated to be necessary and viable for the ground conditions based on our current knowledge with site materials. A contractor should be prepared to lubricate the exterior of the casing pipe continuously during advancement, as cobbles can exert enough pressure and friction to prevent casing advancement. A comprehensive detailed design should look at jacking forces with and without lubrication as well. Lubrication generally consists of a non-toxic mixture of bentonite and water.

Risk Reduction by Lowering Alignment

Lithos believes that lowering the alignment to be completely within bedrock, if hydraulically viable, will reduce risk to third parties at a nominal cost of deeper shaft excavations and hydraulic accommodations to ensure drainage. Unless the top of bedrock can be verified through additional subsurface exploration or geophysics, we recommend at least one pipe diameter bedrock cover as a buffer.

Spillway Foundation

The tunnel is currently planned to intersect the spillway foundation secant piles. As design progresses, it will be important to coordinate these aspects to ensure the tunneling technique can mine through the secant piles and not compromise the structural capacity and seepage retention. Though not optimal, MTBM and shielded tunneling can excavate through unreinforced concrete whereas a pipe ram cannot.

Practical guidelines include no steel reinforcement in the secant piles where the tunnel intersects, preparing a portal through the secant piles by cutting a hole in the concrete with an intermediate shaft prior to tunneling, and including additional casing pipe contact grout ports with a prescriptive tunnel-based contact grout program to provide a consistent impermeable barrier after tunneling.

Soil Bentonite Wall

The tunnel is currently planned to intersect a soil bentonite wall. In general, soil bentonite walls prevent little tangible difficulty for tunnel excavation. However, design usually includes additional casing pipe contact grout ports with a prescriptive tunnel-based contact grout program to restore the impermeability of the barrier.

Casing Pipe Materials

In general, CDOT requires steel encasements or support under their roadways. The techniques listed herein favor steel casing pipe. Two approaches are commonly utilized. The first involves multi-pass tunneling, in which a steel casing sized approximately 96 inches is installed via tunneling. After, a 60-inch carrier pipe (HDPE, FRPM, or other) are inserted within the casing and the annulus between carrier pipe and casing pipe filed with a high mobility low strength grout to eliminate cathodic protection, corrosion, or other concerns. The second technique involves single pass construction, in which a steel or, outside of CDOT right of way, FRPM casing pipe also serves as the carrier pipe. The casing can be slip or spray lined after install to improve hydraulic performance and increase lifespan. Cost benefits are often dictated by supplier pricing with either approach. Casing thickness is likely to be ¾ inch minimum for pipe jacking or 1 inch minimum for pipe ramming, pending contractor means and methods.

Lithos does not recommend use of reinforced concrete pipe (RCP) for either casing pipe or carrier pipe in tunneling applications over approximately 300 linear feet. RCP quality control is generally insufficient for tunneling applications.

Field Engineering

Finally, the project owner should consider retaining a firm qualified and experienced in performing construction phase services, including construction observation to assess ground conditions as tunneling occurs. These construction phase services would also include instrumentation and monitoring of the ground surface. These field observations during construction allow the city's team and contractor to efficiently develop and modify tunneling procedures appropriate to the encountered ground conditions.

6. Preliminary Opinion of Probable Cost

The table below presents preliminary opinions of costs for tunnel construction based on Lithos' experience with general market values for similar construction. The opinions of costs are subject to change with a further understanding of project constraints and requirements. The costs include construction shafts, dewatering, excavation, installation of casing pipe, and contact grouting of the casing pipe. Costs do not include re-lining a steel casing, if necessary, for corrosion or hydraulic performance or installation of a carrier pipe if different from the casing.

Table 3: AACE 18R-97 Class 5 Estimated Costs for Trenchless Construction (60-inch Capacity)

Crossing	Construction Shifts[1]	Approx. Length (LF)	Unit Cost[4] ($/LF)	Total Cost
Shielded Tunneling	65	630	3,400	$2.1 M
MTBM[2]	25	630	4,300	$2.7 M
Pipe Ram[3]	35	1x 300 1x 330	3,000	$2.1 M

[1] *Excavation and initial support only. 12-hour shifts.*

[2] *MTBM sized to 96-inch diameter to better process boulders*

[3] *Guided Pipe Ram drive lengths assume a bi-directional launch shaft at the portal. A 300-foot drive will be conducted under US-36 and a 330-foot drive will be conducted under OSMP property. Tunneling costs are approximately $1.2 million for just the US-36 segment.*

[4] *Costs include mobilization, shafts/portals, tunnel excavation, steel casing installation, contact grout, dewatering as necessary, instrumentation and monitoring, and 20% overhead and profit.*

> *Steel casing can be lined after tunneling and corrosion treated, or a carrier pipe can be installed at additional cost. Notable omissions include water treatment and disposal, final structures (e.g. manholes or outlet portals), contingency, or secant pile portal preparation.*

Closing

Based on our preliminary review of the site and information available to date, there are no fatal flaws which would preclude the use of a properly designed tunnel construction method(s) at the site proposed within this memorandum. Sufficient laydown is available surrounding the site for construction operations and RJH is presently navigating third party constraints. The geologic desktop studies suggest ground conditions consist of sand, gravel, cobbles, and boulders above and below the groundwater table; thus, three approaches are viable: 1) jacking a MTBM at high cost but reduced risk, 2) jacking a tunneling shield with hand mining excavation and horizontal dewatering at lower cost but higher risk, and 3) splitting the drives up and utilizing guided pipe ram at lower cost and risk. Other tunnel methods, with discussed risks, are presented above.

This memorandum constitutes our conceptual-level deliverable for the Project and is based on information provided to Lithos and our understanding of the Project to date. Should any aspect of the design plan, profile, or Project constraints change, Lithos should be consulted, and the information contained herein updated accordingly or incorporated into future drawings, specifications, or the GBR. When the project moves from feasibility to detailed design, additional geotechnical investigations will be needed by the tunnel design engineer to further the project understanding and reduce project risk. We appreciate the opportunity to work with you and look forward to helping the Project team reduce risk on this Project.

If you have any questions regarding the content of this letter, please contact the undersigned.

Sincerely,
Lithos Engineering

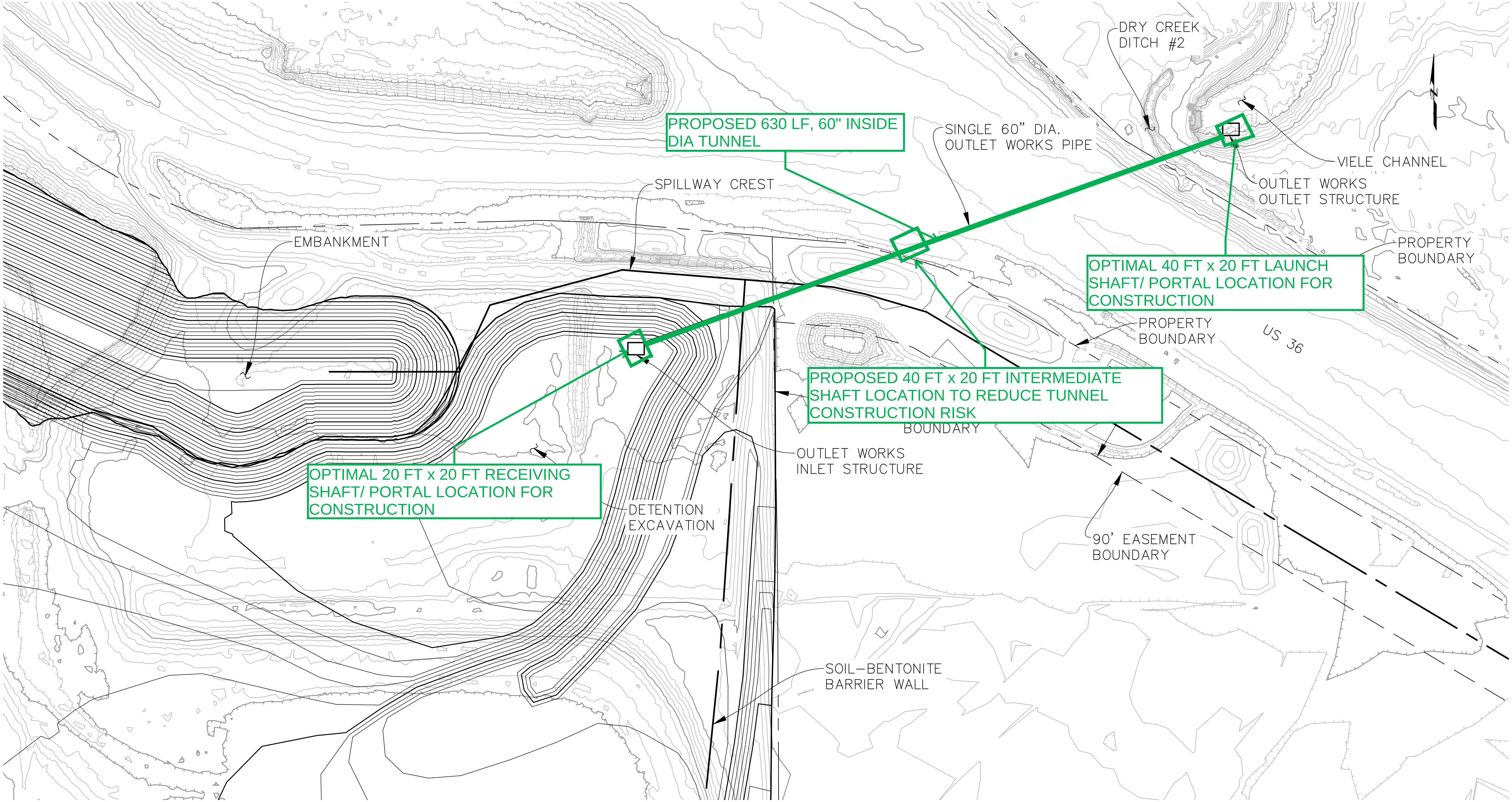
Nate Soule

Ryan Marsters, PE, PG
Senior Professional

Nate Soule, PE, PG
Vice President

ATTACHMENTS

1. Annotated tunnel markup of existing plan and profile



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RJH CONSULTANTS, INC.

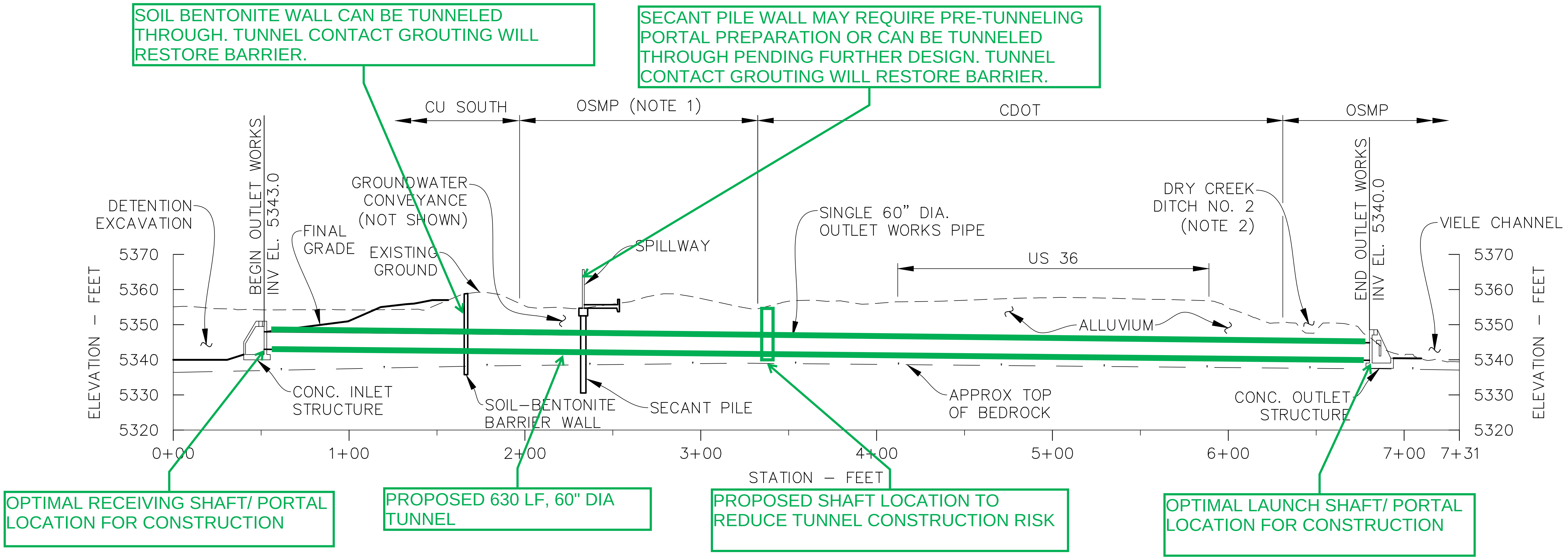
SOUTH BOULDER CREEK
REGIONAL DETENTION

OUTLET WORKS PLAN

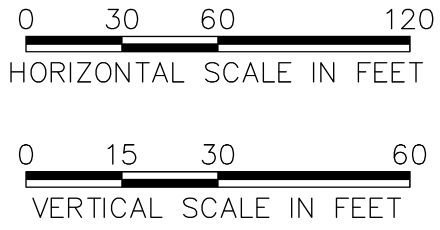
PROJECT NO. 16134

February 2022

Figure 1



NOTES:
1. THE OUTLET WORKS PROFILE CROSSES THE OSMP PROPERTY WITHIN THE 90–FOOT EASEMENT.
2. DRY CREEK DITCH NO. 2 IS AN EARTHEN CHANNEL.



RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK
REGIONAL DETENTION

OUTLET WORKS
PROFILE

PROJECT NO. 16134

February 2022

Figure 2

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APPENDIX G

SITE DRAINAGE ANALYSES

APPENDIX G.1

VIELE CHANNEL HYDROLOGIC EVALUATION (PMF)

RJH CONSULTANTS, INC.

TECHNICAL MEMORANDUM

Project 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Eric Hahn, P.E. - RJH Consultants, Inc.**

DATE: **July 2, 2021**

SUBJECT: **South Boulder Creek Regional Detention Project**
Viele Channel Hydrologic Evaluation

1.0 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present methodology and results of the Viele Channel hydrologic evaluation performed for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City).

2.0 Background

Viele Channel is urban stream that extends through the northwest portion of the Project site. The Project will include construction of an earthen embankment located south of Viele Channel on the CU Boulder South campus property. The alignment of the earthen embankment has generally been located so that the downstream toe of the embankment is about 50 feet away from the top of the right bank of Viele Channel.

A hydrologic evaluation of Viele Channel is required to identify peak flow rates in Viele Channel during various flood events. Results from the Viele Channel hydrologic evaluation will be used to evaluate the impacts of flooding in Viele Channel on key Project components.

3.0 Regulatory Criteria

The earthen embankment will be a jurisdictional dam that will be regulated by the Colorado Office of the State Engineer (SEO). The hydrologic evaluations presented in this memorandum have been prepared consistent with guidelines presented in the SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO Rules) (SEO, 2020) and SEO *Hydrologic Basin Response Parameter Estimation Guidelines* (HBRPEG) (SEO, 2008).

4.0 Basin Characteristics

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 earthen embankment with an approximately 90-foot-wide excavated earthen spillway through the right abutment.

Land cover data for the basin was obtained in GIS format from the National Land Cover Database (NLCD) 2016. A majority of the watershed consists of low to moderately dense residential developments (i.e., a tenth to quarter of an acre plots). State Highway 93 extends across the lower portion of the watershed.

Soils data was obtained from the Natural Resources Conservation Service (NRCS) online Web Soil Survey (WSS). Surficial soils predominantly consist of sandy loams and sands. Depth to bedrock exceeds 60-inches for most of the basin.

The watershed boundary was delineated using a U.S. Geological Survey (USGS) 1-meter Digital Elevation Model (DEM) imported into the ArcGIS computer program. The watershed was divided into two sub-basins based on the location of Viele Lake. The topographic sub-basin parameters used in the hydrologic analyses were calculated using ArcGIS and are provided in Table 2.1. A basin map is presented on Figure 1.

TABLE 2.1
SUB-BASIN PARAMETERS

Sub-basin	Area (mi^2)	L (mi)$^{(1)}$	L_{ca} (mi)$^{(2)}$	Maximum Elev. (ft)	Minimum Elev. (ft)	Slope (ft/mi)$^{(3)}$
Upstream	0.52	1.38	0.55	6,043	5,502	392
Downstream	0.60	1.73	1.06	5,560	5,343	125

Notes:
1. Length of longest watercourse.
2. Length along primary watercourse from sub-basin outlet to a point opposite of the centroid of the drainage basin.
3. Slope along the longest watercourse.

5.0 Precipitation

5.1 General

RJH developed probable maximum precipitation (PMP) estimates using the SEO's Regional Extreme Precipitation Study (REPS) Tool. PMP depths were evaluated for the General Storm and Local Storm. The General Storm represents a large area, long-duration storm event typically associated with a major synoptic weather feature. The Local Storm represents an intense, short-duration storm that typically occurs over smaller areas than the general storm.

The 10,000-year, 1,000-year, and 100-year rainfall depths were identified using the SEO's MetPortal tool. For each precipitation event, rainfall depths were identified for the 2-hour and 6-hour.

Precipitation depths for all storm events were adjusted by a factor of 1.07 to account for future climate change impacts based on requirements in the SEO Rules) (SEO, 2020).

5.2 Probable Maximum Precipitation

RJH used the REPS Tool to develop PMP depths for the 2-hour Local Storm, 6-hour Local Storm, 24-hour Local Storm, and 72-hour General Storm (SEO, 2020). The REPS Tool calculates PMP depths for numerous durations for the general storm and local storms for a user-entered GIS shapefile. Tool output includes basin average PMP depths during the General Storm and Local Storm, and temporal distributions for the General Storm and various types of Local Storms. Basin average PMP depths for the General Storm and Local Storms are presented in Tables 5.1 and 5.2, respectively.

TABLE 5.1
REPS TOOL BASIN AVERAGE GENERAL STORM PMP DEPTHS

Duration (hours)	Precipitation (inches)
1	7.3
6	14.5
12	16.0
24	22.5
48	24.7
72	24.9

TABLE 5.2
REPS TOOL BASIN AVERAGE LOCAL STORM PMP DEPTHS

Duration (hours)	Precipitation (inches)
0.083	1.3
0.25	3.5
1	8.9
2	12.6
3	15.0
4	16.6
5	16.6
6	18.1
12	23.8
24	24.0

5.3 Precipitation Frequency

RJH used MetPortal and the REPS Tool to estimate an annual exceedance probability (AEP) for each PMP storm and develop temporal depth distributions for more frequent events. MetPortal analyzes the following storms: Local Storm 2-hour Synthetic Storm, MEC 6-hour Front-Loaded Synthetic Storm, and MLC/TSR 48-hour Center-Loaded Synthetic Storm.

PMP depth estimates from the REPS Tool and a GIS shapefile of the drainage basin are entered into MetPortal, and the tool estimates a range of AEPs for each storm. Estimates of AEP for the PMP events are presented in Table 5.3.

TABLE 5.3
PMP AEP ESTIMATES

MetPortal PMP Storm	AEP Estimate
MLC/TSR 48-hour Center-Loaded Synthetic Storm	AEP < 10^{-7}
Local Storm 2-hour Synthetic Storm	AEP < 10^{-7}
MEC 6-hour Front-Loaded Synthetic Storm	AEP < 10^{-7}

RJH generated temporal depth distributions for the 10,000-year, 1,000-year, and 100-year precipitation events using MetPortal. A GIS shapefile of the drainage basin and recurrence interval of interest are entered into MetPortal, and the tool develops temporal depth distributions for each storm. Cumulative precipitation depths for the 10,000-year, 1,000-year, and 100-year precipitation events are presented in Table 5.4.

TABLE 5.4
METPORTAL CUMULATIVE DEPTHS

Storm	Recurrence Interval	Cumulative Precipitation (inches)
Local Storm 2-hour Synthetic Storm	10,000-year	4.9
	1,000-year	3.7
	100-year	2.6
MEC 6-hour Front-Loaded Synthetic Storm	10,000-year	5.7
	1,000-year	4.2
	100-year	2.9

6.0 Hydrologic Parameters

6.1 General

Rainfall-runoff modeling requires the input of several hydrologic parameters including loss rate and unit hydrograph parameters. Hydrologic parameters were developed in general accordance with the HBRPEG (SEO, 2008). The hydrologic parameters selected for the Viele Channel watershed are described in the following sections.

6.2 Rainfall Losses

The portion of rainfall that does not contribute to runoff is lost to interception, evaporation, surface retention, and infiltration. RJH used the Green-Ampt method, which is also the method recommended by the SEO for evaluation of rainfall losses on urban or developed drainage basins. The Green-Ampt method simulates rainfall losses as a two-phase process: surface retention losses (initial abstractions) and infiltration into the soil matrix. The Green-Ampt method requires the development of the following parameters:

- **Initial abstractions (IA):** IA values account for interception and surface retention and were based on the land use, slope, and percentage of vegetation in each subbasin. IA values were developed using Table 9 of the HBRPEG (SEO, 2008), for urban areas.
- **Percent Impervious Area (RTIMP):** The effective RTIMP represents directly connected impervious areas within the basin. Potential impervious areas include rock outcrops, parking lots, paved roads, ponds etc. IA values were developed using Table 9 of the HBRPEG (SEO, 2008).
- **Hydraulic Conductivity (XKSAT):** XKSAT values represent infiltration of rainfall into saturated soils. Bare ground hydraulic conductivity values were identified based on textural information for each soil type extracted from the soils reports. Hydraulic conductivity values were selected based on values presented in Table 10 of the

HBRPEG for all soil types except for sands, gravels, and cobbles, which were based on other published data.

Loss parameters were developed for the top 18 inches of the soil profile. The hydraulic conductivity for each soil profile was calculated using a harmonic average of the vertical soil layers.

Adjustment to the bare ground XKSAT values for vegetation were applied based on Figure 8 in the HBRPEG (SEO, 2008) for soil map units with a XKSAT value less than 1.2 inches/hour. Weighted average XKSAT values were calculated for each soil map unit and then each sub-basin using the geometric average equation.

- **Wetting Front Capillary Suction (PSIF):** Wetting front capillary suction (PSIF) values represent the amount of attraction infiltrated water has to the soil void spaces and were developed using Figure 4 of the HBRPEG (SEO, 2008) based on weighted average bare ground XKSAT values.
- **Soil Moisture Deficit at the Start of Rainfall (DTHETA):** Values for soil moisture deficit at start of rainfall (DTHETA) were developed using Figure 4 of the HBRPEG (SEO, 2008). A “Normal” antecedent moisture condition was selected to account for some irrigation of residential and commercial land.
- **Saturated Soil Moisture Content:** Saturated soil moisture content is equivalent to the effective porosity of the soil. Effective porosity values for each soil type were obtained from published references and a weighted average was calculated for each soil map unit and then each sub-basin using the same computational approach used for the XKSAT values.
- **Initial Soil Moisture Content:** Equal to the saturated soil moisture content minus DTHETA.

A soil profile storage analysis was not performed because the depth to bedrock is significantly more than 18 inches, and we do not anticipate that soil profile storage will limit infiltration. A summary of loss parameters for the Green-Ampt method is presented in Table 6.1.

TABLE 6.1
GREEN-AMPT LOSS PARAMETERS

Subbasin	Percent Impervious (RTIMP) (%)	Initial Abstraction (IA) (in)	Wetting Front Capillary Suction (PSIF) (in)	Soil Moisture Deficit (DTHETA)	Saturated Soil Moisture Content	Initial Soil Moisture Content	Vegetation Adjusted XKSAT (in/hr)
Upstream	25.6	0.28	3.36	0.26	0.37	0.11	0.32
Downstream	27.9	0.28	4.47	0.26	0.38	0.12	0.29

6.3 Unit Hydrograph

A unit hydrograph is the direct runoff hydrograph resulting from a unit 1-inch depth of excess rainfall produced by a storm of uniform intensity and specified duration over a basin. The

Hydrologic Basin Response Parameter Estimation Guidelines (HBRPEG) (SEO, 2008) recommends using the Clark unit hydrograph procedure for urban basins.

The Clark unit hydrograph method requires the development of three parameters: time of concentration, storage coefficient, and time-area relation. The time of concentration was calculated using the Clark unit hydrograph equation for urban basins:

$$T_c = 3.2 \times A^{0.1} \times L^{0.25} \times L_{ca}^{0.25} \times S^{-0.14} \times RTIMP^{-0.36}$$

Where: T_c = time of concentration (hours)

A = basin area (sq. miles)

L = length of longest watercourse (miles)

L_{ca} = length along “L” from the basin discharge point to a point perpendicular to the centroid of the drainage basin (miles)

S = overall slope of L (feet/mile)

RTIMP = percent impervious area

The storage coefficient for the Clark unit hydrograph method relates the effects of direct runoff storage to the shape of the unit hydrograph and was calculated using the following equation:

$$R = 0.37 \times T_c^{1.11} \times L^{0.8} \times A^{-0.57}$$

Where: R = storage coefficient

T_c = time of concentration (hours)

L = length of longest watercourse (miles)

A = basin area (sq. miles)

Time of concentration and storage coefficients are presented in Table 6.2.

TABLE 6.2
CLARK UNIT HYDROGRAPH PARAMETERS

Subbasin	Time of concentration (hour)	Storage Coefficient (hour)
Upstream	0.38	0.54
Downstream	0.24	0.39

The time-area relation is a graphical parameter that quantifies the accumulated area of the subbasin that contributes runoff to the subbasin outlet at a given time during the storm event. Time-area relationships can be developed by detailed analysis of the watershed or by use of synthetic time-area relations. RJH used a synthetic time-area relation for urban basins from HBRPEG, which is presented in Table 6.3.

TABLE 6.3
CLARK UNIT HYDROGRAPH TIME-AREA RELATION

Travel Time as a percent of Tc	Contributing Area as a percent of Total Area
0	0
10	5
20	16
30	30
40	65
50	77
60	84
70	90
80	94
90	97
100	100

7.0 Precipitation Runoff and Routing

7.1 General

This section presents the rainfall-runoff modeling results for the Viele Channel basin. The basin parameters, hydrologic parameters, and precipitation data discussed in the preceding sections were input into a U.S. Army Corps of Engineer (USACE) HEC-HMS rainfall-runoff computer model to develop the inflow hydrograph for each storm.

7.2 Reservoir Routing

Reservoir routing was performed to account for hydrograph attenuation provided by Viele Lake Dam. The required HEC-HMS inputs for modeling a reservoir and spillway include an elevation-capacity relationship for the reservoir and a rating curve for the spillway. The reservoir elevation-capacity and spillway rating curve were estimated using the USGS 1-meter DEM data. We assumed the reservoir pool was at the spillway crest in the HEC-HMS model. A summary of key reservoir and spillway characteristics that were used in the HEC-HMS model are presented in Table 7.1.

TABLE 7.1
VIELE LAKE AND SPILLWAY CHARACTERISTICS

Maximum Normal Pool El.	5500.0
Dam Crest El.	5504.5
Storage Capacity at Spillway Invert	22.5 ac-ft
Storage Capacity at Dam Crest	100 ac-ft
Spillway Invert El.	5500.0
Spillway Discharge at Dam Crest	1,900 cfs

Viele Lake and spillway do not have sufficient hydraulic capacity to rout the probable maximum flood (PMF), and the dam would breach during a PMF. RJH evaluated overtopping breach parameters using the Froehlich method in general accordance with recommendations from the *Guidelines for Dam Breach Analysis* (SEO, 2010) based on dam size and storage intensity.

A summary of overtopping breach parameters is presented in Table 7.2.

TABLE 7.2
OVERTOPPING BREACH PARAMETER ESTIMATES

Parameter	Froehlich 2008
Bottom Breach Width, B_b (ft)	40
Breach Formation Time, T_f (hr)	0.65
Breach Side Slope Ratio, Z_b (ZH:1V)	1

7.3 Channel Routing

Channel routing is used to account for floodplain storage and resulting peak attenuation of a flood wave as it travels through a channel or river. Channel routing parameters were developed using the Muskingum-Cunge methodology, which is appropriate for well-defined stream channels like Viele Channel. Channel routing was performed for the upstream sub-basin hydrograph as it travels through the downstream sub-basin.

The Muskingum-Cunge method requires identification of channel geometry, reach length, channel slope, and Manning's "n" roughness value. A Manning's n value of 0.03 was selected based on satellite imagery and channel geometry. Channel geometry was defined using the eight-point method, which consists of assigning eight points to define an appropriate shape for the cross section. Eight-point cross sections were developed using USGS 1-meter DEM data. A reach length of 6,924 feet and a slope of 2.3-percent were calculated using ArcGIS.

7.4 HEC-HMS Model

A summary of results from the HEC-HMS model are presented in Table 7.3. The 72-hour general storm and 24-hour local storm were not modeled because the 2-hour local storm provided a significantly higher peak flow than the 6-hour local storm.

TABLE 7.3
HEC-HMS PEAK FLOWS IN VIELE CHANNEL ADJACENT TO PROJECT SITE

Flood Event	Storm	Peak Inflow (cfs)	Inflow Volume (ac-ft)
PMF	2-hour Local Storm	6,033	741
	6-hour Local Storm	3,849	993
10,000-year	2-hour Local Storm	2,069	235
	MEC 6-hour	1,285	228
1,000-year	2-hour Local Storm	1,375	162
	MEC 6-hour	829	142
100-year	2-hour Local Storm	839	96
	MEC 6-hour	430	75

8.0 Conclusions

Based on the hydrologic evaluation of the Viele Channel watershed upstream of the Project site, we offer the following conclusions:

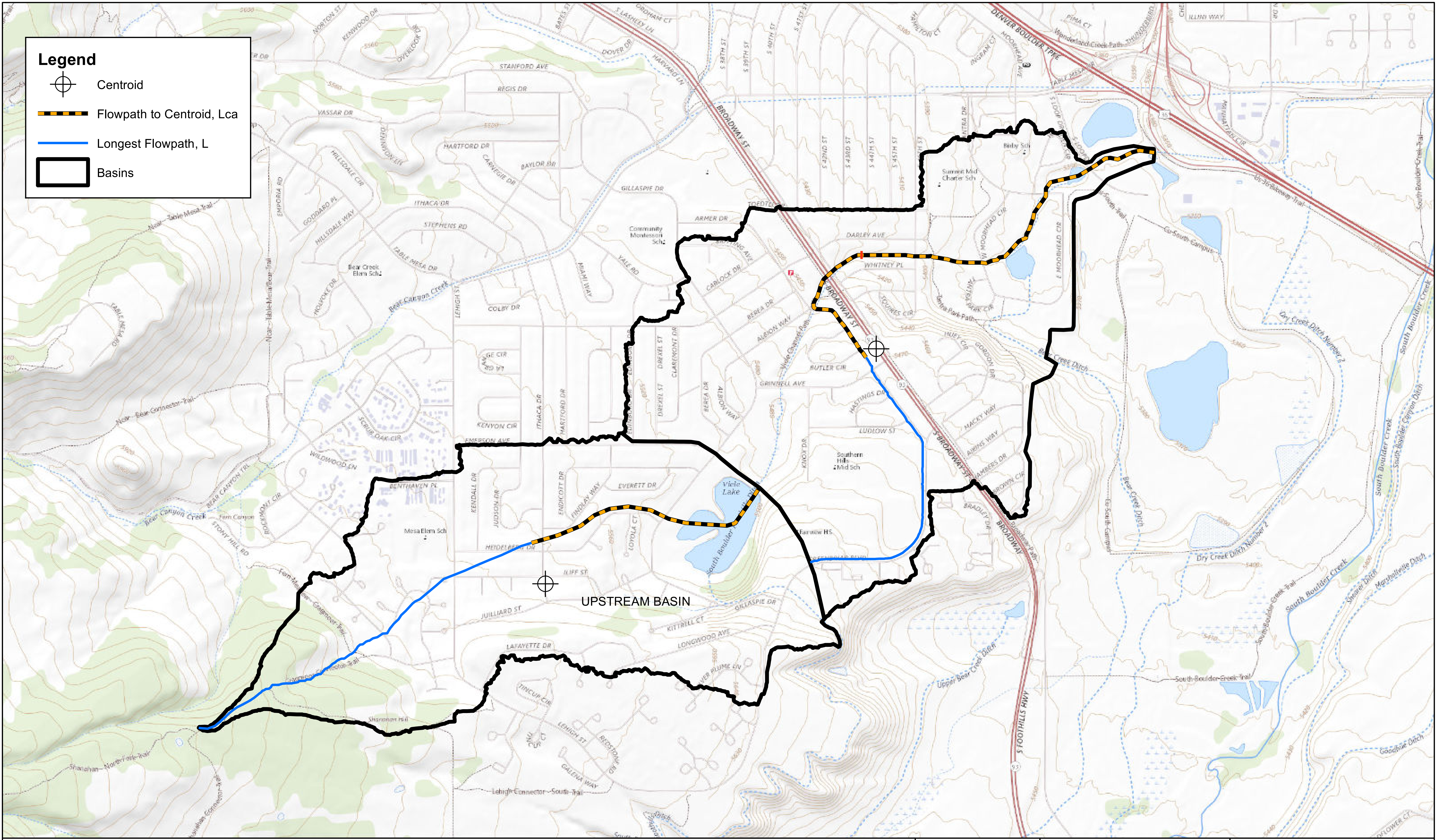
- The Viele Channel watershed at the Project site is approximately 1.2 square miles. Viele Lake is located in approximately the center of the watershed. Viele Lake is formed by a low-hazard, jurisdictional dam.
- Viele Lake and spillway do not have sufficient hydraulic capacity to rout the probable maximum flood (PMF), and the dam would breach during a PMF.
- The controlling PMF event for peak flow is the 2-hour Local storm. The peak flow rate for this event is 6,033 cfs.

9.0 References

Colorado Office of the State Engineer (2020). *Rules and Regulations for Dam Safety and Dam Construction.*

Colorado Office of the State Engineer (2008). *Hydrologic Basin Response Parameter Estimation Guidelines*.

Natural Resources Conservation Service (NRCS) online Web Soil Survey (WSS).



0 500 1,000 2,000
Feet

N

THIS DOCUMENT MUST BE REPRODUCED IN COLOR

RJH CONSULTANTS, INC.

SOUTH BOULDER CREEK PRELIMINARY DESIGN REPORT

VIELE CHANNEL BASIN PARAMETERS

PROJECT NUMBER 16134

April 2021

Figure 1

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APPENDIX G.2

VIELE CHANNEL HYDRAULIC EVALUATION (PMF)

CONSULTANTS, INC.

TECHNICAL MEMORANDUM

Project 16134

TO: **Brandon Coleman, P.E. – City of Boulder**

FROM: **Eric Hahn, P.E. - RJH Consultants, Inc.**

DATE: **August 12, 2021**

RE: **South Boulder Creek Regional Detention Project**
Viele Channel Flood Impacts on Dam Embankment

1.0 Purpose

This memorandum has been prepared by RJH Consultants, Inc. (RJH) to present the methodology and results of the hydraulic evaluation performed for the South Boulder Creek (SBC) Regional Detention Project (Project) for the City of Boulder (City) to evaluate impacts of Viele Channel flood flows on the dam embankment.

2.0 Background

Viele Channel is an urban stream that extends through the northwest portion of the Project site. The Project will include the construction of an earthen embankment located south of Viele Channel on the University of Colorado (CU) Boulder South campus property. The alignment of the earthen embankment has generally been located so that the downstream toe of the embankment is about 50 feet away from the top of the right bank of Viele Channel.

Hydraulic analysis of Viele Channel is required to identify the impacts of flooding in Viele Channel on the dam embankment and to support design of facilities to mitigate these impacts, if needed. Hydrologic analysis to identify peak flow rates in Viele Channel for various flood events is presented in the *Viele Channel Hydrologic Evaluation Memorandum* (RJH, 2021).

3.0 Regulatory Criteria

The earthen embankment will be a jurisdictional dam that will be regulated by the Colorado Office of the State Engineer (SEO). The hydraulic analyses presented in this memorandum have been prepared consistent with guidelines presented in the SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO Rules) (SEO, 2020). The dam will likely be classified as an extreme hydrologic hazard dam; and for preliminary design, the City has requested that designs be developed for this classification. The SEO Rules require that an extreme hydrologic hazard dam be designed to safely convey flows during the Probable Maximum Flood (PMF) event. This includes adjacent drainages where PMF flows could cause damage to Project components.

4.0 Channel Characteristics

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 below 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 downstream slope of the dam embankment is planned to consist of grass-covered earthfill at a 4H:1V slope. The downstream toe of the dam will include buried drains and filters. The downstream toe areas will also be covered with grass.

5.0 Hydraulic Model

The flow regime beyond the main channel of Viele Channel will consist of shallow overland flow. There will be several flow splits and a portion of the flow will overtop U.S. Interstate 36 (US36). A two-dimensional hydraulic model is needed to adequately model these hydraulic conditions. RJH developed a two-dimensional hydraulic model using HEC-RAS 5.0.7. Key model components are described as follows:

- A terrain model was prepared using a U.S. Geological Survey (USGS) 1-meter Digital Elevation Model (DEM), supplemented with data from the 2017 topographic survey and the Project features.
- A hydraulic mesh was delineated extending approximately 0.15 and 0.6 miles upstream and downstream of US36, respectively. The mesh was extended laterally to cover the entire flooded area during the PMF passage. A 10 feet cell size was used to facilitate a reasonable running time. Break-lines with a maximum of cell size of 5 feet were introduced into the model to adequately represent flow paths in the vicinity of roadways and channels.
- A Manning's n raster layer was prepared using data from the USGS National Land Cover Database (NLCD, 2016).
- Culvert structures along Viele Channel were modeled as connections within the hydraulic mesh.
- A normal depth boundary condition was used at the downstream end of the model.
- 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. This hydrograph has a peak flow rate of 6,030 cfs.
- An unsteady model simulation was performed using a computation interval of 1 second and a 10 seconds mapping output interval.

A plan of the terrain model used for the model simulation is presented on Figure 5.1.

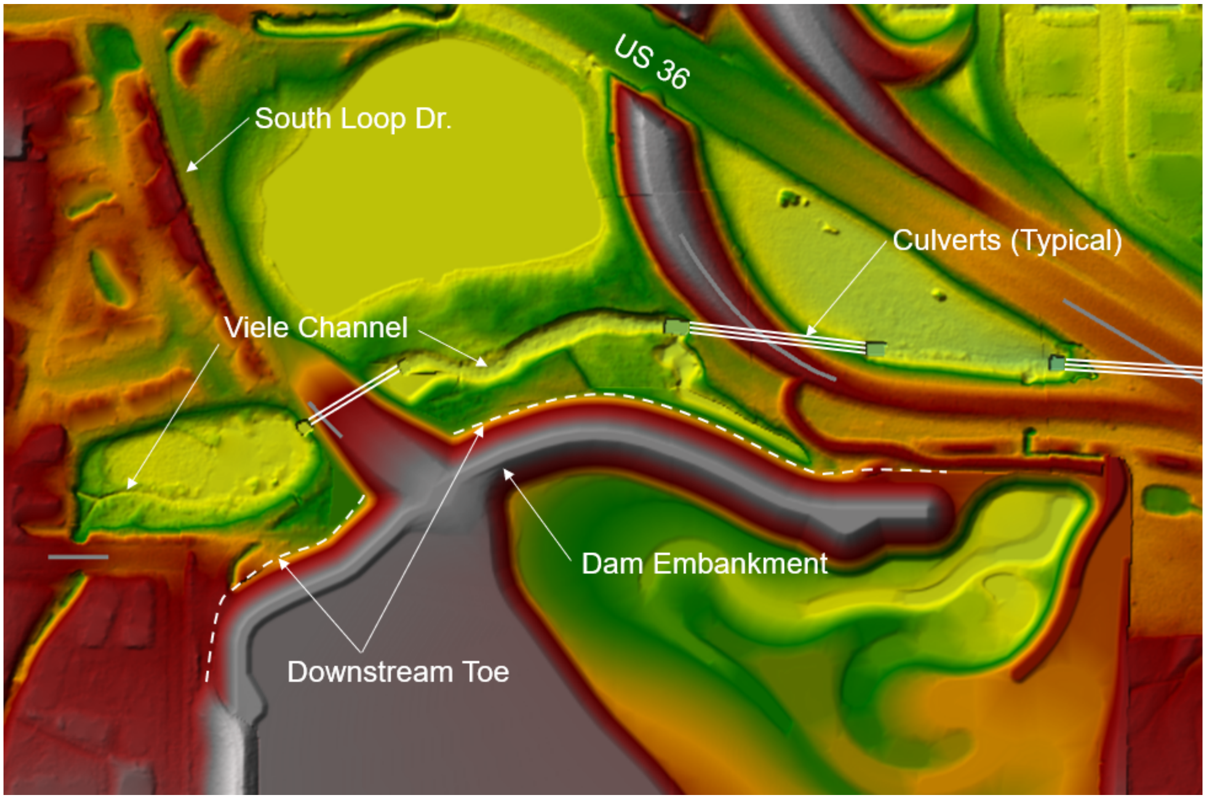

Figure 5.1 – Plan of Terrain Model

Model results are presented on Figure 5.2. Computed velocities along a majority of the downstream slope of the embankment would be less than 0.5 feet per second during the PMF. These velocities would not be expected to cause erosion of grass-covered earthfill materials. There is an approximately 130-foot-long segment of the downstream slope where the velocities would be between about 2 to 4 feet per second (fps). The flow depths in this area would be less than 2 feet. These velocities would likely not cause erosion of grass-covered earthfill materials if the grass cover was dense. If grass cover is not dense, then minor erosion would be expected. We do not anticipate that minor erosion in this area would be a dam safety risk.

Flow velocities above 6 ft/s would be confined to areas away from the dam including the main channel, drainage structures, and roadways.

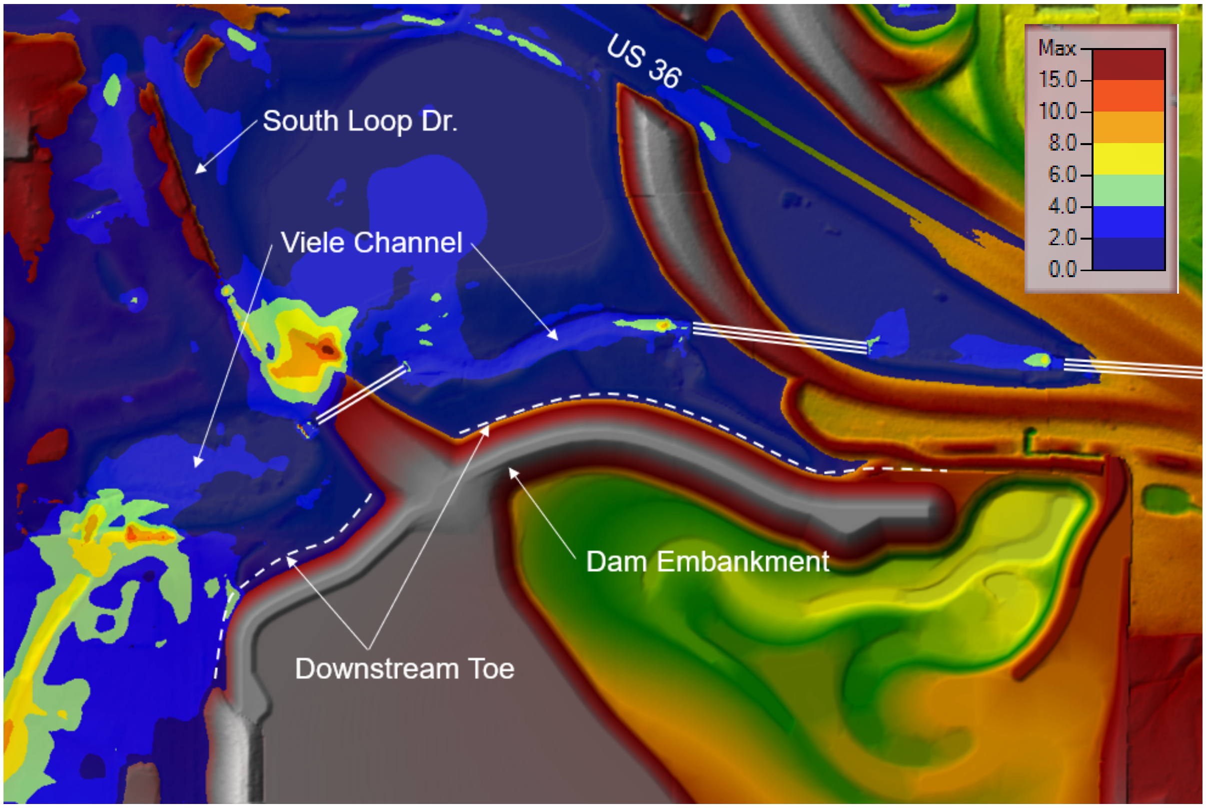

Figure 5.2 – Plan of HEC-RAS Model Results

6.0 Recommendations

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

7.0 References

Colorado Office of the State Engineer (2020). *Rules and Regulations for Dam Safety and Dam Construction.*

RJH Consultants (2021). *Viele Channel Hydrologic Evaluation Memorandum* (RJH, 2021).

APPENDIX G.3

DRY CREEK DITCH NO. 2 MODIFICATIONS

DRAFT MEMORANDUM

Project 16134

TO: **Brandon Coleman, P.E. - City of Boulder**

FROM: **Robert Huzjak, P.E. - RJH Consultants, Inc.**

DATE: **March 1, 2022**

RE: **South Boulder Creek Regional Detention Project**
Dry Creek Ditch No. 2 Memorandum

1.0 Introduction

The purpose of this memorandum is to present a summary of existing conditions and potential impacts to Dry Creek Ditch No. 2 (DCD2) from implementation of the South Boulder Creek (SBC) Regional Detention Project (Project).

2.0 Background

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 City of Boulder Open Space and Mountain Parks (OSMP) property to the Project site. Multiple turnout structures are located along this segment of the ditch that facilitate flood irrigation of OSMP property.

DCD2 extends below U.S. Highway 36 (US36) in a 6-foot by 4-foot reinforced concrete box culvert. The culvert discharges to a 6-foot-wide by 3-foot-high rectangular concrete-lined channel downstream of US36. The concrete lined channel transitions to a 5.25-foot-wide by 2-foot-high concrete-lined channel approximately 85 feet downstream of the culvert and exits to an approximately 7-foot-wide earthen ditch 375 feet downstream of the culvert outlet. Photographs at select locations along the ditch are attached.

The capacity of the ditch varies significantly by location. The approximate ditch capacity at various locations was identified based on the existing topography and sizes of facilities and is presented on Figure 1. Based on information from the City of Boulder Water Resources Department, the decreed water right in DCD2 at the headgate is 44 cubic feet per second (cfs).

3.0 Potential Impacts

The proposed spillway alignment intersects DCD2 approximately 75 feet upstream of US36. The Project will need to be designed to facilitate conveyance of the decreed flow rate in the ditch and maintain the ability to flood irrigate OSMP property.

The portion of DCD2 upstream of US36 is currently, and will continue to be, inundated during large flood events in SBC. However, the depth and duration of inundation will be higher than existing conditions. Based on hydraulic modeling performed by the Project team, the Project would increase the 100-year water surface elevation at the US36 culvert by approximately 2.5 feet (1.1 psi) and would increase the 100-year flow through the culvert by 70 cfs (from 290 to 360 cfs). Graphs of hydraulic loading are attached.

We recommend meeting with the DCD2 Company to discuss these issues and better understand their concerns.

Photograph 1: DCD2 earthen channel south of US36, looking north.

Photograph 2: DCD2 box culvert on north side of US36 discharging to concrete-lined channel, looking south.

Photograph 3: DCD2 concrete channel along north side of US36, looking west.

Photograph 4: DCD2 concrete lined channel transitioning to earth ditch north of US36, looking east.

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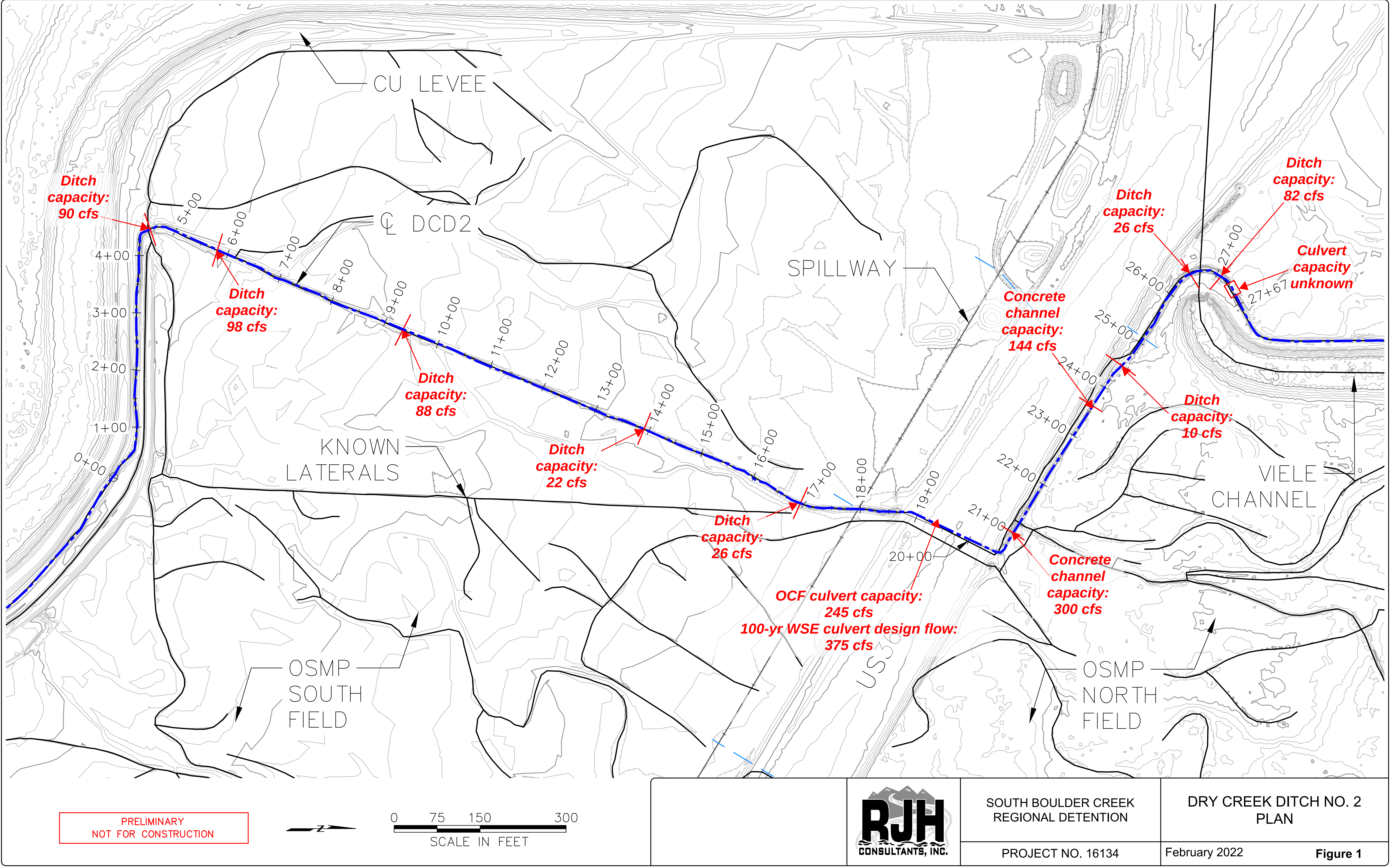


PRELIMINARY
NOT FOR CONSTRUCTION

RJH CONSULTANTS, INC.

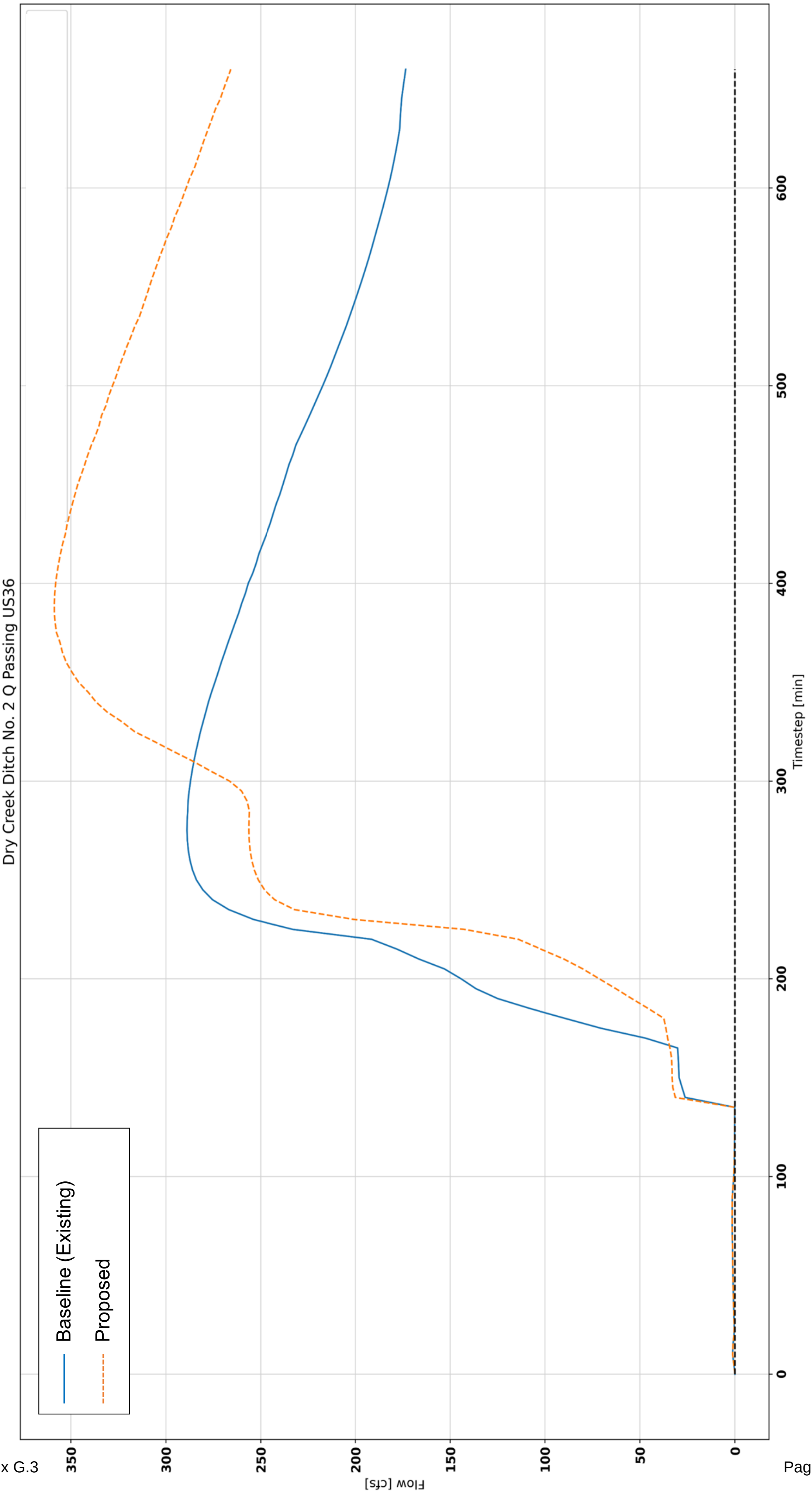
SOUTH BOULDER CREEK
REGIONAL DETENTION

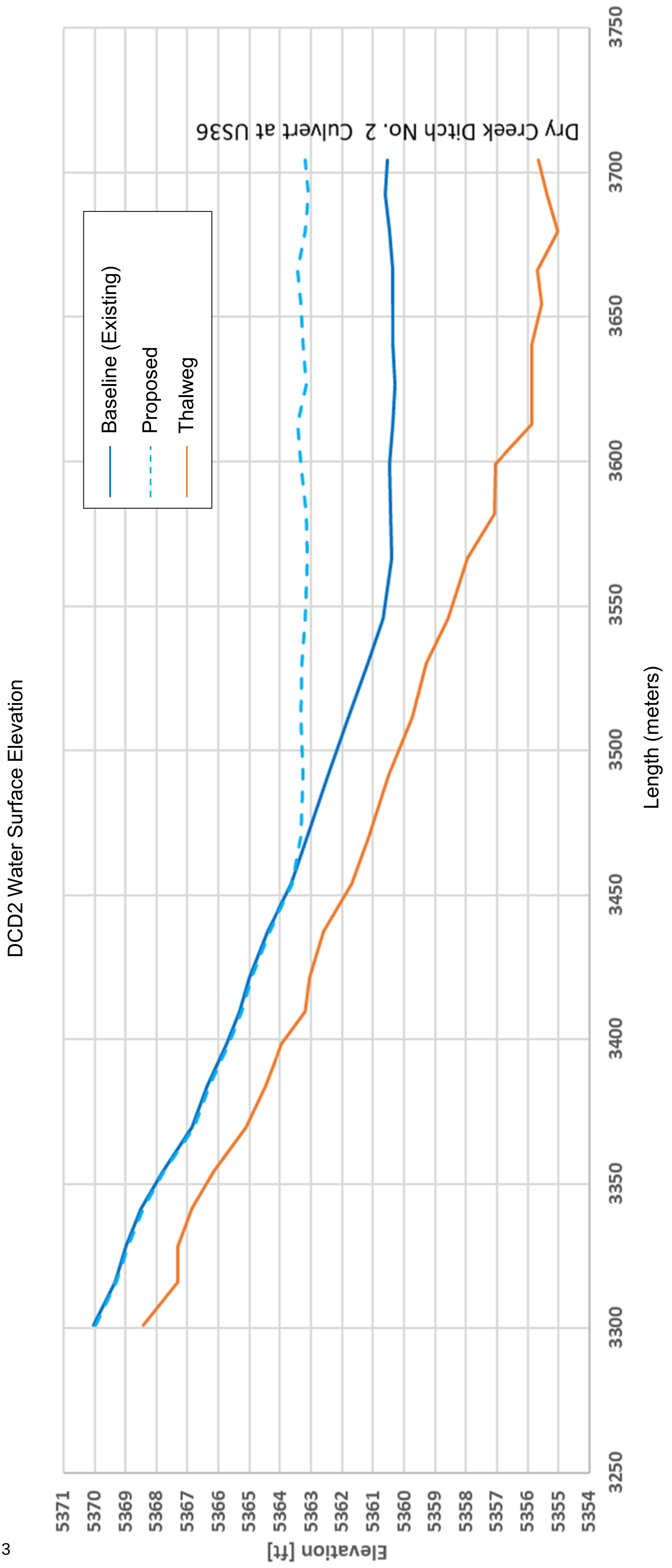
PROJECT NO. 16134

DRY CREEK DITCH NO. 2
PLAN

February 2022

Figure 1


Dry Creek Ditch No. 2 Q Passing US36
Flow [cfs]
0
50
100
150
200
250
300
350
Timestep [min]
0
100
200
300
400
500
600
Baseline (Existing)
Proposed


DCD2 Water Surface Elevation
Dry Creek Ditch No. 2 Culvert at US36
Baseline (Existing)
Proposed
Thalweg
Elevation [ft]
5371
5370
5369
5368
5367
5366
5365
5364
5363
5362
5361
5360
5359
5358
5357
5356
5355
5354
Length (meters)
3250
3300
3350
3400
3450
3500
3550
3600
3650
3700
3750

APPENDIX H

JURISDICTIONAL DETERMINATION

APPENDIX H.1

REQUEST FOR APPROVED JURISDICTIONAL DETERMINATION

ERO Resources Corp.

November 11, 2021

Mr. Kiel Downing
Denver Regulatory Office
U.S. Army Corps of Engineers
9307 S. Wadsworth Blvd.
Littleton, Colorado 80128-6901

RE: Request for Approved Jurisdictional Determination for South Boulder Creek Detention Project - Viele Channel Ditch, Dry Creek Ditch Number 2, Pond 1, Pond 2, Pond 3, Pond 4, and wetlands, Boulder County, Colorado

Dear Mr. Downing,

On behalf of the City of Boulder (project proponent), ERO Resources Corporation (ERO) is requesting an Approved Jurisdictional Determination (AJD) for the Viele Channel Ditch, Dry Creek Ditch Number 2, Pond 1, Pond 2, Pond 3, Pond 4, and the wetlands located within the South Boulder Creek Detention Project area boundary near U.S. 36 in the City of Boulder and Boulder County, Colorado (project area; Figure 1). The project proponent is planning a flood mitigation project and would like to confirm the jurisdictional status of the waters and wetlands within the project area boundary. In 2019, CORVUS Environmental Consulting, LLC delineated waters and wetlands within the project area (CORVUS 2021) and ERO reviewed the entirety of the project area on February 21, February 22, and May 26, 2021 (2021 site visits) to confirm the wetlands within the project area contained characteristics of waters of the U.S. or had a defined surface connection to a known water of the U.S. Additionally, ERO met with Matt Montgomery of the U.S. Army Corps of Engineers (Corps) on August 17, 2021, to walk the site and discuss the proposed project.

Project Area

The project area is in Sections 9 and 10, Township 1 South, Range 70 West of the 6th Principal Meridian in Boulder County, Colorado (Figure 1). The UTM coordinates for the approximate center of the project area are 480494mE, 4425331mN, Zone 13 North. The longitude/latitude of the project area is 105.228438°W/39.977917°N. The elevation of the project area ranges from 5,280 to 5,310 feet above sea level. Photo points of the project area are shown on Figure 2, and the photo log is attached. Wetland delineations datapoint locations are shown on Figure 2, and the wetland determination datasheets are attached.

Denver
1842 Clarkson St.
Denver, CO 80218
303.830.1188

Durango
1015 ½ Main Avenue
Durango, CO 81301
970.422.2136

Hotchkiss
P.O. Box 932
161 South 2nd St.
Hotchkiss, CO 81419
970.872.3020

Idaho
4001 East Main Street
Emmett, ID 83617
208.365.7684

www.eroresources.com

Consultants in
Natural Resources
and the Environment

Project Area Description

The project area is located mostly within a former sand and gravel mine site on CU South Campus, along an existing levee, and within open meadow along U.S. 36 near the Table Mesa Drive/South Boulder Road exit (Figure 1). The project area is bounded by South Boulder Creek (SBC) and associated riparian corridor to the east, the University of Colorado CU South Campus, City of Boulder Open Space, and South Boulder Neighborhoods to the west and south, and South Boulder Road to the north (Figure 2). The project area consists primarily of undeveloped uplands; recreational facilities, roads, and trails; a levee; as well as several ponds and wetlands (Figure 2).

The majority of the vegetation in the project area is dominated by native and nonnative upland species including smooth brome (*Bromus inermis*), crested wheatgrass (*Agropyron cristatum*), alfalfa (*Medicago sativa*), bindweed (*Convolvulus arvensis*), cheatgrass (*Bromus tectorum*). big bluestem (*Andropogon gerardii*), Indian grass (*Sorgastrum nutans*), switchgrass (*Panicum virgatum*), orchard grass (*Dactylis glomerata*), and tall fescue (*Schedonorus arundinaceus*). In addition, wild licorice (*Glycyrrhiza lepidota*), western ragweed (*Ambrosia psilostachya*), fringed sage (*Artemisia frigida*), yucca (*Yucca glauca*), prickly pear cactus (*Opuntia polyacantha*), and three-leaf sumac (*Rhus trilobata*) occur within uplands in the project area (Photo 1 through 3).

The riparian and wetland areas within the project area consist wide variety of grasses, forbs, and graminoids including redtop (*Agrostis gigantea*), swamp milkweed (*Asclepias incarnata*), Nebraska sedge (*Carex nebrascensis*), woolly sedge (*Carex pellita*), clustered field sedge (*Carex praegracilis*), Canada thistle (*Cirsium arvense*), common teasel (*Dipsacus fullonum*), Russian olive (*Elaeagnus angustifolia*), fringed willowherb (*Epilobium ciliatum*), Nutall's sunflower (*Helianthus nutallii*), Baltic rush (*Juncus balticus*), Inland rush (*Juncus interior*), Narrowleaf Bird's-foot Trefoil (*Lotus tenuis*), scratchgrass (*Muhlenbergia asperifolia*), reed canary grass (*Phalaris arundinacea*), switchgrass (*Panicum virgatum*), narrowleaf plaintain (*Plantago lanceolata*), Kentucky bluegrass (*Poa pratensis*), Nuttall's alkaligrass (*Puccinellia nuttalliana*), tall fescue (*Schedonorus arundinaceus*), three-square bulrush (*Schoenoplectus pungens*), prairie cordgrass (*Spartina pectinata*), alkali sacaton (*Sporobolus airoides*), and cattails (*Typha angustifolia* and *Typha latifolia*). Woody plants that are common in riparian and wetland areas include plains cottonwood (*Populus deltoides*), narrowleaf cottonwood (*Populus angustifolia*), peachleaf willow (*Salix amygdaloides*), alder (*Alnus incana*), box elder (*Acer negundo*), green ash (*Fraxinus pennsylvanica*), Siberian elm (*Ulmus pumila*), and shrubs including chokecherry (*Prunus virginiana*), sandbar willow (*Salix exigua*), hawthorn (*Crataegus* spp.), leadplant (*Amorpha fruticosa*), and snowberry (*Symphoricarpos occidentalis*). (Photos 3 through 14).

Wetlands and Other Waters

Viele Channel Ditch

The Viele Channel Ditch is shown on the U.S. Geological Survey (USGS) Louisville, Colorado topographic quadrangle and the National Hydrography Dataset (NHD) as an intermittent Stream/River and as a Canal/Ditch within the project area (Figure 1, Figure 2; Photo 3 and Photo 4). Downstream of the project area, Viele Channel Ditch continues northeast for approximately 0.5 miles before joining South Boulder Creek, a known jurisdictional water of the U.S. Reviewing historical imagery, the Viele Channel appears to have been constructed between 1971 and 1983 to convey stormwater off South Boulder neighborhoods west of the project area (NETR 2021). During the 2021 site visits, ERO mapped 0.01 acre (46 feet) of open water and 0.50 acre of wetlands along the Viele Channel Ditch in the project area (Figure 2). During the 2021 site visits, ERO and CORVUS observed wetlands, open water, and the occasional ordinary high-water mark (OHWM) along the length of the Viele Channel Ditch. Wetlands within Viele Channel Ditch are dominated by broadleaf cattail and sandbar willow as well as Nutall's sunflower, mountain rush, woolly sedge, and redtop (Photo 3 and Photo 4; SP19).

Dry Creek Number 2 Ditch

Dry Creek Number 2 Ditch is shown on the USGS Louisville, Colorado topographic quadrangle and the NHD as Canal/Ditch in the project area (Figure 1, Figure 2). Dry Creek Number 2 Ditch is an irrigation ditch that diverts water from South Boulder Creek upstream of the project area west of South Foothills Highway, moves irrigation water through the project area, and then back into South Boulder Creek via the Viele Channel Ditch. The water within Dry Creek Number 2 Ditch is used to irrigate the open space agricultural fields within and adjacent to the project area. Dry Creek Number 2 Ditch enters the project from the south, flows north and then west along the eastern edge of the existing levee, turns northeast to leave the project area, and returns to enter a culvert under U.S. 36. North of U.S. 36, Dry Creek Number 2 Ditch joins wetlands north of U.S. 36 that abut the Viele Channel Ditch within the project area (Photo 4). Reviewing historical imagery, the Dry Creek Number 2 Ditch appears to have been constructed prior to 1955 for the purpose of irrigation (NETR 2021). During the 2019 and 2021 site visits, ERO and CORVUS mapped 0.472 acre (4869 feet) of OHWM and 2.81 acre of wetlands along Dry Creek Number 2 Ditch in the project area (Figure 2). During the 2019 and 2021 site visits, ERO and CORVUS observed wetlands and an OHWM along the length of Dry Creek Number 2 Ditch in the project area (Figure 2). Wetlands in the ditch were dominated by plains cottonwood, sandbar willow, redtop, inland rush, woolly sedge, Baltic rush, Nebraska sedge, and tall fescue (Photo 4 and Photo 5; SP23).

Pond 1 and 2

Pond 1 and Pond 2 abut and occur south of the Viele Channel Ditch in the northwestern portion of the project area (Figure 2). Reviewing historical imagery, Pond 1 and Pond 2 appear to be water-filled depressions constructed in upland incidental to U.S. 36 construction activity occurring between 1971 and 1983 for the purpose of obtaining fill (NETR 2021). Pond 1 and Pond 2 currently receive surface flow via culverts at South Loop Drive, south of Pond 2, which connect these features to the wetlands that abut Pond 3. ERO and CORVUS observed a connection between the Pond 1 and Viele Channel Ditch near the Viele Channel culverts at U.S. 36 (Figure 2). During the 2019 and 2021 site visits, ERO and CORVUS mapped 0.05 acre (68.5 feet) of open water and 0.09 acre of wetlands along Pond 1 and 0.45 acre (228 feet) of open water and 0.42 acre of wetlands along Pond 2 within the project area (Figure 2). Wetlands abutting Pond 1 and Pond 2 were dominated by broadleaf cattail and sandbar willow (Photos 6 and 7).

Pond 3

Pond 3 occurs in the central portion of the project area, south of Viele Channel Ditch, Pond 1, Pond 2, and South Loop Drive (Figure 2). Reviewing historical imagery, Pond 3 appears to be a water-filled depression constructed in upland incidental to mining activity occurring in the 1960's (NETR 2021). Pond 3 receives surface and/or ground water flow from the south and connects to Pond 1 and 2 via wetlands and culverts at South Loop Drive (Figure 2). During the 2019 and 2021 site visits, ERO and CORVUS observed wetlands abutting the open water of Pond 3 and mapped 2.55 acre of open water and 0.55 acre of wetlands along this feature in the project area (Figure 2). Wetlands abutting Pond 3 were dominated by Siberian elm, plains cottonwood, three-square bulrush, common teasel, alkali sacaton, narrow-leaf bird's foot trefoil, and woolly sedge as well as broadleaf cattail and sandbar willow (Photo 8; SP10).

Pond 4

Pond 4 occurs in the central portion of the project area, southeast of the tennis courts (Figure 2). Reviewing historical imagery, Pond 4 appears to be a water-filled depression constructed in upland incidental to mining activity occurring in the 1960's (NETR 2021). Currently, Pond 4 appears to receive surface and/or ground water flow from the southwest, but ERO could not find a defined surface connection to downgradient wetlands located to the east, outside of the project area along the levee. During the 2019 and 2021 site visits, ERO and CORVUS observed wetlands abutting the open water of Pond 4 and mapped 0.25 acre of open water and 0.07 acre of wetlands along this feature in the project area (Figure 2). Wetlands abutting Pond 4 were dominated by broadleaf cattail and sandbar willow (Photo 9).

Wetlands

Wetlands occur in various locations throughout the project area including abutting the Viele Channel, Dry Creek Ditch Number 2, and Ponds 1 through 4, as discussed above (Figure 2). Outside of these features, wetlands are scattered in various locations throughout the project area including east of the existing levee in the eastern portion of the project area, along the U.S. 36 corridor, along the proposed construction access south of South Boulder Road in the northern portion of the project area, and in depressional areas within the boundary of the former mine site. During the 2019 and 2021 site visits, ERO and CORVUS observed these wetlands and mapped a total of 18.99 acre of wetlands within the project area (Figure 2).

These wetlands were dominated by a variety of species including common teasel, mountain rush, woolly sedge, redtop, Kentucky bluegrass, peachleaf willow, sandbar willow, alkali sacaton, scratch grass, narrowleaf plantain, swamp milkweed, fringed willowherb, Canada thistle, Russian olive, prairie cordgrass, clustered field sedge, and Nuttall's alkaligrass (Photos 4 through 14; SP1, SP2, SP4, SP12, SP15, SP25, SP27, and SP28).

Except as noted above, ERO could not precisely determine where many these wetlands receive surface and/or ground water flow and most appeared to lack connection with a downstream water.

Conclusions

Based on the 2021 site visits, ERO believes many of the wetlands and other waters within the project area may be considered nonjurisdictional. Many of the features lack a surface connection to a known water of the U.S. or fall into the category of features that are not jurisdictional including artificial ponds constructed in upland incidental to former construction or mining activities, or stormwater control features. The Viele Channel and Dry Creek Ditch Number 2 appear to be, respectively, a stormwater ditch and an irrigation ditch, both of which seem to have been constructed in upland and do not appear to be relocated natural tributaries (NERT 2021). The Viele Channel does have a direct hydrologic connection to South Boulder Creek, a known water of the U.S. As such, the Viele Channel as well as wetlands abutting the Viele Channel may be considered jurisdictional under current regulations.

On behalf of the project proponent, ERO requests an approved JD of the Viele Channel Ditch, Dry Creek Ditch Number 2, Pond 1, Pond 2, Pond 3, Pond 4, and the wetlands in the project area. Please contact me at chenke@eroresources.com if you need additional information to make this determination or have any questions.

Sincerely,

Clint Henke
Senior Biologist/Project Manager

Cc via email: Brandon Coleman –Engineering Project Manager, City of Boulder
Eric Hahn – RJH Consultants, Inc.

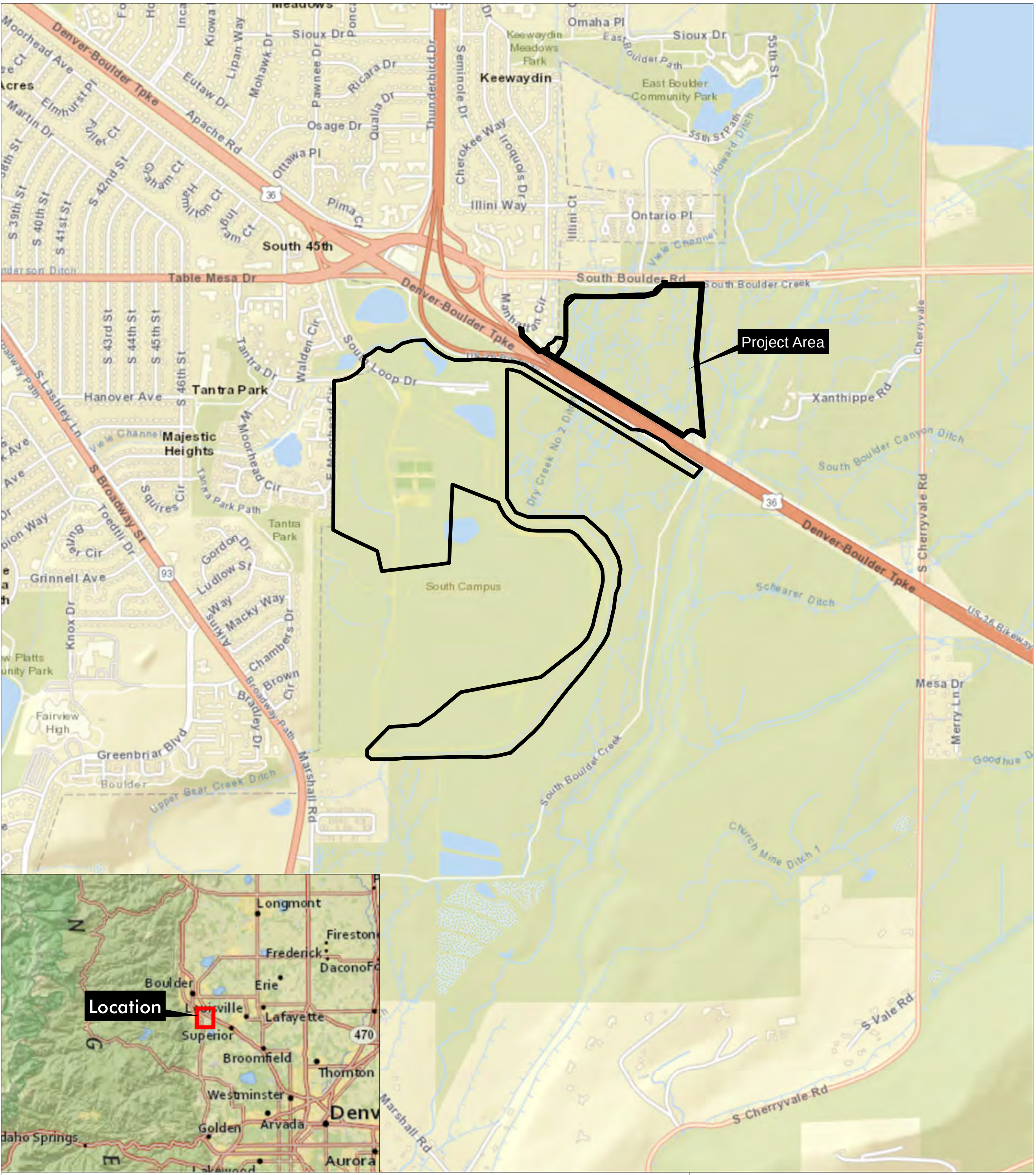
Attachments: Figures 1 and 2; Photo Log; Routine Wetland Determination Forms

References

CORVUS 2021. South Boulder Creek Regional Detention Project – Wetland Delineation Existing Conditions Draft Memorandum. February 9, 2021.

NETR 2021. Historic Aerials. https://www.historicaerials.com/viewer. Last accessed August 25, 2021.

Path: P:\20_100\20_171 South Boulder Detention Pond\Maps\20_171 Figure 1.mxd



South Boulder Creek Regional Detention Project

Sections 9 and 10, T1S, R70W; 6th PM

UTM NAD 83: Zone 13N; 480494mE, 4425331mN

Longitude 105.228438°W, Latitude 39.977917°N

USGS Louisville, CO Quadrangle

Boulder County, Colorado

0 750 1,500 Feet

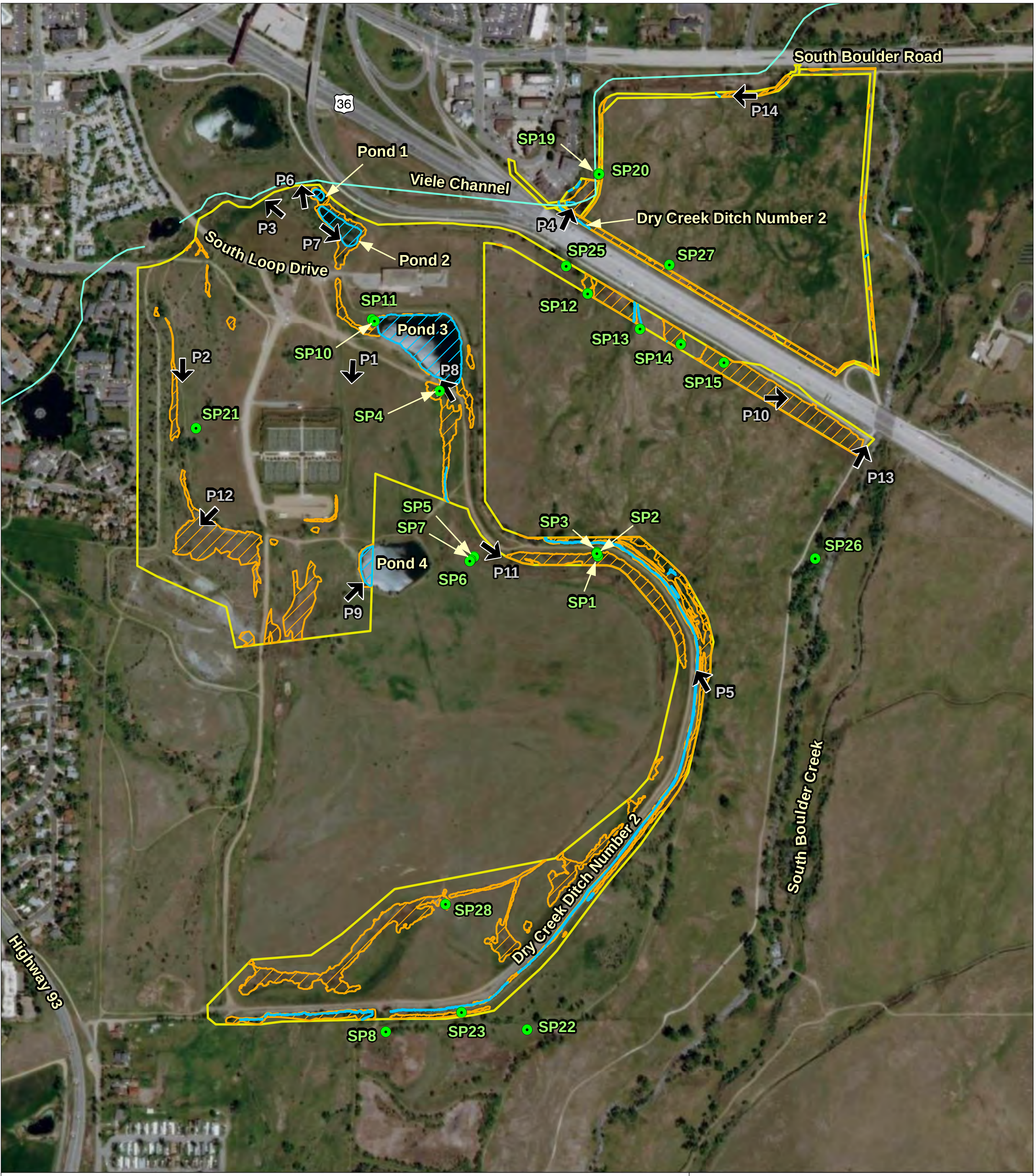
N

Figure 1
Vicinity Map

Prepared for: City of Boulder
File: 20_171 Figure 1.mxd (GS)
July 1, 2021

ERO Resources Corp.

Path: P:\20_100\20_171 South Boulder Detention Pond\Maps\20_171 JD Figure 2.mxd



South Boulder Creek Regional Detention Project

● Sample Point

→ Photo Point

Channel

Water (3.858 ac)

Wetland (18.995 ac)

Project Area Boundary

0 375 750 Feet

N

Figure 2
Existing Conditions

Image Source: Maxar©, May 2020

Prepared for: City of Boulder
File: 20_171 JD Figure 2.mxd (GS)
October 27, 2021

ERO Resources Corp.

Photo 1 - Uplands within the CU South portion of the project area. View is to the south.

Photo 2 - Uplands and wetlands along a trail within the western portion of the project area. View is to the south.

Photo 3 - Uplands adjacent to the Viele Channel within the northwestern portion of the project area. View is to the northwest.

Photo 4 - Overview of Dry Creek Ditch Number 2 and Viele Channel in the northern portion of the project area. View is to the northeast.

Photo 5 - Dry Creek Ditch Number 2 in the central portion of the project area. View is to the northeast.

Photo 6 - Pond 1 in the northwestern portion of the project area. View is to the northwest.

Photo 7 - Pond 2 in the northwestern portion of the project area. View is to the southeast.

Photo 8 - Pond 3 in the central portion the project area. View is to the northwest.

Photo 9 - Inflow to Pond 4 in the central portion of the project area. View is to the northeast.

Photo 10 - Wetlands along U.S. 36 in the eastern portion of the project area. View is to the east.

Photo 11 - Overview of wetlands along the existing levee in the central portion of the project area. View is to the southeast.

Photo 12 - Wetlands in the southwestern portion of the project area. View is to the southwest.

Photo 13 - South Boulder Creek underpass in the eastern portion of the project area. View is to the northeast.

Photo 14 - Wetlands along the proposed access road near the Viele Channel in the northern portion of the project area. View is to the west.

Note: No Sample Point data exist for numbers 9, 16, 17, 18 and 24. These are not missing data.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: South Boulder Creek City/County: Boulder/Boulder Sampling Date: 9/11/19

Applicant/Owner: See report State: CO Sampling Point: SP1

Investigator(s): Carla DeMasters PWS Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): toe of slope Local relief (concave, convex, none): concave Slope (%): 1

Subregion (LRR): G Lat: 39.978428 Long: -105.226804 Datum: NAD 83

Soil Map Unit Name: Nh - Niwot soils NWI classification: PEMIC

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ____

Hydric Soil Present? Yes ✓ No ____

Wetland Hydrology Present? Yes ✓ No ____

Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks: Location adjacent to berm, evidence of ponding water.

VEGETATION – Use scientific names of plants.

	Tree Stratum (Plot size: 30')	Absolute % Cover	Dominant Species?	Indicator Status
	1.			
	2.			
	3.			
	4.			
			= Total Cover	
	Sapling/Shrub Stratum (Plot size: 15')			
	1.			
	2.			
	3.			
	4.			
	5.			
			= Total Cover	
WM	Herb Stratum (Plot size: 5')			GP
FAC	1. Dipsacus fullonum	35	Y	FACU
FACW	2. Juncus balticus	20	Y	FACW
OBL	3. Typha angustifolia	2	N	OBL
OBL	4. Carex pellita	25	Y	OBL
OBL	5. Carex nebrascensis	5	N	OBL
OBL	6. Carex brevior	1	N	OBL
FAC	7. Cirsiom arvense	1	N	FACU
	8.			
	9.			
	10.			
		89	= Total Cover	
		45 18.		
	Woody Vine Stratum (Plot size: ____)			
	1.			
	2.			
			= Total Cover	

% Bare Ground in Herb Stratum 10

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 2 (A)

Total Number of Dominant Species Across All Strata: 3 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 66% (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	33	x 1 = 33
FACW species	20	x 2 = 40
FAC species	0	x 3 = 0
FACU species	36.	x 4 = 144
UPL species	0.	x 5 = 0
Column Totals:	89 (A)	217 (B)

Prevalence Index = B/A = 2.43

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 66
- ✓ 3 - Prevalence Index is ≤3.0[1] 2.43
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks: Veg passes both dominance + prevalence test.

Sampling Point: SP1.

SOIL

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-4	10YR 2/2	100					Sacl lm to Sacl.	Many fine roots. +some lg. rhizomes.
4-11	10YR 3/2	85	5YR 4/6	15	C	PL/M	Sa cl lm	Various gravel
11-14	2.5Y 3/1	80	10YR 3/4	20	C	PL/M	Sa lm.	0.5-3" diameter.

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ✓ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ✓ Redox Dark Surface (F6)
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16) (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16) (**LRR H outside of MLRA 72 & 73**)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ___

Depth (inches): ___

Hydric Soil Present? Yes ✓ No ___

Remarks: Earthworms present.
15% redox on average.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3) @14"
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ✓ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**)
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ___ FAC-Neutral Test (D5)
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___

Water Table Present? Yes ✓ No ✓ Depth (inches): ___

Saturation Present? (includes capillary fringe) Yes ✓ No ___ Depth (inches): 14"

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: sample adjacent to area that held open water.
>2% Oxidized rhizospheres.

US Army Corps of Engineers

Great Plains – Version 2.0

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/11/19
Applicant/Owner: See Report State: CO Sampling Point: SP 2
Investigator(s): CD. Section, Township, Range: S9 T1S R70 W
Landform (hillslope, terrace, etc.): hillslope Local relief (concave, convex, none): convex Slope (%): 5
Subregion (LRR): G Lat: 39.97846 Long: -105.226815 Datum: NAD 83
Soil Map Unit Name: Nh - Niwot NWI classification: PEMIC
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)
Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___
Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ✓
Hydric Soil Present? Yes ✓ No ___
Wetland Hydrology Present? Yes ✓ No ___

Is the Sampled Area within a Wetland? (transition) Yes ✓ No ✓

Remarks: Sample is located at the transition of wetland to upland + represents the location of the WL boundary

VEGETATION – Use scientific names of plants.

	Tree Stratum (Plot size: 30')	Absolute % Cover	Dominant Species?	Indicator Status
	1.			
	2.			
	3.			
	4.			
	= Total Cover			
	Sapling/Shrub Stratum (Plot size: 15')			
	1.			
	2.			
	3.			
	4.			
	5.			
	= Total Cover			
WMV	**Herb Stratum** (Plot size: 5')			GP
FAC	1. Oenothera villosa	10	N	FACU
FAC	2. Agrostis gigantea	30	Y	FACW
FAC	3. Thermopsis divaricarpa	10	N	FAC
FAC	4. Dipsacus fullonum	10	N	FACU
FAC	5. Symphyo. ericoides	10	N	FACU
OBL	6. Carex pellita	1	N	OBL
FACW	7. Panicum virgatum	1	N	FAC
FACU	8. Verbascum thapsus	1	N	UPL
FAC	9. Poa pratensis	15	Y	FACU
	10.			
	= Total Cover	88 44 18		
	Woody Vine Stratum (Plot size: ___)			
	1.			
	2.			
	= Total Cover			

% Bare Ground in Herb Stratum 10

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 1 (A)
Total Number of Dominant Species Across All Strata: 2 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 50% (A/B)

note would pass in WM.

Prevalence Index worksheet:

Total % Cover of:		Multiply by:	
OBL species	1	x 1 =	1
FACW species	30	x 2 =	60
FAC species	11	x 3 =	33
FACU species	45	x 4 =	180
UPL species	1	x 5 =	5
Column Totals:	88 (A)		279 (B)

Prevalence Index = B/A = ___

Hydrophytic Vegetation Indicators:

___ 1 - Rapid Test for Hydrophytic Vegetation
___ 2 - Dominance Test is >50% (=50)
___ 3 - Prevalence Index is ≤3.0[1] (3.17)
___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ___ (Based on western Mtns)

Remarks: Veg would have passed dom. test in Western Mtns. region, so to be conservative, veg. passes.

SOIL

Sampling Point: SP 2

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-7	10YR4/2	100					Sa cl lm	Many fine roots
7-16	10YR3/2	90	7.5YR5/8	10	C	M	Sa lm	large cobbles 1-6" dia.
							Sa lm	
16+	10YR 4/2	90	10YR4/6	10	C	M	sa lm	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR F**)
- ☐ 1 cm Muck (A9) (**LRR F, G, H**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☑ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ High Plains Depressions (F16)
  (MLRA 72 & 73 of LRR H)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) (**LRR I, J**)
- ☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ☐ Dark Surface (S7) (**LRR G**)
- ☐ High Plains Depressions (F16)
  (LRR H outside of MLRA 72 & 73)
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Very Shallow Dark Surface (TF12)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: /

Depth (inches):

Hydric Soil Present? Yes ✓ No ___

Remarks: Soil wetted for colors w/ sprayer. ~~Does not~~ meets redox d.s. b/c value is 3 @ 7"

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1)
- ☐ Sediment Deposits (B2)
- ☐ Drift Deposits (B3)
- ☐ Algal Mat or Crust (B4)
- ☐ Iron Deposits (B5)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Dry-Season Water Table (C2)
- ☑ Oxidized Rhizospheres on Living Roots (C3)
  (where not tilled) @ 7-16"
- ☐ Presence of Reduced Iron (C4)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ☐ Surface Soil Cracks (B6)
- ☐ Sparsely Vegetated Concave Surface (B8)
- ☐ Drainage Patterns (B10)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (where tilled)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Geomorphic Position (D2)
- ☐ FAC-Neutral Test (D5) ND
- ☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___

Water Table Present? Yes ___ No ✓ Depth (inches): ___

Saturation Present? Yes ___ No ✓ Depth (inches): ___
(includes capillary fringe)

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: oxidized rhizos w/in upper 12".

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/11/19
Applicant/Owner: See Report State: CO. Sampling Point: SP 3
Investigator(s): CD Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): hillslope Local relief (concave, convex, none): convex Slope (%): 10.
Subregion (LRR): G Lat: 39.978481 Long: -105.226817 Datum: NAD83
Soil Map Unit Name: Nh' Niwot NWI classification:
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)
Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___
Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ___	No ✓	Is the Sampled Area within a Wetland?	Yes ___ No ✓
Hydric Soil Present?	Yes ___	No ✓		
Wetland Hydrology Present?	Yes ___	No ✓		

Remarks: Very large cobbles, hard to dig past 12".

VEGETATION – Use scientific names of plants.

		Absolute % Cover	Dominant Species?	Indicator Status
	Tree Stratum (Plot size: 30')			
	1.			
	2.			
	3.			
	4.			
			= Total Cover	
	Sapling/Shrub Stratum (Plot size: 15')			
	1.			
	2.			
	3.			
	4.			
	5.			
			= Total Cover	
WM	Herb Stratum (Plot size: ___)			GP
FACU	1. Andropogon gerardii	10	Y	FACU
FAC	2. Thermopsis divaricarpa	10	Y	FAC
FACU	3. Ambrosia psilostachya	15	Y	FACU
UPL	4. Potentilla recta	15	Y	UPL
FAC	5. Symphyot ericoides	1	N	FACU
FACW	6. Panicum virgatum	1	N	FAC
FAC	7. Poa pratensis	20	Y	FACU
	8.			
	9.			
	10.			
		72	= Total Cover	
			36	14
	Woody Vine Stratum (Plot size: ___)			
	1.			
	2.			
			= Total Cover	
	% Bare Ground in Herb Stratum 20			

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 1 (A)
Total Number of Dominant Species Across All Strata: 5 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 20 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species		x 1 =
FACW species		x 2 =
FAC species		x 3 =
FACU species		x 4 =
UPL species		x 5 =
Column Totals:	(A)	(B)

Prevalence Index = B/A =

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ___ 2 - Dominance Test is >50% NO
- ___ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ___ No ✓

Remarks: upland plant community.

SOIL

Sampling Point: SP3

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix		Redox Features				Texture	Remarks
	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-6	10YR4/2	100.					Sa lm.	Many fine roots
6-12	10YR4/3	100					Sa lm.	
								large cobbles
								throughout
								0.5-5'.

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

__ Histosol (A1)
__ Histic Epipedon (A2)
__ Black Histic (A3)
__ Hydrogen Sulfide (A4)
__ Stratified Layers (A5) (**LRR F**)
__ 1 cm Muck (A9) (**LRR F, G, H**)
__ Depleted Below Dark Surface (A11)
__ Thick Dark Surface (A12)
__ Sandy Mucky Mineral (S1)
__ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
__ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
__ Sandy Gleyed Matrix (S4)
__ Sandy Redox (S5)
__ Stripped Matrix (S6)
__ Loamy Mucky Mineral (F1)
__ Loamy Gleyed Matrix (F2)
__ Depleted Matrix (F3)
__ Redox Dark Surface (F6)
__ Depleted Dark Surface (F7)
__ Redox Depressions (F8)
__ High Plains Depressions (F16)
(**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

__ 1 cm Muck (A9) (**LRR I, J**)
__ Coast Prairie Redox (A16) (**LRR F, G, H**)
__ Dark Surface (S7) (**LRR G**)
__ High Plains Depressions (F16)
(**LRR H outside of MLRA 72 & 73**)
__ Reduced Vertic (F18)
__ Red Parent Material (TF2)
__ Very Shallow Dark Surface (TF12)
__ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ________________
Depth (inches): ________________

Hydric Soil Present? Yes _____ No ✓

Remarks: Soil wetted w/ sprayer, Soil too light to meet indicators.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

__ Surface Water (A1)
__ High Water Table (A2)
__ Saturation (A3)
__ Water Marks (B1)
__ Sediment Deposits (B2)
__ Drift Deposits (B3)
__ Algal Mat or Crust (B4)
__ Iron Deposits (B5)
__ Inundation Visible on Aerial Imagery (B7)
__ Water-Stained Leaves (B9)
__ Salt Crust (B11)
__ Aquatic Invertebrates (B13)
__ Hydrogen Sulfide Odor (C1)
__ Dry-Season Water Table (C2)
__ Oxidized Rhizospheres on Living Roots (C3)
(**where not tilled**)
__ Presence of Reduced Iron (C4)
__ Thin Muck Surface (C7)
__ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

__ Surface Soil Cracks (B6)
__ Sparsely Vegetated Concave Surface (B8)
__ Drainage Patterns (B10)
__ Oxidized Rhizospheres on Living Roots (C3)
(**where tilled**)
__ Crayfish Burrows (C8)
__ Saturation Visible on Aerial Imagery (C9)
__ Geomorphic Position (D2)
__ FAC-Neutral Test (D5)
__ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes _____ No ✓ Depth (inches): ________
Water Table Present? Yes _____ No ✓ Depth (inches): ________
Saturation Present? (includes capillary fringe) Yes _____ No ✓ Depth (inches): ________

Wetland Hydrology Present? Yes _____ No ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: No redox.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/13/19

Applicant/Owner: See Report State: ____ Sampling Point: SP 4

Investigator(s): CD, AG Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): ditch floodplain depression Local relief (concave, convex, none): concave Slope (%): 1

Subregion (LRR): G Lat: 39.980905 Long: -105.229868 Datum: NAD83

Soil Map Unit Name: Nh- Niwot NWI classification: —

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ____	Is the Sampled Area within a Wetland?	Yes ✓ No ____
Hydric Soil Present? Yes ✓ No ____		
Wetland Hydrology Present? Yes ✓ No ____		

Remarks: Concave surface, halophytic plant community, close to ditch (mini-floodplain).

VEGETATION – Use scientific names of plants.

		Absolute % Cover	Dominant Species?	Indicator Status
	Tree Stratum (Plot size: 30')			
FACW	1. SALIX Amygdaloides	5	Y	FACW
	2.			
	3.			
	4.			
		5 = Total Cover		
	Sapling/Shrub Stratum (Plot size: 15')			
FACW	1. Salix exigua	10	Y	FACW
	2.			
	3.			
	4.			
	5.			
		10 = Total Cover		6P
WM	**Herb Stratum** (Plot size: 5')			
FAC	1. Sporobolis aeroides	40.	Y	FAC
FACW	2. Puccinelia nuttalis	20.	Y	OBL
UPL.	3. Bromus japonicus	2	N	UPL.
FACU	4. Chicory intybus	2	N	FACU.
FAC	5. Hordeum jubatum.	1	N	FACW.
FACU	6. Lactuca serriola.	1	N	FAC
FAC	7. Sch. arundinacea.	5.	N	FACU.
UPL	8. Medicago sativa	1	N	UPL.
FACU	9. Poa compressa	1	N	FACU-
FACU.	10. Pascopyrum smithii	1	N.	FACU.
		74 = Total Cover 37. 15.		
	Woody Vine Stratum (Plot size: ____)			
	1.			
	2.			
		____ = Total Cover		
	% Bare Ground in Herb Stratum 25			

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 4 (A)

Total Number of Dominant Species Across All Strata: 4 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 100. (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	____	x 1 = ____
FACW species	____	x 2 = ____
FAC species	____	x 3 = ____
FACU species	____	x 4 = ____
UPL species	____	x 5 = ____
Column Totals:	____ (A)	____ (B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- __ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50%
- __ 3 - Prevalence Index is ≤3.0[1]
- __ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- __ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks: Halophytic vegetation.

SOIL

Sampling Point: SP4.

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox %	Type[1]	Loc[2]	Texture	Remarks
0-2	10YR3/2	100.					Sandy.	many fine roots.
2-12+	10YR2/2.	70.	5YR5/8.	5.	C	PL, M.		Sand loam?
	10YR4/4.	25					Sand.	Very large cobbles throughout.
							sand loam.	0.5-6" diameter.

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

☐ Histosol (A1)
☐ Histic Epipedon (A2)
☐ Black Histic (A3)
☐ Hydrogen Sulfide (A4)
☐ Stratified Layers (A5) (**LRR F**)
☐ 1 cm Muck (A9) (**LRR F, G, H**)
☐ Depleted Below Dark Surface (A11)
☐ Thick Dark Surface (A12)
☐ Sandy Mucky Mineral (S1)
☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
☐ Sandy Gleyed Matrix (S4)
☐ Sandy Redox (S5)
☐ Stripped Matrix (S6)
☐ Loamy Mucky Mineral (F1)
☐ Loamy Gleyed Matrix (F2)
☐ Depleted Matrix (F3)
☑ Redox Dark Surface (F6)
☐ Depleted Dark Surface (F7)
☐ Redox Depressions (F8)
☐ High Plains Depressions (F16)
(**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

☐ 1 cm Muck (A9) (**LRR I, J**)
☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
☐ Dark Surface (S7) (**LRR G**)
☐ High Plains Depressions (F16)
(**LRR H outside of MLRA 72 & 73**)
☐ Reduced Vertic (F18)
☐ Red Parent Material (TF2)
☐ Very Shallow Dark Surface (TF12)
☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type:
Depth (inches):

Could pass, but layer at 7"

Hydric Soil Present? Yes ✓ No ___

Remarks: Soil extremely dry – had to wet with sprayer to reveal indicators. Had to pull out soil samples at each horizon, could not dig "slice" of profile.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

☐ Surface Water (A1)
☐ High Water Table (A2)
☐ Saturation (A3)
☐ Water Marks (B1)
☐ Sediment Deposits (B2)
☐ Drift Deposits (B3)
☐ Algal Mat or Crust (B4)
☐ Iron Deposits (B5)
☐ Inundation Visible on Aerial Imagery (B7)
☐ Water-Stained Leaves (B9)
☐ Salt Crust (B11)
☐ Aquatic Invertebrates (B13)
☐ Hydrogen Sulfide Odor (C1)
☐ Dry-Season Water Table (C2)
☑ Oxidized Rhizospheres on Living Roots (C3) 5%
(**where not tilled**)
☐ Presence of Reduced Iron (C4)
☐ Thin Muck Surface (C7)
☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

☑ Surface Soil Cracks (B6)
☐ Sparsely Vegetated Concave Surface (B8)
☐ Drainage Patterns (B10)
☐ Oxidized Rhizospheres on Living Roots (C3)
(**where tilled**)
☐ Crayfish Burrows (C8)
☐ Saturation Visible on Aerial Imagery (C9)
☑ Geomorphic Position (D2)
☑ FAC-Neutral Test (D5)
☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___
Water Table Present? Yes ___ No ✓ Depth (inches): ___
Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ___

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Depressional surface (D2) also.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder/Boulder Sampling Date: 9-13-19
Applicant/Owner: See Report State: CO Sampling Point: SP-5
Investigator(s): Demarius/Giles Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): hillslope Local relief (concave, convex, none): convex Slope (%): 1-2
Subregion (LRR): G Lat: 39.978419 Long: -105.229211 Datum: NAD 83
Soil Map Unit Name: Nh - Niwot NWI classification: PEM1C
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)
Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___
Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ✓	No	Is the Sampled Area within a Wetland? Yes ✓ No
Hydric Soil Present?	Yes ✓	No	
Wetland Hydrology Present?	Yes ✓	No	

Remarks: Gentle slope.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1.			
2.			
3.			
4.			
	0 = Total Cover		
Sapling/Shrub Stratum (Plot size: 15')			
1.			
2.			
3.			
4.			
5.			
	0 = Total Cover		
Herb Stratum (Plot size: 5')			
1. Agrostis gigantia	35	Y	FACW
2. Daucus carota	5	N	UPL
3. Juncus balticus	5	N	FACW
4. Bromus inermis	10	Y	UPL
5. Symphotrichum virg.	5	N	FACU
6. Poa pratensis	3	N	FACU
7. Juncus saximontanus	1	N	FACW
8. Lotus tenuis	10	Y	FACU
9. Oenothera villosa	1	N	FACU
10. * Sporobolus asper	5	N	FAC
Schoenoplectus pungens	10	Y	OBL
	90 = Total Cover	45	18
Woody Vine Stratum (Plot size:)			
1.			
2.			
	= Total Cover		

% Bare Ground in Herb Stratum 5

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 2 (A)
Total Number of Dominant Species Across All Strata: 4 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 50 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	10	x 1 = 10
FACW species	41	x 2 = 82
FAC species	5	x 3 = 15
FACU species	19	x 4 = 76
UPL species	15	x 5 = 75
Column Totals:	90 (A)	258 (B)

Prevalence Index = B/A = 2.86

Hydrophytic Vegetation Indicators:

N 1 - Rapid Test for Hydrophytic Vegetation
N 2 - Dominance Test is >50% 25.
Y 3 - Prevalence Index is ≤3.0[1] (2.86)
__ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
__ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes X No ___

Remarks: Very diverse plant community at transition from upland to wetland, some deep rooted.

* could not find indicator status for S. asper, used S. airoides FAC

wet

SOIL

Sampling Point: SP5

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-2	10YR3/2	100.					clM	Some roots.
2-6	10YR3/1	80					clM.	Few cobbles.
6-13	10YR 5/4	15	10YR5/6	5	C	M.	clM	
6-13+	10YR 3/1.	90.	10YR4/6.	10.	C	PL,M.	clM.	Blocky structure.
13+								
13+								

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR F**)
- ☐ 1 cm Muck (A9) (**LRR F, G, H**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☑ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) (**LRR I, J**)
- ☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ☐ Dark Surface (S7) (**LRR G**)
- ☐ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Very Shallow Dark Surface (TF12)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type:

Depth (inches):

Hydric Soil Present? Yes ✓ No ____

Remarks: Less cobbles than other sites.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1)
- ☐ Sediment Deposits (B2)
- ☐ Drift Deposits (B3)
- ☐ Algal Mat or Crust (B4)
- ☐ Iron Deposits (B5)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Dry-Season Water Table (C2)
- ☑ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**)
- ☐ Presence of Reduced Iron (C4)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ☐ Surface Soil Cracks (B6)
- ☐ Sparsely Vegetated Concave Surface (B8)
- ☐ Drainage Patterns (B10)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Geomorphic Position (D2)
- ☐ FAC-Neutral Test (D5) NO.
- ☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ____ No ✓ Depth (inches): ____

Water Table Present? Yes ____ No ✓ Depth (inches): ____

Saturation Present? (includes capillary fringe) Yes ____ No ✓ Depth (inches): ____

Wetland Hydrology Present? Yes ✓ No ____

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Does not pass FAC neutral.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC. City/County: Boulder / " Sampling Date: 9/13/19.
Applicant/Owner: See Report State: CO Sampling Point: SP6
Investigator(s): CD, AG Section, Township, Range: S9 T1S R70 W
Landform (hillslope, terrace, etc.): gentle slope Local relief (concave, convex, none): convex Slope (%): 1-2
Subregion (LRR): G Lat: 39.978373 Long: -105.229275 Datum: NAD83
Soil Map Unit Name: GP = Gravel Pits NWI classification: —
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)
Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___
Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ___
Hydric Soil Present? Yes ___ No ✓
Wetland Hydrology Present? Yes ___ No ✓

Is the Sampled Area within a Wetland? Yes ___ No ✓

Remarks: Very broad transition zone to wetlands at this site. This sample is just on transition zone.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ___)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ___)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: 5')			
1. Bromus inermis.	30.	Y	UPL
2. Aster porteri	5	N	UPL
3. Ambrosia psilostachya.	1	N	FACU.
4. Juncus balticus.	15	Y	FACW
5. Carex praegracilis	15	Y	FACW.
6. Poa pratensis.	5	N	FACU
7. Virgulus ericoides.	5	N	FACU
8. Cirsium arvense.	1.	N	FACU.
9. Plantago lanceolata.	1	N	FAC.
10. Daucus carrota	3	N	UPL
= Total Cover	81	40	16.
Woody Vine Stratum (Plot size: ___)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 2

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 2 (A)
Total Number of Dominant Species Across All Strata: 3 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 66. (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	0	x 1 = 0
FACW species	30	x 2 = 60
FAC species	1	x 3 = 3
FACU species	12	x 4 = 48
UPL species	38	x 5 = 190
Column Totals:	81 (A)	301 (B)

Prevalence Index = B/A = 3.71.

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 66.
- ___ 3 - Prevalence Index is ≤3.0[1] NO.
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ___

Remarks: Bare ground & veg do not include litter.
Veg species w/ wetland status are also deep rooted.
Dominated by Smooth Brome (UPL).

SOIL

Sampling Point: SP6 UP

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-4	10YR 4/2	100					cl lM	
4-9+	10YR 3/1	50	10YR 3/6	1	C	M	cl lM	
4-9+	10YR 3/2	45						

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

__ Histosol (A1)
__ Histic Epipedon (A2)
__ Black Histic (A3)
__ Hydrogen Sulfide (A4)
__ Stratified Layers (A5) (**LRR F**)
__ 1 cm Muck (A9) (**LRR F, G, H**)
__ Depleted Below Dark Surface (A11)
__ Thick Dark Surface (A12)
__ Sandy Mucky Mineral (S1)
__ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
__ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
__ Sandy Gleyed Matrix (S4)
__ Sandy Redox (S5)
__ Stripped Matrix (S6)
__ Loamy Mucky Mineral (F1)
__ Loamy Gleyed Matrix (F2)
__ Depleted Matrix (F3)
Y Redox Dark Surface (F6) NO.
__ Depleted Dark Surface (F7)
__ Redox Depressions (F8)
__ High Plains Depressions (F16) (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

__ 1 cm Muck (A9) (**LRR I, J**)
__ Coast Prairie Redox (A16) (**LRR F, G, H**)
__ Dark Surface (S7) (**LRR G**)
__ High Plains Depressions (F16) (**LRR H outside of MLRA 72 & 73**)
__ Reduced Vertic (F18)
__ Red Parent Material (TF2)
__ Very Shallow Dark Surface (TF12)
__ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ________
Depth (inches): ________

Hydric Soil Present? Yes ____ No X

Remarks: Pit located in transition area. Not enough redox (1%) to meet redox dark surface criteria

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

__ Surface Water (A1)
__ High Water Table (A2)
__ Saturation (A3)
__ Water Marks (B1)
__ Sediment Deposits (B2)
__ Drift Deposits (B3)
__ Algal Mat or Crust (B4)
__ Iron Deposits (B5)
__ Inundation Visible on Aerial Imagery (B7)
__ Water-Stained Leaves (B9)
__ Salt Crust (B11)
__ Aquatic Invertebrates (B13)
__ Hydrogen Sulfide Odor (C1)
__ Dry-Season Water Table (C2)
__ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**)
__ Presence of Reduced Iron (C4)
__ Thin Muck Surface (C7)
__ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

__ Surface Soil Cracks (B6)
__ Sparsely Vegetated Concave Surface (B8)
__ Drainage Patterns (B10)
__ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
__ Crayfish Burrows (C8)
__ Saturation Visible on Aerial Imagery (C9)
__ Geomorphic Position (D2)
__ FAC-Neutral Test (D5)
__ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ____ No ✓ Depth (inches): ______
Water Table Present? Yes ____ No ✓ Depth (inches): ______
Saturation Present? (includes capillary fringe) Yes ____ No ✓ Depth (inches): ______

Wetland Hydrology Present? Yes ____ No ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Soil pit is in a transitional area.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder /' Sampling Date: 9/13/19
Applicant/Owner: See Report State: CO Sampling Point: SP-7
Investigator(s): CD, AG Section, Township, Range: S9 T1S R70 W
Landform (hillslope, terrace, etc.): gentle hillslope Local relief (concave, convex, none): convex Slope (%): 1
Subregion (LRR): G Lat: 39.978356 Long: -105.229287 Datum: NAD 83
Soil Map Unit Name: GP - Gravel pits NWI classification: —
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)
Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____
Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ____	No ✓	**Is the Sampled Area within a Wetland?** Yes ____ No ✓
Hydric Soil Present?	Yes ____	No ASSUMED	
Wetland Hydrology Present?	Yes ____	No ASSUMED	

Remarks: Soil + Hydrology assumed Vegetation only recorded. Immediately upslope of SP6.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: 5')			
1. Bromus inermis	40	Y	UPL
2. Aster porteri	30	Y	UPL
3. Ambrosia psilostachya	10	N	FACU
4.			
5.			
6.			
7.			
8.			
9.			
10.			
= Total Cover	80		
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 20

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 0 (A)
Total Number of Dominant Species Across All Strata: 2 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 0 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	0	x 1 = 0
FACW species	0	x 2 = 0
FAC species	0	x 3 = 0
FACU species	10	x 4 = 40
UPL species	70	x 5 = 350
Column Totals:	80 (A)	390 (B)

Prevalence Index = B/A = 4.8

Hydrophytic Vegetation Indicators:

- ____ 1 - Rapid Test for Hydrophytic Vegetation
- ____ 2 - Dominance Test is >50% NO
- ____ 3 - Prevalence Index is ≤3.0[1] NO
- ____ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet) NO
- ____ Problematic Hydrophytic Vegetation[1] (Explain) NO

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ____ No ✓

Remarks: Smooth brome (UPL) dominated.

SOIL

Sampling Point: SP-7

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]	Texture	Remarks

[1] Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2] Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

___ Histosol (A1)
___ Histic Epipedon (A2)
___ Black Histic (A3)
___ Hydrogen Sulfide (A4)
___ Stratified Layers (A5) (**LRR F**)
___ 1 cm Muck (A9) (**LRR F, G, H**)
___ Depleted Below Dark Surface (A11)
___ Thick Dark Surface (A12)
___ Sandy Mucky Mineral (S1)
___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
___ Sandy Gleyed Matrix (S4)
___ Sandy Redox (S5)
___ Stripped Matrix (S6)
___ Loamy Mucky Mineral (F1)
___ Loamy Gleyed Matrix (F2)
___ Depleted Matrix (F3)
___ Redox Dark Surface (F6)
___ Depleted Dark Surface (F7)
___ Redox Depressions (F8)
___ High Plains Depressions (F16)
(**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

___ 1 cm Muck (A9) (**LRR I, J**)
___ Coast Prairie Redox (A16) (**LRR F, G, H**)
___ Dark Surface (S7) (**LRR G**)
___ High Plains Depressions (F16)
(**LRR H outside of MLRA 72 & 73**)
___ Reduced Vertic (F18)
___ Red Parent Material (TF2)
___ Very Shallow Dark Surface (TF12)
___ Other (Explain in Remarks)

[3] Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____________

Depth (inches): ____________

Hydric Soil Present? **Yes** ______ **No** ✓

Remarks: Assumed.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

___ Surface Water (A1)
___ High Water Table (A2)
___ Saturation (A3)
___ Water Marks (B1)
___ Sediment Deposits (B2)
___ Drift Deposits (B3)
___ Algal Mat or Crust (B4)
___ Iron Deposits (B5)
___ Inundation Visible on Aerial Imagery (B7)
___ Water-Stained Leaves (B9)
___ Salt Crust (B11)
___ Aquatic Invertebrates (B13)
___ Hydrogen Sulfide Odor (C1)
___ Dry-Season Water Table (C2)
___ Oxidized Rhizospheres on Living Roots (C3)
(**where not tilled**)
___ Presence of Reduced Iron (C4)
___ Thin Muck Surface (C7)
___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

___ Surface Soil Cracks (B6)
___ Sparsely Vegetated Concave Surface (B8)
___ Drainage Patterns (B10)
___ Oxidized Rhizospheres on Living Roots (C3)
(**where tilled**)
___ Crayfish Burrows (C8)
___ Saturation Visible on Aerial Imagery (C9)
___ Geomorphic Position (D2)
___ FAC-Neutral Test (D5)
___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ______ No ______ Depth (inches): ____________
Water Table Present? Yes ______ No ______ Depth (inches): ____________
Saturation Present? (includes capillary fringe) Yes ______ No ______ Depth (inches): ____________

Wetland Hydrology Present? **Yes** ______ **No** ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Assumed.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/13/19. wet.

Applicant/Owner: See Report State: CO Sampling Point: SP-8

Investigator(s): CD, AG Section, Township, Range: S9 T1S R70 W

Landform (hillslope, terrace, etc.): toe of slope Local relief (concave, convex, none): convex Slope (%): 5

Subregion (LRR): G Lat: 39.971312 Long: -105.23095 Datum: NAD83

Soil Map Unit Name: Nh - Niwot NWI classification: PEM/FX

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)

Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___

Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ✓	No ___	**Is the Sampled Area within a Wetland?**	Yes ✓ No ___
Hydric Soil Present?	Yes ✓	No ___		
Wetland Hydrology Present?	Yes ✓	No ___		

Remarks: Boundary of cattail marsh, toe of slope.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: 15')			
1. FRAXINUS penn sapl.	3	N	FAC.
2. Salix Exigua.	15	Y	FACW.
3.			
4.			
5.			
= Total Cover	18.		9 2
Herb Stratum (Plot size: 5')			
1. Typha angustifolia	20	Y	OBL
2. Dipsacus fullonum	15	Y	FACU.
3. Epilobium ciliatum	5	N	FACW.
4. Juncus interior	5	N	FACW.
5. Carex pellita.	15	Y	OBL
6. Schoeno pungens	15	Y	OBL
7. Juncus balticus.	10.	Y	FACW
8. Geum macrophyllum	3.	N.	FACW
9.			
10.			
= Total Cover	88	44 18.	
Woody Vine Stratum (Plot size: ___)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 10.

Remarks:

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 5 (A)

Total Number of Dominant Species Across All Strata: 6 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 83 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	___	x 1 = ___
FACW species	___	x 2 = ___
FAC species	___	x 3 = ___
FACU species	___	x 4 = ___
UPL species	___	x 5 = ___
Column Totals:	___ (A)	___ (B)

Prevalence Index = B/A = ___

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 83.
- ___ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ___

SOIL

Sampling Point: SP-8.

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix		Redox Features				Texture	Remarks
	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-2	10 YR 4/3	100.					sa lm.	many fine roots
2-7	10YR 3/2	60.					cl lm.	Medium cobbles
	10YR 4/3	33	7.5YR5/8	7	C	PL/M.	cl lm.	1-3"
7-12+	10YR 4/1	50					sa cl. lm.	
	10YR 5/6	30	10YR 5/8	20	C	M.	sa cl. lm.	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

☐ Histosol (A1)
☐ Histic Epipedon (A2)
☐ Black Histic (A3)
☐ Hydrogen Sulfide (A4)
☐ Stratified Layers (A5) (**LRR F**)
☐ 1 cm Muck (A9) (**LRR F, G, H**)
☐ Depleted Below Dark Surface (A11)
☐ Thick Dark Surface (A12)
☐ Sandy Mucky Mineral (S1)
☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
☐ Sandy Gleyed Matrix (S4)
☐ Sandy Redox (S5)
☐ Stripped Matrix (S6)
☐ Loamy Mucky Mineral (F1)
☐ Loamy Gleyed Matrix (F2)
☐ Depleted Matrix (F3)
☑ Redox Dark Surface (F6)
☐ Depleted Dark Surface (F7)
☐ Redox Depressions (F8)
☐ High Plains Depressions (F16)
(**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

☐ 1 cm Muck (A9) (**LRR I, J**)
☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
☐ Dark Surface (S7) (**LRR G**)
☐ High Plains Depressions (F16)
(**LRR H outside of MLRA 72 & 73**)
☐ Reduced Vertic (F18)
☐ Red Parent Material (TF2)
☐ Very Shallow Dark Surface (TF12)
☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type:
Depth (inches):

Hydric Soil Present? Yes ✓ No ___

Remarks: 7 + 20 % redox at 2-7" and 7-12" respectively = F6. redox met.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

☐ Surface Water (A1)
☐ High Water Table (A2)
☐ Saturation (A3)
☐ Water Marks (B1)
☐ Sediment Deposits (B2)
☐ Drift Deposits (B3)
☐ Algal Mat or Crust (B4)
☐ Iron Deposits (B5)
☐ Inundation Visible on Aerial Imagery (B7)
☐ Water-Stained Leaves (B9)
☐ Salt Crust (B11)
☐ Aquatic Invertebrates (B13)
☑ Hydrogen Sulfide Odor (C1)
☐ Dry-Season Water Table (C2)
☑ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**)
☐ Presence of Reduced Iron (C4)
☐ Thin Muck Surface (C7)
☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

☐ Surface Soil Cracks (B6)
☐ Sparsely Vegetated Concave Surface (B8)
☐ Drainage Patterns (B10)
☐ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
☐ Crayfish Burrows (C8)
☐ Saturation Visible on Aerial Imagery (C9)
☐ Geomorphic Position (D2)
☐ FAC-Neutral Test (D5)
☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___
Water Table Present? Yes ___ No ✓ Depth (inches): ___
Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ___

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks:

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC. City/County: Boulder/ " Sampling Date: Sept 18, 2019
Applicant/Owner: See Report State: CO Sampling Point: SP 10
Investigator(s): CD, AG Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): pond fringe. Local relief (concave, convex, none): concave Slope (%): 0-1
Subregion (LRR): G Lat: 39.98195 Long: -105.231131 Datum: NAD83
Soil Map Unit Name: Nh - Niwot NWI classification: PUBGx
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)
Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____
Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ____
Hydric Soil Present? Yes ✓ No ____
Wetland Hydrology Present? Yes ✓ No ____

Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks: Pond wetland fringe, very dry presently.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1. Ulmus pumila.	5	Y	UPL
2.			
3.			
4.			
	5 = Total Cover		
Sapling/Shrub Stratum (Plot size: 15')			
1. Populus deltoides	5	Y	FAC
2. Salix exigua.	3	N.	FACW
3.			
4.			
5.			
	8 = Total Cover		
Herb Stratum (Plot size: 5')			
1. Schoenoplectus pungens	20	Y	OBL
2. Dipsacus fullonum	20	Y	FACU
3. Sporobolus airoides.	20.	Y	FAC
4. Plantago lanceolatus.	1	N	FAC
5. Lotus tenuis	10	Y	FACU
6. Carex pellita	15	Y	OBL
7.			
8.			
9.			
10.			
50/20	86 = Total Cover 43/18		
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
	= Total Cover		
% Bare Ground in Herb Stratum 5 (not litter).			

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 4 (A)
Total Number of Dominant Species Across All Strata: 7 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 57% (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	____	x 1 = ____
FACW species	____	x 2 = ____
FAC species	____	x 3 = ____
FACU species	____	x 4 = ____
UPL species	____	x 5 = ____
Column Totals:	____ (A)	____ (B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- ____ 1 - Rapid Test for Hydrophytic Vegetation
- Y 2 - Dominance Test is >50% (57)
- ____ 3 - Prevalence Index is ≤3.0[1]
- ____ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ____ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes X No ____

UPL
~~FAC~~
OBL
FACW
~~FAC~~
FACU
OBL

Remarks: PEM fringe boardered by PSS and upland.

US Army Corps of Engineers
Great Plains – Version 2.0

SOIL

Sampling Point: SP-10

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

F3

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox %	Type¹	Loc²	Texture	Remarks
0-4	10YR 4/2	100					Silty clay	
4-9	Gley1 3/10Y	98	10YR 4/4	2	C	M		subangular blocky
9-15	"	"	"	"	"	"	"	structure with
								salt crystals

¹Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. ²Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR F**)
- ☐ 1 cm Muck (A9) (**LRR F, G, H**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☒ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils³:

- ☐ 1 cm Muck (A9) (**LRR I, J**)
- ☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ☐ Dark Surface (S7) (**LRR G**)
- ☐ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Very Shallow Dark Surface (TF12)
- ☐ Other (Explain in Remarks)

³Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (inches): ____

Hydric Soil Present? Yes X No ____

Remarks:
difference between 4"-9" and 9"-15" layer is presence of salt in 4-9. Dense roots to 10"

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1)
- ☐ Sediment Deposits (B2)
- ☐ Drift Deposits (B3)
- ☐ Algal Mat or Crust (B4)
- ☐ Iron Deposits (B5)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Aquatic Invertebrates (B13)
- ☒ Hydrogen Sulfide Odor (C1)
- ☐ Dry-Season Water Table (C2)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**)
- ☐ Presence of Reduced Iron (C4)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ☐ Surface Soil Cracks (B6)
- ☐ Sparsely Vegetated Concave Surface (B8) ← playa
- ☐ Drainage Patterns (B10)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☒ Geomorphic Position (D2)
- No FAC-Neutral Test (D5)
- ☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ____ No X Depth (inches): ____

Water Table Present? Yes ____ No X Depth (inches): ____

Saturation Present? Yes ____ No X Depth (inches): ____
(includes capillary fringe)

Wetland Hydrology Present? Yes ✓ No ____

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks:
Low elevation fringe of a pond.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/18/19

Applicant/Owner: See Report State: CO Sampling Point: SP-11

Investigator(s): CD, AG Section, Township, Range: 59 T1S R70W

Landform (hillslope, terrace, etc.): terrace to slope Local relief (concave, convex, none): convex Slope (%): 0-1

Subregion (LRR): G Lat: 39.981978 Long: -105.231171 Datum: NAD83

Soil Map Unit Name: Nh - Niwot NWI classification: —

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ____ No X	**Is the Sampled Area within a Wetland?** Yes ____ No X
Hydric Soil Present?	Yes ____ No X	
Wetland Hydrology Present?	Yes ____ No X	

Remarks: Further upslope from pond fringe.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1. Populus Deltoides	10	Y	FAC
2.			
3.			
4.			
	10 = Total Cover		
Sapling/Shrub Stratum (Plot size: 15')			
1. Ulmus pumila	5	Y	UPL
2.			
3.			
4.			
5.			
	5 = Total Cover		
Herb Stratum (Plot size: 5')			
1. Lactuca serriola	5	N	FAC
2. Lotus tenuis	10	N	FAC
3. Dipsacus fullonum	15	Y	FACU
4. Panicum virgatum	1	N	FAC
5. Bromus inermus	15	Y	UPL
6. Bromus japonicus	5	N	UPL
7. Spirobolus airoides	30	Y	FAC
8. Dactylis glomerata	2	N	FACU
9.			
10.			
	83 = Total Cover 41 16		
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
	= Total Cover		
% Bare Ground in Herb Stratum 2 (not incl. litter)			

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 2 (A)

Total Number of Dominant Species Across All Strata: 5 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 40 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	0	x 1 = 0
FACW species	0	x 2 = 0
FAC species	46	x 3 = 138
FACU species	17	x 4 = 68
UPL species	20	x 5 = 100
Column Totals:	83 (A)	306 (B)

Prevalence Index = B/A = 3.6

Hydrophytic Vegetation Indicators:

N 1 - Rapid Test for Hydrophytic Vegetation

N 2 - Dominance Test is >50% (40)

N 3 - Prevalence Index is ≤3.0[1] (3.6)

N 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)

N Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ____ No X

Remarks: Veg. does not pass any test.

SOIL

Sampling Point: SP-11.

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-8	10YR 3/3	100					Silty clay	many roots to 4"
8-12	10YR 3/1	50					Silty clay	Subangular blocky
	10YR 3/2	40					Silty clay	8-12"
	10YR 4/4	10					Clay loam	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ___ Redox Dark Surface (F6)
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16)
  (MLRA 72 & 73 of LRR H)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16)
  (LRR H outside of MLRA 72 & 73)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ___

Depth (inches): ___

Hydric Soil Present? Yes ___ No X

Remarks: Very cobbly, difficult to dig past 12".

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ___ Oxidized Rhizospheres on Living Roots (C3)
  (where not tilled)
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3)
  (where tilled)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ___ FAC-Neutral Test (D5) NO.
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No / Depth (inches): ___

Water Table Present? Yes ___ No / Depth (inches): ___

Saturation Present? (includes capillary fringe) Yes ___ No / Depth (inches): ___

Wetland Hydrology Present? Yes ___ No X

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: No hydrology.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder/" Sampling Date: 9/18/19

Applicant/Owner: See report State: CO Sampling Point: SP12

Investigator(s): CD Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): Old SBC floodplain Local relief (concave, convex, none): none Slope (%): 0-1

Subregion (LRR): G Lat: 39.982351 Long: -105.226978 Datum: NAD83

Soil Map Unit Name: Nh-Niwot NWI classification: PEM1A

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ____

Hydric Soil Present? Yes ✓ No ____

Wetland Hydrology Present? Yes ✓ No ____

Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks: City of Boulder Property, adjacent to US 36.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: 5')			6P.
1. Agrostis gigantea	25	Y	FACW
2. Dipsacus fullonum	15	Y	FACU
3. Muhlenbergia asperifolia	15	Y	FACW
4. Plantago lanceolata	10	Y	FAC
5. Cirsium arvense	5	N	FACU
6. Mentha arvense	5	N	FACW
7. Juncus balticus	5	N	FACW
8. Oenothera villosa	1	N	FACU
9. Sorgastrum nutans	1	N	FACU
10. Carex pellita	1	N	OBL
Juncus interior	1	N	FACW
= Total Cover	84	44	16
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 5 (not incl. liter)

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 3 (A)

Total Number of Dominant Species Across All Strata: 4 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 75% (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species		x 1 =
FACW species		x 2 =
FAC species		x 3 =
FACU species		x 4 =
UPL species		x 5 =
Column Totals:	(A)	(B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% (75)
- ___ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks: Very diverse vegetation, many more species present at low amounts (not dominant).

SOIL

Sampling Point: SP 12

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-5"+	10YR3/2	95	7.5YR4/6	5	C	M,PL		Many fine roots. Hi OM.

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

__ Histosol (A1)
__ Histic Epipedon (A2)
__ Black Histic (A3)
__ Hydrogen Sulfide (A4)
__ Stratified Layers (A5) (**LRR F**)
__ 1 cm Muck (A9) (**LRR F, G, H**)
__ Depleted Below Dark Surface (A11)
__ Thick Dark Surface (A12)
__ Sandy Mucky Mineral (S1)
__ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
__ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
__ Sandy Gleyed Matrix (S4)
__ Sandy Redox (S5)
__ Stripped Matrix (S6)
__ Loamy Mucky Mineral (F1)
__ Loamy Gleyed Matrix (F2)
__ Depleted Matrix (F3)
✓ Redox Dark Surface (F6)
__ Depleted Dark Surface (F7)
__ Redox Depressions (F8)
__ High Plains Depressions (F16)
(**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

__ 1 cm Muck (A9) (**LRR I, J**)
__ Coast Prairie Redox (A16) (**LRR F, G, H**)
__ Dark Surface (S7) (**LRR G**)
__ High Plains Depressions (F16)
(**LRR H outside of MLRA 72 & 73**)
__ Reduced Vertic (F18)
__ Red Parent Material (TF2)
__ Very Shallow Dark Surface (TF12)
__ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ________
Depth (inches): ________

assumed.
Hydric Soil Present? Yes ✓ No ___

Remarks:
Could not dig deeper than 5" → hard cement @5" ? ROW? (Near gate).

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

__ Surface Water (A1)
__ High Water Table (A2)
__ Saturation (A3)
__ Water Marks (B1)
__ Sediment Deposits (B2)
__ Drift Deposits (B3)
__ Algal Mat or Crust (B4)
__ Iron Deposits (B5)
__ Inundation Visible on Aerial Imagery (B7)
__ Water-Stained Leaves (B9)
__ Salt Crust (B11)
__ Aquatic Invertebrates (B13)
__ Hydrogen Sulfide Odor (C1)
__ Dry-Season Water Table (C2)
✓ Oxidized Rhizospheres on Living Roots (C3)
(**where not tilled**)
__ Presence of Reduced Iron (C4)
__ Thin Muck Surface (C7)
__ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

__ Surface Soil Cracks (B6)
__ Sparsely Vegetated Concave Surface (B8)
__ Drainage Patterns (B10)
__ Oxidized Rhizospheres on Living Roots (C3)
(**where tilled**)
__ Crayfish Burrows (C8)
__ Saturation Visible on Aerial Imagery (C9)
__ Geomorphic Position (D2)
✓ FAC-Neutral Test (D5)
__ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ________
Water Table Present? Yes ___ No ✓ Depth (inches): ________
Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ________

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks:

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC. City/County: Boulder / " Sampling Date: 9/18/19

Applicant/Owner: See Report State: CO Sampling Point: SP 13

Investigator(s): CD. Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): terrace Local relief (concave, convex, none): convex. Slope (%): 0-1

Subregion (LRR): G Lat: 39.981826 Long: -105.225968 Datum: NAD 83

Soil Map Unit Name: Nh-Niwot NWI classification: PEM1A

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ____ No ✓

Hydric Soil Present? Yes ____ No ✓

Wetland Hydrology Present? Yes ____ No ✓

Is the Sampled Area within a Wetland? Yes ____ No ✓

Remarks: sample in between 2 ditches (dry), several feet above adjacent areas. Old alluvial floodplain soils.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: ____)			
1. Andropogon gerardii	50	Y	FACU
2. Panicum virgatum	5	N	FAC
3. Apocynum cannabinum	15	N	FAC
4. Sorghastrum nutans	10	N	FACU
5. Rosa arkansana	5	N	FACU
6. Achillea millefolia	1	N	FACU
7. Artemesia ludoviciana	1	N	UPL
8. Symphyotrichum ericoides	1	N	FACU
9.			
10.			
= Total Cover	88		
	44 17		
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 1 (not incl. litter)

Remarks: Relic tallgrass system on old alluvial floodplain. Dominated by big bluestem.

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 0 (A)

Total Number of Dominant Species Across All Strata: 1 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 0. (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	0	x 1 = 0
FACW species	0	x 2 = 0
FAC species	20	x 3 = 60
FACU species	67	x 4 = 268
UPL species	1	x 5 = 5
Column Totals:	88 (A)	329 (B)

Prevalence Index = B/A = 3.74

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ___ 2 - Dominance Test is >50% ND (0)
- ___ 3 - Prevalence Index is ≤3.0[1] ND (3.7)
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ____ No ✓

SOIL

Sampling Point: SP 13

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-5	10YR4/2	100.					cl. lm.	many fine roots.
5-10	10YR 3/3.	100.					cl. lm.	gran. structure.
10-12+	10YR 4/2	60	10YR4/6	40	C	M.	clay	
↓								
								large cobbles
								throughout
								0.5– 4" diam.

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ___ Redox Dark Surface (F6) No.
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16) (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16) (**LRR H outside of MLRA 72 & 73**)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: /

Depth (inches): /

Hydric Soil Present? Yes ___ No ✓

Remarks: Soil very dry, had to wet with mister. Clay layer at 10". Does not meet redox dark surface b/c not at least 4" in top 12".

(Valmont clay loam).

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**) NO.
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ___ FAC-Neutral Test (D5) NO.
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___

Water Table Present? Yes ___ No ✓ Depth (inches): ___

Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ___

Wetland Hydrology Present? Yes ___ No ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: No oxidized rhizospheres.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC. City/County: Boulder / " Sampling Date: 9/18/19.
Applicant/Owner: See Report State: CO Sampling Point: SP 14.
Investigator(s): CD Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): gentle hillslope Local relief (concave, convex, none): convex Slope (%): 2.
Subregion (LRR): G Lat: 39.981597 Long: -105.225169 Datum: NAD 83
Soil Map Unit Name: Nh-Niwot NWI classification: PEM1A
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)
Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____
Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N. (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ✓	No ✓	**Is the Sampled Area within a Wetland?** Yes ____ No ✓
Hydric Soil Present?	Yes ____	No ✓	
Wetland Hydrology Present?	Yes ____	No ✓	

Remarks: Transitional pit. Turns to WL to the west of pit. Very close to wetland/upland line.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: ____)			
1. Asclepias speciosus	2	N	FAC
2. Apocynum cannabinum	15	N	FAC
3. Agrostis gigantea	15	Y	FACW
4. Festuca arundinaceae	20	Y	FACU
5. Juncus balticus	15	Y	FACW
6. Carex pellita	10	Y	OBL
7. Sorghastrum nutans	10	Y	FACU
8. Plantago lanceolata	5	N	FAC
9. Rosa arkansana	2	N	FACU
10. Geum macrophyllum	1	N	FACW
Panicum virgatum	1	N	FAC
= Total Cover	86.	43	17
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 1

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 3 (A)
Total Number of Dominant Species Across All Strata: 5 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 60% (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species		x 1 =
FACW species		x 2 =
FAC species		x 3 =
FACU species		x 4 =
UPL species		x 5 =
Column Totals:	(A)	(B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- ____ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% (60).
- ____ 3 - Prevalence Index is ≤3.0[1]
- ____ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ____ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks:

SOIL

Sampling Point: SP 14

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-6	10YR3/1	100					sacl lu	Many fine roots
6-10	10YR3/2	98	10YR6/8	2	C	M	cl. lm	"
10-14	10YR4/4	60	lighter					
↓	10YR4/2	40	darker				sacl lm.	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11) No
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ___ Redox Dark Surface (F6) No
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16) (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16) (**LRR H outside of MLRA 72 & 73**)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (inches): ____

Hydric Soil Present? Yes ___ No X

Remarks: Deep dark soil, but does not meet criteria: Earthworms. Not enough redox to meet F6. Needs 60% or more chroma ≤2 for All.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**)
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ✓ FAC-Neutral Test (D5)
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ____

Water Table Present? Yes ___ No ✓ Depth (inches): ____

Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ____

Wetland Hydrology Present? Yes ___ No X

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks:

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBL. City/County: Boulder / " Sampling Date: 9/18/19.
Applicant/Owner: See Report State: CO Sampling Point: SP 15
Investigator(s): CD. Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): Swale Local relief (concave, convex, none): concave Slope (%): 0
Subregion (LRR): G Lat: 39.981321 Long: -105.22434 Datum: NAD83
Soil Map Unit Name: Nn-Niwot NWI classification: PEM1A
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)
Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____
Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ____
Hydric Soil Present? Yes ✓ No ____
Wetland Hydrology Present? Yes ✓ No ____
Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks: Old alluvial floodplain of SBC, ridge + swale landscape, adjacent to US36.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: ____)			
1. Asclepias incarnata	15	Y	FACW.
2. Dipsacus fullonum	10.	Y	FACU
3. Carex pellita.	25	Y	OBL.
4. Mentha sp.	5	N	FACW.
5. Rosa arkansana	5	N	FACU
6. Juncus longistylis.	2	N	FACW.
7. Mentha arvense.	5	N	FACW
8. Geum macrophyllum.	5	N	FACW
9. Muhlenbergia asperifolia.	1	N.	FACW
10. Epilobium ciliatum.	10.	Y	FACW
Polygonum sp.	10.	Y	OBL
= Total Cover			
Woody Vine Stratum (Plot size: ____)			
1. Spartina pectinata	1	N.	FACW
2.			
= Total Cover	94.		

% Bare Ground in Herb Stratum 2

46 18

Remarks: Swamp milkweed in seed dispersal.

no smell
opp.
Axil clust.

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 4 (A)
Total Number of Dominant Species Across All Strata: 5 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 80. (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:	
OBL species		x 1 =	
FACW species		x 2 =	
FAC species		x 3 =	
FACU species		x 4 =	
UPL species		x 5 =	
Column Totals:	(A)		(B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- ____ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 80.
- ____ 3 - Prevalence Index is ≤3.0[1]
- ____ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ____ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

SOIL

Sampling Point: SP 15

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix		Redox Features				Texture	Remarks
	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-4	10YR 3/2	100.						many fine roots.
4-10	10YR 3/1	85	10YR 3/6.	15	C	PLM	cl lm.	
10-16	10YR 2.5/1	75	7.5YR 4/4	25	c	M.	sa cl lm.	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ✓ Redox Dark Surface (F6)
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16)
  (MLRA 72 & 73 of LRR H)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16)
  (LRR H outside of MLRA 72 & 73)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ______
Depth (inches): ______

Hydric Soil Present? Yes ✓ No ______

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ✓ Oxidized Rhizospheres on Living Roots (C3)
  (where not tilled)
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3)
  (where tilled)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ___ FAC-Neutral Test (D5)
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ______ No ✓ Depth (inches): ______
Water Table Present? Yes ______ No ✓✓ Depth (inches): ______
Saturation Present? Yes ______ No ______ Depth (inches): ______
(includes capillary fringe)

Wetland Hydrology Present? Yes ✓ No ______

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks:

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/19/19

Applicant/Owner: See Report State: CO Sampling Point: SP19

Investigator(s): CD Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): hillslope Local relief (concave, convex, none): convex Slope (%): 10

Subregion (LRR): G Lat: 39.984147 Long: -105.226771 Datum: NAD83

Soil Map Unit Name: Nh- Niwot NWI classification: PEM1C

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ✓	No ____
Hydric Soil Present?	Yes ✓	No ____
Wetland Hydrology Present?	Yes ✓	No ____

Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks: Hillslope adjacent to seep from BoCity OS to Viele ditch.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: ____)			
1. Helianthus nuttallii	25	Y	FACW
2. Solidago gigantea	5	N	FAC
3. Symphyotrichum lanceolatum	2	N	FACW
4. Juncus balticus	15	Y	FACW
5. Carex pellita	15	Y	OBL
6. Dipsacus fullonum	5	N	FACU
7. Agrostis gigantea	10	Y	FACW
8. Rosa arkansana	5	N	FACU
9. Poa pratensis	5	N	FACU
10. Juncus longistylus	1	N	FACW
= Total Cover	88	44 17	
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 5

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 4 (A)

Total Number of Dominant Species Across All Strata: 4 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 100. (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	____	x 1 = ____
FACW species	____	x 2 = ____
FAC species	____	x 3 = ____
FACU species	____	x 4 = ____
UPL species	____	x 5 = ____
Column Totals:	____ (A)	____ (B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- ✓ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 100.
- ___ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks: Juncus interior and Agrostis scabra also present. Blue eye grass (Sisirchrium).

PL + crack spearmint, Salix?, ← Seep area: Polygonum, teasel, epicil, Juncus x5

Willow. Carex, goldenrod, mentha, cerlan, typang, agrslab, geum,

US Army Corps of Engineers

Great Plains – Version 2.0

SOIL

Sampling Point: SP 19

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-4.	10YR 3/2	100					sa loam	Many medium rhizomes.
4-12	10YR 3/2.	70	10YR 5/8	30	C	PL, M	sa cl.	
12-16.	10YR 4/4.	50.						
	10YR 3/2.	40	10YR 5/6.	10.	C	M.	loamy sand.	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR F**)
- ☐ 1 cm Muck (A9) (**LRR F, G, H**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☑ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ High Plains Depressions (F16) **(MLRA 72 & 73 of LRR H)**

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) (**LRR I, J**)
- ☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ☐ Dark Surface (S7) (**LRR G**)
- ☐ High Plains Depressions (F16) **(LRR H outside of MLRA 72 & 73)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Very Shallow Dark Surface (TF12)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ______
Depth (inches): ______

Hydric Soil Present? Yes ☑ No ☐

Remarks: Many large cobbles throughout 0.5-5".

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1)
- ☐ Sediment Deposits (B2)
- ☐ Drift Deposits (B3)
- ☐ Algal Mat or Crust (B4)
- ☐ Iron Deposits (B5)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Dry-Season Water Table (C2)
- ☑ Oxidized Rhizospheres on Living Roots (C3) **(where not tilled)** 75%.
- ☐ Presence of Reduced Iron (C4)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ☐ Surface Soil Cracks (B6)
- ☐ Sparsely Vegetated Concave Surface (B8)
- ☐ Drainage Patterns (B10)
- ☐ Oxidized Rhizospheres on Living Roots (C3) **(where tilled)**
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Geomorphic Position (D2)
- ☐ FAC-Neutral Test (D5)
- ☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:
Surface Water Present? Yes ☐ No ☑ Depth (inches): ______
Water Table Present? Yes ☐ No ☑ Depth (inches): ______
Saturation Present? (includes capillary fringe) Yes ☐ No ☑ Depth (inches): ______

Wetland Hydrology Present? Yes ☑ No ☐

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: 20% ox. rhizospheres

Overstory of whole site: Elaeag. ang. Prun. amer. Fraxt. penn.
Sal. ex., Sal. frag., Populus,

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/19/19
Applicant/Owner: See Report State: CO Sampling Point: SP 20
Investigator(s): CD Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): hillslope Local relief (concave, convex, none): convex Slope (%): 10
Subregion (LRR): G Lat: 39.984146 Long: -105.226748 Datum: NAD83
Soil Map Unit Name: Nh-Niwot NWI classification: PEM1C
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)
Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____
Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N. (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ____ No ✓ Hydric Soil Present? Yes ____ No ✓ Wetland Hydrology Present? Yes ____ No ✓	Is the Sampled Area within a Wetland?	Vegetation only. Yes ____ No ✓

Remarks: Directly upslope of SP19. (~5 ft.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ____)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: ____)			
1. Panicum virgatum	5	N	FAC
2. Sorghastrum nutans	5	N	FACU
3. Rosa arkansana	20	Y	FACU
4. Dipsacus fullonum	5	N	FACU
5. Andropogon gerardii	10	Y	FACU
6. Juncus balticus	15	Y	FACW
7. Carex pellita	15	Y	OBL
8. Agrostis gigantea	2	N	FACW
9. Bromus inermis	1	N	UPL
10. Poa pratensis	10	Y	FACU
= Total Cover	88	44 17.	
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
= Total Cover			
% Bare Ground in Herb Stratum 10			

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 2 (A)
Total Number of Dominant Species Across All Strata: 5 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 40 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	15	x 1 = 15
FACW species	17	x 2 = 34
FAC species	5	x 3 = 15
FACU species	50	x 4 = 200
UPL species	1	x 5 = 5
Column Totals:	88 (A)	269 (B)

Prevalence Index = B/A = 3.05

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ___ 2 - Dominance Test is >50% NO
- ___ 3 - Prevalence Index is ≤3.0[1] No
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ____ No ✓

Remarks:

33
88 ⟌269
264
5

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SOIL

Sampling Po SP 20

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix		Redox Features				Texture	Remarks
	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-5	10YR3/2	100.					Sa LM.	Many fine roots
5-10	10YR 3/2	100					Sa lm.	many cobbles
10-16+	10YR5/4	100					coarse sand	throughout
								.1-3" diameter.

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR F**)
- ☐ 1 cm Muck (A9) (**LRR F, G, H**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) (**LRR I, J**)
- ☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ☐ Dark Surface (S7) (**LRR G**)
- ☐ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Very Shallow Dark Surface (TF12)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ______

Depth (inches): ______

Hydric Soil Present? Yes ______ No ✓

Remarks: Moistened soil for color w/ spray bottle. No redox

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1)
- ☐ Sediment Deposits (B2)
- ☐ Drift Deposits (B3)
- ☐ Algal Mat or Crust (B4)
- ☐ Iron Deposits (B5)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Dry-Season Water Table (C2)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**)
- ☐ Presence of Reduced Iron (C4)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ☐ Surface Soil Cracks (B6)
- ☐ Sparsely Vegetated Concave Surface (B8)
- ☐ Drainage Patterns (B10)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Geomorphic Position (D2)
- ☐ FAC-Neutral Test (D5)
- ☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ______ No ✓ Depth (inches): ______

Water Table Present? Yes ______ No ✓ Depth (inches): ______

Saturation Present? Yes ______ No ✓ Depth (inches): ______
(includes capillary fringe)

Wetland Hydrology Present? Yes ______ No ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Upslope of where seep begins.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/23/19
Applicant/Owner: See Report State: CO Sampling Point: SP-21
Investigator(s): Carla, KL Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): swale. Local relief (concave, convex, none): concave Slope (%): 0 – 1
Subregion (LRR): G Lat: 39.980362 Long: -105.234603 Datum: NAD 83
Soil Map Unit Name: GP - Gravel pits NWI classification: —
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)
Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes X No ____
Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ____
Hydric Soil Present? Yes ____ No ✓
Wetland Hydrology Present? Yes ____ No ✓

Is the Sampled Area within a Wetland? Yes ____ No ✓

Remarks: Willow overstory w/ smooth brome understory
Passes vegetation b/c of willow overstory (deep rooted).

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1. Populus deltoides	15	Y	FAC
2.			
3.			
4.			
	15 = Total Cover		
Sapling/Shrub Stratum (Plot size: 15')			
1. Salix exigua	75	Y	FACW
2. Fraxinus pennsylvanica	5	N	FAC
3.			
4.			
5.			
	80 = Total Cover		
Herb Stratum (Plot size: 5')			
1. Bromus inermis	30	Y	UPL
2. Cirsium arvense	5	N	FACU
3. Cynoglossum officinalis	2	N	FACU
4. Juncus sp.	1	N	OBL
5. Nepeta cataria	1	N	FACU
6.			
7.			
8.			
9.			
10.			
	39 = Total Cover 20 8.		
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
	0 = Total Cover		

FAC
FACW
UPL.

% Bare Ground in Herb Stratum 40

Remarks: Liter included in bare ground.

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 2 (A)
Total Number of Dominant Species Across All Strata: 3 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 66 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	1	x 1 = 1
FACW species	75	x 2 = 150
FAC species	20	x 3 = 60
FACU species	8	x 4 = 32
UPL species	30.	x 5 = 150.
Column Totals:	134. (A)	393. (B)

Prevalence Index = B/A = <3.

2
134 | 393
268.

134
x 3
402

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50%
- ✓ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

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SOIL

Sampling Point: SP-21

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-4	10YR3/2						Sandy Loam	large cobbles.
4-6+	10YR 4/3.	50	10YR5/6.	2%.	C	M.		large cobbles
	10YR 3/2	48.						

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ___ Redox Dark Surface (F6) NO.
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ___

Depth (inches): ___

Hydric Soil Present? Yes ___ No ✓

Remarks: Very large cobbles (1-6"), difficult to dig past 6". We don't meet 5% redox for redox dark surface. Redox are prominent.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ___ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**) NO.
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ___ FAC-Neutral Test (D5) NO.
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___

Water Table Present? Yes ___ No ✓ Depth (inches): ___

Saturation Present? Yes ___ No ✓ Depth (inches): ___
(includes capillary fringe)

Wetland Hydrology Present? Yes ___ No ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Soil is very dry.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/23/19

Applicant/Owner: See Report State: CO Sampling Point: SP-22

Investigator(s): CD, KLH Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): swale Local relief (concave, convex, none): concave Slope (%): 1

Subregion (LRR): G Lat: 39.971342 Long: -105.228205 Datum: NAD83

Soil Map Unit Name: Nh - Niwot NWI classification: ______

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ✓	No ____
Hydric Soil Present?	Yes ✓	No ____
Wetland Hydrology Present?	Yes ✓	No ____

Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks:

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ______)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ______)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: 5')			
1. Eleocharis palustris	15	N	OBL
2. Typha angustifolia	2	N	OBL
3. Sporobolus airoides	55	Y	FAC
4. Juncus interior	5	N	FACW
5. Spartina pectinata	5	N	FACW
6. Rumex crispus	1	N	FAC
7. Bromus japonicus	1	N	UPL
8. Neolepia campestre	1	N	UPL
9.			
10.			
= Total Cover	85 42 16		
Woody Vine Stratum (Plot size: ______)			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 15

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 1 (A)

Total Number of Dominant Species Across All Strata: 1 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 100 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	17	x 1 = 17
FACW species	10	x 2 = 20
FAC species	56	x 3 = 168
FACU species	0	x 4 = 0
UPL species	2	x 5 = 10
Column Totals:	85 (A)	215 (B)

Prevalence Index = B/A = 2.52

1
56
3
168

2.
85 | 215
170
45.

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50%
- ✓ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks: Perimeter veg includes mostly BRIN, community documented in other sample points.

SOIL

Sampling Point: SP-22

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-4	10YR2/2	95	10YR4/6	5	C	M	clLM	Could not dig below
4-10	GLEY1 3/10Y	70						10, large cobbles
	10YR 3/4	30						

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ✓ Redox Dark Surface (F6)
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ______

Depth (inches): ______

Hydric Soil Present? Yes ✓ No ___

Remarks: Many large cobbles, 0.5"-6" diameter. Dry soil. Soil wetted w/ sprayer to evaluate hydric soil indicators

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ✓ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**) 2%
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ✓ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ___ Geomorphic Position (D2)
- ___ FAC-Neutral Test (D5) NA
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ______

Water Table Present? Yes ___ No ✓ Depth (inches): ______

Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ______

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Cannot pass FAC neutral b/c only one dominant species that is FAC.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 9/23/19.

Applicant/Owner: See Report State: CO Sampling Point: SP-23

Investigator(s): CD, KLH. Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): former ditch/swale Local relief (concave, convex, none): concave Slope (%): 1

Subregion (LRR): G Lat: 39.971603 Long: -105.229477 Datum: NAD83.

Soil Map Unit Name: Nh - Niwot NWI classification: R4SBCx

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N. (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ✓	No ____
Hydric Soil Present?	Yes ✓	No ____
Wetland Hydrology Present?	Yes ✓	No ____

Is the Sampled Area within a Wetland? Yes ✓ No ____

Remarks: PSS w/ cottonwood overstory. Sample in old filled in ditch, now a swale.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1. Populus deltoides	40	Y	FAC
2.			
3.			
4.			
	40 = Total Cover		
Sapling/Shrub Stratum (Plot size: 15')			
1. Salix exigua.	30.	Y.	FACW
2. Salix amygdaloides	1	N	FACW.
3.			
4.			
5.			
	31 = Total Cover		
Herb Stratum (Plot size: 5')			
1. Agrostis gigantea	20.	Y.	FACW.
2. Juncus interior	10.	Y.	FACW
3. Symphyotrichum lanceolatum	3	N.	FACW
4. Carex pellita.	10.	Y	OBL
5. Juncus balticus.	10.	Y	FACW
6. Carex nebrascensis	15	Y	OBL.
7. Schedonorus arundinaceus	15	Y	FACU.
8.			
9.			
10.			
	83. = Total Cover 41 16		
Woody Vine Stratum (Plot size: ____)			
1.			
2.			
	____ = Total Cover		

% Bare Ground in Herb Stratum 10.

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–):	7	(A)
Total Number of Dominant Species Across All Strata:	8	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	87	(A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	____	x 1 = ____
FACW species	____	x 2 = ____
FAC species	____	x 3 = ____
FACU species	____	x 4 = ____
UPL species	____	x 5 = ____
Column Totals:	____ (A)	____ (B)

Prevalence Index = B/A = ____

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 87
- ___ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ____

Remarks:

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SOIL

Sampling Point: SP-23

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix Color (moist)	Matrix %	Redox Features Color (moist)	Redox Features %	Type[1]	Loc[2]	Texture	Remarks
0-6	10YR 3/2.	95	10YR 3/6	5	C	M.	sacllu	
6-12	GLEY 1 2.5/10Y	90	10YR 4/4.	10	C	M.	cllm.	large cobbles

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ___ Histosol (A1)
- ___ Histic Epipedon (A2)
- ___ Black Histic (A3)
- ___ Hydrogen Sulfide (A4)
- ___ Stratified Layers (A5) (**LRR F**)
- ___ 1 cm Muck (A9) (**LRR F, G, H**)
- ___ Depleted Below Dark Surface (A11)
- ___ Thick Dark Surface (A12)
- ___ Sandy Mucky Mineral (S1)
- ___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ___ Sandy Gleyed Matrix (S4)
- ___ Sandy Redox (S5)
- ___ Stripped Matrix (S6)
- ___ Loamy Mucky Mineral (F1)
- ___ Loamy Gleyed Matrix (F2)
- ___ Depleted Matrix (F3)
- ✓ Redox Dark Surface (F6)
- ___ Depleted Dark Surface (F7)
- ___ Redox Depressions (F8)
- ___ High Plains Depressions (F16) (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ___ 1 cm Muck (A9) (**LRR I, J**)
- ___ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ___ Dark Surface (S7) (**LRR G**)
- ___ High Plains Depressions (F16) (**LRR H outside of MLRA 72 & 73**)
- ___ Reduced Vertic (F18)
- ___ Red Parent Material (TF2)
- ___ Very Shallow Dark Surface (TF12)
- ___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type:

Depth (inches):

Hydric Soil Present? Yes ✓ No ___

Remarks: Many large cobbles throughout.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ___ Surface Water (A1)
- ___ High Water Table (A2)
- ___ Saturation (A3)
- ___ Water Marks (B1)
- ___ Sediment Deposits (B2)
- ___ Drift Deposits (B3)
- ___ Algal Mat or Crust (B4)
- ___ Iron Deposits (B5)
- ___ Inundation Visible on Aerial Imagery (B7)
- ___ Water-Stained Leaves (B9)
- ___ Salt Crust (B11)
- ___ Aquatic Invertebrates (B13)
- ___ Hydrogen Sulfide Odor (C1)
- ___ Dry-Season Water Table (C2)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**)
- ___ Presence of Reduced Iron (C4)
- ___ Thin Muck Surface (C7)
- ___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ___ Surface Soil Cracks (B6)
- ___ Sparsely Vegetated Concave Surface (B8)
- ___ Drainage Patterns (B10)
- ___ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
- ___ Crayfish Burrows (C8)
- ___ Saturation Visible on Aerial Imagery (C9)
- ✓ Geomorphic Position (D2)
- ✓ FAC-Neutral Test (D5)
- ___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No ✓ Depth (inches): ___

Water Table Present? Yes ___ No ✓ Depth (inches): ___

Saturation Present? (includes capillary fringe) Yes ___ No ✓ Depth (inches): ___

Wetland Hydrology Present? Yes ✓ No ___

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Passes two secondary indicators.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder/Boulder Sampling Date: 10/3/19

Applicant/Owner: See Report State: CO Sampling Point: SP-25

Investigator(s): Carla/KL Section, Township, Range: S9 T1S R70W

Landform (hillslope, terrace, etc.): Swale Local relief (concave, convex, none): Concave Slope (%): ~1

Subregion (LRR): G Lat: 39.982767 Long: -105.227398 Datum: NAD83

Soil Map Unit Name: Nh - Niwot NWI classification: —

Are climatic / hydrologic conditions on the site typical for this time of year? Yes X No ____ (If no, explain in Remarks.)

Are Vegetation ____, Soil ____, or Hydrology ____ significantly disturbed? N Are "Normal Circumstances" present? Yes X No ____

Are Vegetation ____, Soil ____, or Hydrology ____ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes X No ____

Hydric Soil Present? Yes X No ____

Wetland Hydrology Present? Yes ____ No X

Is the Sampled Area within a Wetland? Yes ____ No X

Remarks: Area is marginal, but appears to be drying.

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size: 30')	Absolute % Cover	Dominant Species?	Indicator Status
1. Populus deltoides	5	Y	FAC
2. Salix amygdaloides	10	Y	FACW
3. JUNIPERUS COMMUNIS	3	N	UPL
4.			
	18	= Total Cover	
Sapling/Shrub Stratum (Plot size: 15')			
1. Salix exigua	70	Y	FACW
2.			
3.			
4.			
5.			
	70	= Total Cover	
Herb Stratum (Plot size: 5')			
1. Dipsacus fullonum	30	Y	FACU
2. Cirsium arvense	10	Y	FACU
3. Agropyron cristatum	3	N	UPL
4.			
5.			
6.			
7.			
8.			
9.			
10.			
	43	= Total Cover	
Woody Vine Stratum (Plot size: 15')			
1.			
2.			
		= Total Cover	

% Bare Ground in Herb Stratum 35

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 3 (A)

Total Number of Dominant Species Across All Strata: 5 (B)

Percent of Dominant Species That Are OBL, FACW, or FAC: 60% (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	—	x 1 = 0
FACW species	80	x 2 = 160
FAC species	5	x 3 = 15
FACU species	40	x 4 = 160
UPL species	6	x 5 = 30
Column Totals:	95 (A)	365 (B)

Prevalence Index = B/A = 4

Hydrophytic Vegetation Indicators:

____ 1 - Rapid Test for Hydrophytic Vegetation

X 2 - Dominance Test is >50% 60

____ 3 - Prevalence Index is ≤3.0[1]

____ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)

____ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes X No ____

Remarks: Drying willow community.

SOIL

Sampling Point: SP-25

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-6	10YR 2/2	95	10YR 4/6	5	C	M	Clay Loam	
6-13	10YR 2/2	100					Clay Loam	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

___ Histosol (A1)
___ Histic Epipedon (A2)
___ Black Histic (A3)
___ Hydrogen Sulfide (A4)
___ Stratified Layers (A5) (**LRR F**)
___ 1 cm Muck (A9) (**LRR F, G, H**)
___ Depleted Below Dark Surface (A11)
___ Thick Dark Surface (A12)
___ Sandy Mucky Mineral (S1)
___ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
___ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
___ Sandy Gleyed Matrix (S4)
___ Sandy Redox (S5)
___ Stripped Matrix (S6)
___ Loamy Mucky Mineral (F1)
___ Loamy Gleyed Matrix (F2)
___ Depleted Matrix (F3)
X Redox Dark Surface (F6)
___ Depleted Dark Surface (F7)
___ Redox Depressions (F8)
___ High Plains Depressions (F16) (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

___ 1 cm Muck (A9) (**LRR I, J**)
___ Coast Prairie Redox (A16) (**LRR F, G, H**)
___ Dark Surface (S7) (**LRR G**)
___ High Plains Depressions (F16) (**LRR H outside of MLRA 72 & 73**)
___ Reduced Vertic (F18)
___ Red Parent Material (TF2)
___ Very Shallow Dark Surface (TF12)
___ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ___
Depth (inches): ___

Hydric Soil Present? Yes ✓ No ___

Remarks: Many large cobbles below 4 inches.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

___ Surface Water (A1)
___ High Water Table (A2)
___ Saturation (A3)
___ Water Marks (B1)
___ Sediment Deposits (B2)
___ Drift Deposits (B3)
___ Algal Mat or Crust (B4)
___ Iron Deposits (B5)
___ Inundation Visible on Aerial Imagery (B7)
___ Water-Stained Leaves (B9)
___ Salt Crust (B11)
___ Aquatic Invertebrates (B13)
___ Hydrogen Sulfide Odor (C1)
___ Dry-Season Water Table (C2)
___ Oxidized Rhizospheres on Living Roots (C3) (**where not tilled**) NO
___ Presence of Reduced Iron (C4)
___ Thin Muck Surface (C7)
___ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

___ Surface Soil Cracks (B6)
___ Sparsely Vegetated Concave Surface (B8)
___ Drainage Patterns (B10)
___ Oxidized Rhizospheres on Living Roots (C3) (**where tilled**)
___ Crayfish Burrows (C8)
___ Saturation Visible on Aerial Imagery (C9)
X Geomorphic Position (D2)
___ FAC-Neutral Test (D5) N
___ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ___ No X Depth (inches): ___
Water Table Present? Yes ___ No X Depth (inches): ___
Saturation Present? (includes capillary fringe) Yes ___ No X Depth (inches): ___

Wetland Hydrology Present? Yes ___ No X

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: No evidence of WL hydrology, no oxidized rhizosphere

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 10/3/19
Applicant/Owner: See Report State: CO Sampling Point: SP-26
Investigator(s): CD/KL. Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): SBC floodplain Local relief (concave, convex, none): Concave. Slope (%): 1
Subregion (LRR): G Lat: 39.978379 Long: -105.222575 Datum: NAD83
Soil Map Unit Name: Nh-Niwot NWI classification: RpIFO.
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)
Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___
Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N. (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ___
Hydric Soil Present? Yes ✓ No ___
Wetland Hydrology Present? Yes ___ No ✓

Is the Sampled Area within a Wetland? Yes ___ No ✓

Remarks:

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1. Populus deltoides	30	Y	FAC.
2. Salix amygdaloides.	1	N	
3.			
4.			
	30 = Total Cover		
Sapling/Shrub Stratum (Plot size: 15')			
1.			
2.			
3.			
4.			
5.			
	= Total Cover		
Herb Stratum (Plot size: 5')			
1. Schedonorus arundin.	40	Y	FACU.
2. Agrostis gigantea	5	N	FACW
3. Thermopsis divaricarpa	20.	Y	FAC
4. Dactylis glomerata	2	N	FACU.
5. Cirsium arvense.	2	N	FACU.
6. Asclepias speciosa.	5	N	FAC.
7. Plantago lanceolata	2	N	FAC.
8. Poa pratensis	10	N.	FACU.
9.			
10.			
	86. = Total Cover 43 17.		
Woody Vine Stratum (Plot size: ___)			
1.			
2.			
	= Total Cover		

% Bare Ground in Herb Stratum 3 (not incl. litter)

Remarks:

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 2 (A)
Total Number of Dominant Species Across All Strata: 3 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 66. (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species	0	x 1 =
FACW species	5	x 2 =
FAC species	57	x 3 =
FACU species	57.	x 4 =
UPL species	0	x 5 =
Column Totals:	(A)	(B)

Prevalence Index = B/A = ___

Hydrophytic Vegetation Indicators:

___ 1 - Rapid Test for Hydrophytic Vegetation
✓ 2 - Dominance Test is >50% 66.
___ 3 - Prevalence Index is ≤3.0[1]
___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ___

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SOIL

Sampling Point: SP26

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix		Redox Features				Texture	Remarks
	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-6	10YR 2/2	100					sacl.lm	most roots to 11".
6-11	10YR 3/2	100					Sa loam	
11-16	10YR 3/2	20	10YR 6/2	60	D	M.	lo san	
			10YR 6/8	20	C	M.	↓	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

☐ Histosol (A1)
☐ Histic Epipedon (A2)
☐ Black Histic (A3)
☐ Hydrogen Sulfide (A4)
☐ Stratified Layers (A5) (**LRR F**)
☐ 1 cm Muck (A9) (**LRR F, G, H**)
☐ Depleted Below Dark Surface (A11)
☐ Thick Dark Surface (A12)
☐ Sandy Mucky Mineral (S1)
☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
☐ Sandy Gleyed Matrix (S4)
☐ Sandy Redox (S5)
☐ Stripped Matrix (S6)
☐ Loamy Mucky Mineral (F1)
☐ Loamy Gleyed Matrix (F2)
☐ Depleted Matrix (F3)
☐ Redox Dark Surface (F6)
☑ Depleted Dark Surface (F7)
☐ Redox Depressions (F8)
☐ High Plains Depressions (F16)
(**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

☐ 1 cm Muck (A9) (**LRR I, J**)
☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
☐ Dark Surface (S7) (**LRR G**)
☐ High Plains Depressions (F16)
(**LRR H outside of MLRA 72 & 73**)
☐ Reduced Vertic (F18)
☐ Red Parent Material (TF2)
☐ Very Shallow Dark Surface (TF12)
☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ______
Depth (inches): ______

Hydric Soil Present? Yes ✓ No ____

Remarks: Sandy floodplain alluvium soie.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

☐ Surface Water (A1)
☐ High Water Table (A2)
☐ Saturation (A3)
☐ Water Marks (B1)
☐ Sediment Deposits (B2)
☐ Drift Deposits (B3)
☐ Algal Mat or Crust (B4)
☐ Iron Deposits (B5)
☐ Inundation Visible on Aerial Imagery (B7)
☐ Water-Stained Leaves (B9)
☐ Salt Crust (B11)
☐ Aquatic Invertebrates (B13)
☐ Hydrogen Sulfide Odor (C1)
☐ Dry-Season Water Table (C2)
☐ Oxidized Rhizospheres on Living Roots (C3)
(**where not tilled**) ND
☐ Presence of Reduced Iron (C4)
☐ Thin Muck Surface (C7)
☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

☐ Surface Soil Cracks (B6)
☐ Sparsely Vegetated Concave Surface (B8)
☐ Drainage Patterns (B10)
☐ Oxidized Rhizospheres on Living Roots (C3)
(**where tilled**)
☐ Crayfish Burrows (C8)
☐ Saturation Visible on Aerial Imagery (C9)
☑ Geomorphic Position (D2)
☐ FAC-Neutral Test (D5) ND.
☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ____ No ✓ Depth (inches): ______
Water Table Present? Yes ____ No ✓ Depth (inches): ______
Saturation Present? Yes ____ No ✓ Depth (inches): ______
(includes capillary fringe)

Wetland Hydrology Present? Yes ____ No ✓

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Plot located in floodplain.
Could not see any oxidized rhizospheres.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC City/County: Boulder / " Sampling Date: 10/3/19
Applicant/Owner: See Report State: CO Sampling Point: SP-27
Investigator(s): KL / Carla Section, Township, Range: S9 T1S R70W
Landform (hillslope, terrace, etc.): Subtle Ridge Local relief (concave, convex, none): Convex Slope (%): 1
Subregion (LRR): G Lat: 39.982782 Long: -105.225396 Datum: NAD83
Soil Map Unit Name: Nh-Niwot NWI classification: PEM1C
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ___ (If no, explain in Remarks.)
Are Vegetation ___, Soil ___, or Hydrology ___ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ___
Are Vegetation ___, Soil ___, or Hydrology ___ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ___
Hydric Soil Present? Yes ✓ No ___
Wetland Hydrology Present? Yes ✓ No ___
Is the Sampled Area within a Wetland? Yes X No ___

Remarks: Ridge and swale microtopography present in meadow.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: 30')			
1. Elaeagnus angustifolia	20	Y	FACU
2.			
3.			
4.			
= Total Cover	20		
Sapling/Shrub Stratum (Plot size: 15')			
1.			
2.			
3.			
4.			
5.			
= Total Cover	0		
Herb Stratum (Plot size: 5')			
1. Dipsacus fullonum	10	N	FACU
2. Spartina pectinata	30	Y	FACW
3. Carex praegracilis	35	Y	FACW
4. Geum macroph.	5	N	FACW
5. Panicum virgatum	2	N	FAC
6. Agrostis gigantea	2	N	FACW
7.			
8.			
9.			
10.			
= Total Cover	84 42 16		
Woody Vine Stratum (Plot size: 15')			
1.			
2.			
= Total Cover			

% Bare Ground in Herb Stratum 10

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC–): 2 (A)
Total Number of Dominant Species Across All Strata: 3 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 66 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species		x 1 =
FACW species		x 2 =
FAC species		x 3 =
FACU species		x 4 =
UPL species		x 5 =
Column Totals:	(A)	(B)

Prevalence Index = B/A = ___

Hydrophytic Vegetation Indicators:

___ 1 - Rapid Test for Hydrophytic Vegetation
X 2 - Dominance Test is >50% 66
___ 3 - Prevalence Index is ≤3.0[1]
___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes X No ___

Remarks: Vegetation is starting to senesce. Community transitions to upland species approx 1 vert. foot above sample point.

SOIL

Sampling Point: SP-27

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth (inches)	Matrix		Redox Features				Texture	Remarks
	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-4	10YR2/2	95	10YR4/4	5	C	PL	Loam	
4-8	10YR3/1	95	10YR4/4	5	C	PL	Sandy Loam	
8-13	10YR3/3						Sandy Loam	

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR F**)
- ☐ 1 cm Muck (A9) (**LRR F, G, H**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
- ☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☒ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) (**LRR I, J**)
- ☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
- ☐ Dark Surface (S7) (**LRR G**)
- ☐ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Very Shallow Dark Surface (TF12)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):
Type: ____
Depth (inches): ____

Hydric Soil Present? Yes ☒ No ☐

Remarks: Distinct redox features present.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1)
- ☐ Sediment Deposits (B2)
- ☐ Drift Deposits (B3)
- ☐ Algal Mat or Crust (B4)
- ☐ Iron Deposits (B5)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Dry-Season Water Table (C2)
- ☒ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**)
- ☐ Presence of Reduced Iron (C4)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

- ☐ Surface Soil Cracks (B6)
- ☐ Sparsely Vegetated Concave Surface (B8)
- ☐ Drainage Patterns (B10)
- ☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Geomorphic Position (D2)
- ☒ FAC-Neutral Test (D5)
- ☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ☐ No ☒ Depth (inches): ____
Water Table Present? Yes ☐ No ☒ Depth (inches): ____
Saturation Present? (includes capillary fringe) Yes ☐ No ☒ Depth (inches): ____

Wetland Hydrology Present? Yes ☒ No ☐

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Wetland ridge is approx. 1-2 vert. Feet higher than adjacent cattail dominated wetland in swale.

WETLAND DETERMINATION DATA FORM – Great Plains Region

Project/Site: SBC. City/County: Boulder / " Sampling Date: 10/9/19.
Applicant/Owner: See Report State: CO Sampling Point: SP-28
Investigator(s): CD, KLH. Section, Township, Range: S9 T1S R70 W
Landform (hillslope, terrace, etc.): depression. Local relief (concave, convex, none): Concave. Slope (%): 0-1
Subregion (LRR): G Lat: 39.973219 Long: -105.229783 Datum: NAD 83
Soil Map Unit Name: Nh - Niwot NWI classification: PEM1C.
Are climatic / hydrologic conditions on the site typical for this time of year? Yes ✓ No ______ (If no, explain in Remarks.)
Are Vegetation ______, Soil ______, or Hydrology ______ significantly disturbed? N Are "Normal Circumstances" present? Yes ✓ No ______
Are Vegetation ______, Soil ______, or Hydrology ______ naturally problematic? N (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present? Yes ✓ No ______
Hydric Soil Present? (ASSUMED) Yes ✓ No ______
Wetland Hydrology Present? Yes ✓ No ______

Is the Sampled Area within a Wetland? Yes ✓ No ______

Remarks: Sparsely vegetated saline concave surface.

VEGETATION – Use scientific names of plants.

	Absolute % Cover	Dominant Species?	Indicator Status
Tree Stratum (Plot size: ______)			
1.			
2.			
3.			
4.			
= Total Cover			
Sapling/Shrub Stratum (Plot size: ______)			
1.			
2.			
3.			
4.			
5.			
= Total Cover			
Herb Stratum (Plot size: ______)			
1. Sporobolus airoides	10	Y	FAC
2. Puccinellia nuttalliana	5	Y	OBL
3. Spergularia marina	1	N	OBL
4. Rumex crispus	1	N	FAC
5. Juncus interior	1	N	FACW
6. Atriplex argentea	1	N	FAC
7.			
8.			
9.			
10.			
= Total Cover 9 2	19		
Woody Vine Stratum (Plot size: ______)			
1.			
2.			
= Total Cover			
% Bare Ground in Herb Stratum 80			

Remarks: Several halophytes.

Dominance Test worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC (excluding FAC−): 2 (A)
Total Number of Dominant Species Across All Strata: 2 (B)
Percent of Dominant Species That Are OBL, FACW, or FAC: 100 (A/B)

Prevalence Index worksheet:

Total % Cover of:		Multiply by:
OBL species		x 1 =
FACW species		x 2 =
FAC species		x 3 =
FACU species		x 4 =
UPL species		x 5 =
Column Totals:	(A)	(B)

Prevalence Index = B/A = ______

Hydrophytic Vegetation Indicators:

- ___ 1 - Rapid Test for Hydrophytic Vegetation
- ✓ 2 - Dominance Test is >50% 100
- ___ 3 - Prevalence Index is ≤3.0[1]
- ___ 4 - Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ___ Problematic Hydrophytic Vegetation[1] (Explain)

[1] Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? Yes ✓ No ______

US Army Corps of Engineers Great Plains – Version 2.0

SOIL

Sampling Point: SP-28

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features				Texture	Remarks
(inches)	Color (moist)	%	Color (moist)	%	Type[1]	Loc[2]		
0-5+	10YR3/2.	75.	10YR6/6.	25	C	M.	sil.cl.lm.	sparse veg. no roots.
								where pit.
								could not dig

[1]Type: C=Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

☐ Histosol (A1)
☐ Histic Epipedon (A2)
☐ Black Histic (A3)
☐ Hydrogen Sulfide (A4)
☐ Stratified Layers (A5) (**LRR F**)
☐ 1 cm Muck (A9) (**LRR F, G, H**)
☐ Depleted Below Dark Surface (A11)
☐ Thick Dark Surface (A12)
☐ Sandy Mucky Mineral (S1)
☐ 2.5 cm Mucky Peat or Peat (S2) (**LRR G, H**)
☐ 5 cm Mucky Peat or Peat (S3) (**LRR F**)
☐ Sandy Gleyed Matrix (S4)
☐ Sandy Redox (S5)
☐ Stripped Matrix (S6)
☐ Loamy Mucky Mineral (F1)
☐ Loamy Gleyed Matrix (F2)
☐ Depleted Matrix (F3)
☐ Redox Dark Surface (F6)
☐ Depleted Dark Surface (F7)
☑ Redox Depressions (F8)
☐ High Plains Depressions (F16)
  (**MLRA 72 & 73 of LRR H**)

Indicators for Problematic Hydric Soils[3]:

☐ 1 cm Muck (A9) (**LRR I, J**)
☐ Coast Prairie Redox (A16) (**LRR F, G, H**)
☐ Dark Surface (S7) (**LRR G**)
☐ High Plains Depressions (F16)
  (**LRR H outside of MLRA 72 & 73**)
☐ Reduced Vertic (F18)
☐ Red Parent Material (TF2)
☐ Very Shallow Dark Surface (TF12)
☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ______

Depth (inches): ______

Hydric Soil Present? Yes ✓ No ______

Remarks: Soils very rocky + hard.
Could only dig 4-6". Wetted w/ spray bottle.

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

☐ Surface Water (A1)
☐ High Water Table (A2)
☐ Saturation (A3)
☐ Water Marks (B1)
☐ Sediment Deposits (B2)
☐ Drift Deposits (B3)
☐ Algal Mat or Crust (B4)
☐ Iron Deposits (B5)
☐ Inundation Visible on Aerial Imagery (B7)
☐ Water-Stained Leaves (B9)
☑ Salt Crust (B11)
☐ Aquatic Invertebrates (B13)
☐ Hydrogen Sulfide Odor (C1)
☐ Dry-Season Water Table (C2)
☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where not tilled**)
☐ Presence of Reduced Iron (C4)
☐ Thin Muck Surface (C7)
☐ Other (Explain in Remarks)

Secondary Indicators (minimum of two required)

☑ Surface Soil Cracks (B6)
☑ Sparsely Vegetated Concave Surface (B8)
☐ Drainage Patterns (B10)
☐ Oxidized Rhizospheres on Living Roots (C3)
  (**where tilled**)
☐ Crayfish Burrows (C8)
☐ Saturation Visible on Aerial Imagery (C9)
☑ Geomorphic Position (D2)
☑ FAC-Neutral Test (D5)
☐ Frost-Heave Hummocks (D7) (**LRR F**)

Field Observations:

Surface Water Present? Yes ______ No ✓ Depth (inches): ______

Water Table Present? Yes ______ No ✓ Depth (inches): ______

Saturation Present? Yes ______ No ✓ Depth (inches): ______
(includes capillary fringe)

Wetland Hydrology Present? Yes ✓ No ______

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Sparsely vegetated concave surface.

APPENDIX H.2

APPROVED JURISDICTIONAL DETERMINATION

DEPARTMENT OF DEFENSE
UNITED STATES OF AMERICA

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, OMAHA DISTRICT
DENVER REGULATORY OFFICE, 9307 SOUTH WADSWORTH BOULEVARD
LITTLETON, COLORADO 80128-6901

May 20, 2022

SUBJECT: Approved Jurisdictional Determination
Corps File No. NWO-2021-01995-DEN, South Boulder Creek Detention Project, Boulder County, CO

Clint Henke
ERO Resources Corp.
1842 Clarkson St.
Denver, CO 80218

Dear Mr. Henke:

This letter is in reference to the proposed South Boulder Creek Detention Project, near U.S. 36 in the City of Boulder, at approximate latitude 39.977917° N, longitude -105.228438° W, in Boulder County, Colorado. We received a request for an Approved Jurisdictional Determination (JD) , submitted on behalf of the City of Boulder, for aquatic resources located within the above project area. The delineated areas located on the subject property have been reviewed in accordance with Section 404 of the Clean Water Act under which the U.S. Army Corps of Engineers regulates the discharge of dredged and fill material, and any excavation activity associated with a dredge and fill project in waters of the United States.

At your request, a JD has been prepared for the subject area. Based on a review of available documentation, we have determined that the aquatic resources labeled Ponds 1 - 4, wetlands W1 - W15, and other waters O7 - O10 on Figure 2a – 2d of the request are not waters of the United States. The attached JD form provides rationale for why these aquatic resources do not meet the definition of waters of the United States. Therefore, a Department of the Army permit is not required for the discharge of fill material into these aquatic resources under Section 404.

We have also determined that Viele Channel Ditch and Dry Creek Number 2 Ditch are jurisdictional. The attached JD form provides rationale for why these aquatic resources meet the definition of waters of the United States. If any work associated with this project requires the placement of dredged or fill material in Viele Channel Ditch and Dry Creek Number 2 Ditch, this office should be notified by a proponent of the project for Department of the Army permits

The JD is attached to this letter. If you are not in agreement with the JD decision, you may request an administrative appeal under regulation 33 CFR 331, by using the attached Appeal Form and Administrative Appeal Process form. The request for appeal must be received within 60 days from the date of this letter. It is not necessary to submit a Request for Appeal if you do not object to the JD.

This JD is valid for a period of five years from the date of this letter, unless new information warrants revisions of the JDs before the expiration date, or unless the Corps has identified, after a possible public notice and comment, that specific geographic areas with rapidly changing environmental conditions merit re-verification on a more frequent basis.

If there are any questions please feel free to contact Matt Montgomery at (720) 922-3852 or by e-mail at matthew.r.montgomery@usace.army.mil, and reference **Corps File No. NWO-2020-01995-DEN**.

Sincerely,

Kiel Downing
Chief, Denver Regulatory Office

Enclosures (4):

cc:
Brandon Coleman –Engineering Project Manager, City of Boulder
Eric Hahn – RJH Consultants, Inc.
Heidi Gerstung, ERO Resources Corp.

APPROVED JURISDICTIONAL DETERMINATION FORM
U.S. Army Corps of Engineers

This form should be completed by following the instructions provided in Section IV of the JD Form Instructional Guidebook.

SECTION I: BACKGROUND INFORMATION

A. REPORT COMPLETION DATE FOR APPROVED JURISDICTIONAL DETERMINATION (JD): May 20, 2022

B. DISTRICT OFFICE, FILE NAME, AND NUMBER:
Denver Regulatory Office
South Boulder Creek Detention Project
NWO-2021-01995-DEN

C. PROJECT LOCATION AND BACKGROUND INFORMATION:
State: CO County/parish/borough: Boulder City: Boulder
Center coordinates of site (lat/long in degree decimal format): Lat. 39.977917°N; Long. -105.228438°W
Name of nearest waterbody: South Boulder Creek
Name of nearest Traditional Navigable Water (TNW) into which the aquatic resource flows: St. Vrain River
Name of watershed or Hydrologic Unit Code (HUC):10190005
☒ Check if map/diagram of review area and/or potential jurisdictional areas is/are available upon request.
☐ Check if other sites (e.g., offsite mitigation sites, disposal sites, etc…) are associated with this action and are recorded on a different JD form.

D. REVIEW PERFORMED FOR SITE EVALUATION (CHECK ALL THAT APPLY):
☒ Office (Desk) Determination. Date: May 20, 2022
☒ Field Determination. Date(s): August 17, 2021

SECTION II: SUMMARY OF FINDINGS

A. RHA SECTION 10 DETERMINATION OF JURISDICTION.

There **Are no** "*navigable waters of the U.S.*" within Rivers and Harbors Act (RHA) jurisdiction (as defined by 33 CFR part 329) in the review area. [*Required*]
☐ Waters subject to the ebb and flow of the tide.
☐ Waters are presently used, or have been used in the past, or may be susceptible for use to transport interstate or foreign commerce. Explain: .

B. CWA SECTION 404 DETERMINATION OF JURISDICTION.

There **Are and are not** "*waters of the U.S.*" within Clean Water Act (CWA) jurisdiction (as defined by 33 CFR part 328) in the review area. [*Required*]

1. **Waters of the U.S.**
   a. **Indicate presence of waters of U.S. in review area (check all that apply):** [1]
      ☐ TNWs, including territorial seas
      ☐ Wetlands adjacent to TNWs
      ☒ Relatively permanent waters[2] (RPWs) that flow directly or indirectly into TNWs
      ☐ Non-RPWs that flow directly or indirectly into TNWs
      ☐ Wetlands directly abutting RPWs that flow directly or indirectly into TNWs
      ☐ Wetlands adjacent to but not directly abutting RPWs that flow directly or indirectly into TNWs
      ☐ Wetlands adjacent to non-RPWs that flow directly or indirectly into TNWs
      ☐ Impoundments of jurisdictional waters
      ☐ Isolated (interstate or intrastate) waters, including isolated wetlands

   b. **Identify (estimate) size of waters of the U.S. in the review area:**
      Non-wetland waters: acre: Viele Channel Ditch (0.01), Dry Creek Number 2 Ditch (0.06)
      Wetlands: acres.

   c. **Limits (boundaries) of jurisdiction** based on: OHWM
      Elevation of established OHWM (if known): .

2. **Non-regulated waters/wetlands (check if applicable):**[3]

---

[1] Boxes checked below shall be supported by completing the appropriate sections in Section III below.
[2] For purposes of this form, an RPW is defined as a tributary that is not a TNW and that typically flows year-round or has continuous flow at least "seasonally" (e.g., typically 3 months).
[3] Supporting documentation is presented in Section III.F.

☒ Potentially jurisdictional waters and/or wetlands were assessed within the review area and determined to be not jurisdictional. Explain: Ponds 1 - 4, wetlands W1 - W15, and other waters O7 - O10 were determined to be preamble waters and are not considered jurisdictional. See reference below in Section III.F.

SECTION III: CWA ANALYSIS

A. TNWs AND WETLANDS ADJACENT TO TNWs

The agencies will assert jurisdiction over TNWs and wetlands adjacent to TNWs. If the aquatic resource is a TNW, complete Section III.A.1 and Section III.D.1. only; if the aquatic resource is a wetland adjacent to a TNW, complete Sections III.A.1 and 2 and Section III.D.1.; otherwise, see Section III.B below.

1. TNW
Identify TNW: .

Summarize rationale supporting determination: .

2. Wetland adjacent to TNW
Summarize rationale supporting conclusion that wetland is "adjacent": .

B. CHARACTERISTICS OF TRIBUTARY (THAT IS NOT A TNW) AND ITS ADJACENT WETLANDS (IF ANY):

This section summarizes information regarding characteristics of the tributary and its adjacent wetlands, if any, and it helps determine whether or not the standards for jurisdiction established under *Rapanos* have been met.

The agencies will assert jurisdiction over non-navigable tributaries of TNWs where the tributaries are "relatively permanent waters" (RPWs), i.e. tributaries that typically flow year-round or have continuous flow at least seasonally (e.g., typically 3 months). A wetland that directly abuts an RPW is also jurisdictional. If the aquatic resource is not a TNW, but has year-round (perennial) flow, skip to Section III.D.2. If the aquatic resource is a wetland directly abutting a tributary with perennial flow, skip to Section III.D.4.

A wetland that is adjacent to but that does not directly abut an RPW requires a significant nexus evaluation. Corps districts and EPA regions will include in the record any available information that documents the existence of a significant nexus between a relatively permanent tributary that is not perennial (and its adjacent wetlands if any) and a traditional navigable water, even though a significant nexus finding is not required as a matter of law.

If the waterbody[4] is not an RPW, or a wetland directly abutting an RPW, a JD will require additional data to determine if the waterbody has a significant nexus with a TNW. If the tributary has adjacent wetlands, the significant nexus evaluation must consider the tributary in combination with all of its adjacent wetlands. This significant nexus evaluation that combines, for analytical purposes, the tributary and all of its adjacent wetlands is used whether the review area identified in the JD request is the tributary, or its adjacent wetlands, or both. If the JD covers a tributary with adjacent wetlands, complete Section III.B.1 for the tributary, Section III.B.2 for any onsite wetlands, and Section III.B.3 for all wetlands adjacent to that tributary, both onsite and offsite. The determination whether a significant nexus exists is determined in Section III.C below.

1. Characteristics of non-TNWs that flow directly or indirectly into TNW

(i) General Area Conditions:
Watershed size: **Pick List**
Drainage area: **Pick List**
Average annual rainfall: inches
Average annual snowfall: inchesF

(ii) Physical Characteristics:
(a) Relationship with TNW:
☐ Tributary flows directly into TNW.
☐ Tributary flows through **Pick List** tributaries before entering TNW.

Project waters are **Pick List** river miles from TNW.
Project waters are **Pick List** river miles from RPW.
Project waters are **Pick List** aerial (straight) miles from TNW.
Project waters are **Pick List** aerial (straight) miles from RPW.
Project waters cross or serve as state boundaries. Explain: .

---

[4] Note that the Instructional Guidebook contains additional information regarding swales, ditches, washes, and erosional features generally and in the arid West.

Identify flow route to TNW[5]: .
Tributary stream order, if known: .

(b) General Tributary Characteristics (check all that apply):

Tributary is:
☐ Natural
☐ Artificial (man-made). Explain: .
☐ Manipulated (man-altered). Explain: .

Tributary properties with respect to top of bank (estimate):
Average width: feet
Average depth: feet
Average side slopes: **Pick List.**

Primary tributary substrate composition (check all that apply):
☐ Silts
☐ Cobbles
☐ Bedrock
☐ Other. Explain: .
☐ Sands
☐ Gravel
☐ Vegetation. Type/% cover:
☐ Concrete
☐ Muck

Tributary condition/stability [e.g., highly eroding, sloughing banks]. Explain: .
Presence of run/riffle/pool complexes. Explain: .
Tributary geometry: **Pick List**
Tributary gradient (approximate average slope): %

(c) Flow:
Tributary provides for: **Pick List**
Estimate average number of flow events in review area/year: **Pick List**
Describe flow regime: .
Other information on duration and volume: .

Surface flow is: **Pick List.** Characteristics: .

Subsurface flow: **Pick List**. Explain findings: .
☐ Dye (or other) test performed: .

Tributary has (check all that apply):
☐ Bed and banks
☐ OHWM[6] (check all indicators that apply):
  ☐ clear, natural line impressed on the bank
  ☐ changes in the character of soil
  ☐ shelving
  ☐ vegetation matted down, bent, or absent
  ☐ leaf litter disturbed or washed away
  ☐ sediment deposition
  ☐ water staining
  ☐ other (list):
  ☐ the presence of litter and debris
  ☐ destruction of terrestrial vegetation
  ☐ the presence of wrack line
  ☐ sediment sorting
  ☐ scour
  ☐ multiple observed or predicted flow events
  ☐ abrupt change in plant community
☐ Discontinuous OHWM.[7] Explain: .

If factors other than the OHWM were used to determine lateral extent of CWA jurisdiction (check all that apply):

☐ High Tide Line indicated by:
  ☐ oil or scum line along shore objects
  ☐ fine shell or debris deposits (foreshore)
  ☐ physical markings/characteristics
  ☐ tidal gauges
  ☐ other (list):

☐ Mean High Water Mark indicated by:
  ☐ survey to available datum;
  ☐ physical markings;
  ☐ vegetation lines/changes in vegetation types.

(iii) Chemical Characteristics:

---

[5] Flow route can be described by identifying, e.g., tributary a, which flows through the review area, to flow into tributary b, which then flows into TNW.

[6] A natural or man-made discontinuity in the OHWM does not necessarily sever jurisdiction (e.g., where the stream temporarily flows underground, or where the OHWM has been removed by development or agricultural practices). Where there is a break in the OHWM that is unrelated to the waterbody's flow regime (e.g., flow over a rock outcrop or through a culvert), the agencies will look for indicators of flow above and below the break.

[7] Ibid.

Characterize tributary (e.g., water color is clear, discolored, oily film; water quality; general watershed characteristics, etc.). Explain: .
Identify specific pollutants, if known: .

(iv) Biological Characteristics. Channel supports (check all that apply):
☐ Riparian corridor. Characteristics (type, average width): .
☐ Wetland fringe. Characteristics: .
☐ Habitat for:
  ☐ Federally Listed species. Explain findings: .
  ☐ Fish/spawn areas. Explain findings: .
  ☐ Other environmentally-sensitive species. Explain findings: .
  ☐ Aquatic/wildlife diversity. Explain findings: .

2. Characteristics of wetlands adjacent to non-TNW that flow directly or indirectly into TNW

(i) Physical Characteristics:
(a) General Wetland Characteristics:
Properties:
Wetland size: acres
Wetland type. Explain: .
Wetland quality. Explain: .
Project wetlands cross or serve as state boundaries. Explain: .

(b) General Flow Relationship with Non-TNW:
Flow is: **Pick List**. Explain: .

Surface flow is**:** **Pick List**
Characteristics: .

Subsurface flow: **Pick List**. Explain findings: .
☐ Dye (or other) test performed: .

(c) Wetland Adjacency Determination with Non-TNW:
☐ Directly abutting
☐ Not directly abutting
  ☐ Discrete wetland hydrologic connection. Explain: .
  ☐ Ecological connection. Explain: .
  ☐ Separated by berm/barrier. Explain: .

(d) Proximity (Relationship) to TNW
Project wetlands are **Pick List** river miles from TNW.
Project waters are **Pick List** aerial (straight) miles from TNW.
Flow is from: **Pick List.**
Estimate approximate location of wetland as within the **Pick List** floodplain.

(ii) Chemical Characteristics:
Characterize wetland system (e.g., water color is clear, brown, oil film on surface; water quality; general watershed characteristics; etc.). Explain: .
Identify specific pollutants, if known: .

(iii) Biological Characteristics. Wetland supports (check all that apply):
☐ Riparian buffer. Characteristics (type, average width): .
☐ Vegetation type/percent cover. Explain: .
☐ Habitat for:
  ☐ Federally Listed species. Explain findings: .
  ☐ Fish/spawn areas. Explain findings: .
  ☐ Other environmentally-sensitive species. Explain findings: .
  ☐ Aquatic/wildlife diversity. Explain findings: .

3. Characteristics of all wetlands adjacent to the tributary (if any)
All wetland(s) being considered in the cumulative analysis: **Pick List**
Approximately ( ) acres in total are being considered in the cumulative analysis.

For each wetland, specify the following:

Directly abuts? (Y/N)	Size (in acres)	Directly abuts? (Y/N)	Size (in acres)

Summarize overall biological, chemical and physical functions being performed: .

C. SIGNIFICANT NEXUS DETERMINATION

A significant nexus analysis will assess the flow characteristics and functions of the tributary itself and the functions performed by any wetlands adjacent to the tributary to determine if they significantly affect the chemical, physical, and biological integrity of a TNW. For each of the following situations, a significant nexus exists if the tributary, in combination with all of its adjacent wetlands, has more than a speculative or insubstantial effect on the chemical, physical and/or biological integrity of a TNW. Considerations when evaluating significant nexus include, but are not limited to the volume, duration, and frequency of the flow of water in the tributary and its proximity to a TNW, and the functions performed by the tributary and all its adjacent wetlands. It is not appropriate to determine significant nexus based solely on any specific threshold of distance (e.g. between a tributary and its adjacent wetland or between a tributary and the TNW). Similarly, the fact an adjacent wetland lies within or outside of a floodplain is not solely determinative of significant nexus.

Draw connections between the features documented and the effects on the TNW, as identified in the *Rapanos* Guidance and discussed in the Instructional Guidebook. Factors to consider include, for example:

- Does the tributary, in combination with its adjacent wetlands (if any), have the capacity to carry pollutants or flood waters to TNWs, or to reduce the amount of pollutants or flood waters reaching a TNW?
- Does the tributary, in combination with its adjacent wetlands (if any), provide habitat and lifecycle support functions for fish and other species, such as feeding, nesting, spawning, or rearing young for species that are present in the TNW?
- Does the tributary, in combination with its adjacent wetlands (if any), have the capacity to transfer nutrients and organic carbon that support downstream foodwebs?
- Does the tributary, in combination with its adjacent wetlands (if any), have other relationships to the physical, chemical, or biological integrity of the TNW?

Note: the above list of considerations is not inclusive and other functions observed or known to occur should be documented below:

1. **Significant nexus findings for non-RPW that has no adjacent wetlands and flows directly or indirectly into TNWs.** Explain findings of presence or absence of significant nexus below, based on the tributary itself, then go to Section III.D: .

2. **Significant nexus findings for non-RPW and its adjacent wetlands, where the non-RPW flows directly or indirectly into TNWs.** Explain findings of presence or absence of significant nexus below, based on the tributary in combination with all of its adjacent wetlands, then go to Section III.D: .

3. **Significant nexus findings for wetlands adjacent to an RPW but that do not directly abut the RPW.** Explain findings of presence or absence of significant nexus below, based on the tributary in combination with all of its adjacent wetlands, then go to Section III.D: .

D. DETERMINATIONS OF JURISDICTIONAL FINDINGS. THE SUBJECT WATERS/WETLANDS ARE (CHECK ALL THAT APPLY):

1. **TNWs and Adjacent Wetlands.** Check all that apply and provide size estimates in review area:
   ☐ TNWs: linear feet width (ft), Or, acres.
   ☐ Wetlands adjacent to TNWs: acres.

2. **RPWs that flow directly or indirectly into TNWs.**
   ☐ Tributaries of TNWs where tributaries typically flow year-round are jurisdictional. Provide data and rationale indicating that tributary is perennial:
   ☒ Tributaries of TNW where tributaries have continuous flow "seasonally" (e.g., typically three months each year) are jurisdictional. Data supporting this conclusion is provided at Section III.B. Provide rationale indicating that tributary flows seasonally: Viele Channel Ditch lacks gage data, but consultants for the project reported the ditch generally flows the majority of the year.

   Dry Creek Number 2 Ditch generally flows between April and October, delivering irrigation flows, and has a decreed water right of 44 cfs at the headgate.

   Provide estimates for jurisdictional waters in the review area (check all that apply):
   ☒ Tributary waters: approx. 0.07 acre
   ☐ Other non-wetland waters: acres.
   Identify type(s) of waters: .

3. **Non-RPWs[8] that flow directly or indirectly into TNWs.**
   ☐ Waterbody that is not a TNW or an RPW, but flows directly or indirectly into a TNW, and it has a significant nexus with a TNW is jurisdictional. Data supporting this conclusion is provided at Section III.C.

   Provide estimates for jurisdictional waters within the review area (check all that apply):
   ☐ Tributary waters: linear feet width (ft).
   ☐ Other non-wetland waters: acres.
   Identify type(s) of waters: **.**

4. **Wetlands directly abutting an RPW that flow directly or indirectly into TNWs.**
   ☐ Wetlands directly abut RPW and thus are jurisdictional as adjacent wetlands.
   ☐ Wetlands directly abutting an RPW where tributaries typically flow year-round. Provide data and rationale indicating that tributary is perennial in Section III.D.2, above. Provide rationale indicating that wetland is directly abutting an RPW:

   ☐ Wetlands directly abutting an RPW where tributaries typically flow "seasonally." Provide data indicating that tributary is seasonal in Section III.B and rationale in Section III.D.2, above. Provide rationale indicating that wetland is directly abutting an RPW:

   Provide acreage estimates for jurisdictional wetlands in the review area: acres.

5. **Wetlands adjacent to but not directly abutting an RPW that flow directly or indirectly into TNWs.**
   ☐ Wetlands that do not directly abut an RPW, but when considered in combination with the tributary to which they are adjacent and with similarly situated adjacent wetlands, have a significant nexus with a TNW are jurisidictional. Data supporting this conclusion is provided at Section III.C.

   Provide acreage estimates for jurisdictional wetlands in the review area: acres.

6. **Wetlands adjacent to non-RPWs that flow directly or indirectly into TNWs.**
   ☐ Wetlands adjacent to such waters, and have when considered in combination with the tributary to which they are adjacent and with similarly situated adjacent wetlands, have a significant nexus with a TNW are jurisdictional. Data supporting this conclusion is provided at Section III.C.

   Provide estimates for jurisdictional wetlands in the review area: acres.

7. **Impoundments of jurisdictional waters.[9]**
   As a general rule, the impoundment of a jurisdictional tributary remains jurisdictional.
   ☐ Demonstrate that impoundment was created from "waters of the U.S.," or
   ☐ Demonstrate that water meets the criteria for one of the categories presented above (1-6), or
   ☐ Demonstrate that water is isolated with a nexus to commerce (see E below).

E. ISOLATED [INTERSTATE OR INTRA-STATE] WATERS, INCLUDING ISOLATED WETLANDS, THE USE, DEGRADATION OR DESTRUCTION OF WHICH COULD AFFECT INTERSTATE COMMERCE, INCLUDING ANY SUCH WATERS (CHECK ALL THAT APPLY):[10]

☐ which are or could be used by interstate or foreign travelers for recreational or other purposes.
☐ from which fish or shellfish are or could be taken and sold in interstate or foreign commerce.
☐ which are or could be used for industrial purposes by industries in interstate commerce.
☐ Interstate isolated waters. Explain: .
☐ Other factors. Explain: .

Identify water body and summarize rationale supporting determination: .

Provide estimates for jurisdictional waters in the review area (check all that apply):
☐ Tributary waters: linear feet width (ft).
☐ Other non-wetland waters: acres.
Identify type(s) of waters: **.**
☐ Wetlands: acres.

F. NON-JURISDICTIONAL WATERS, INCLUDING WETLANDS (CHECK ALL THAT APPLY):

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[8] See Footnote # 3.

[9] To complete the analysis refer to the key in Section III.D.6 of the Instructional Guidebook.

[10] **Prior to asserting or declining CWA jurisdiction based solely on this category, Corps Districts will elevate the action to Corps and EPA HQ for review consistent with the process described in the Corps/EPA *Memorandum Regarding CWA Act Jurisdiction Following Rapanos.***

☐ If potential wetlands were assessed within the review area, these areas did not meet the criteria in the 1987 Corps of Engineers Wetland Delineation Manual and/or appropriate Regional Supplements.

☐ Review area included isolated waters with no substantial nexus to interstate (or foreign) commerce.

  ☐ Prior to the Jan 2001 Supreme Court decision in "*SWANCC*," the review area would have been regulated based <u>solely</u> on the "Migratory Bird Rule" (MBR).

☐ Waters do not meet the "Significant Nexus" standard, where such a finding is required for jurisdiction. Explain: .

☒ Other: (explain, if not covered above): Reference is made to the November 13, 1986 Federal Register (Page 41217), Part 328 (a) Non-tidal drainage and irrigation ditches excavated on dry land, (b) Artificially irrigated areas which would revert to upland if the irrigation ceased, (c) Artificial lakes or ponds created by excavating and/or diking dry land to collect and retain water and which are used exclusively for such purposes as stock watering, irrigations, settling basins, or rice growing, and (e) Waterfilled depressions created in dry land incidental to construction activity and pits excavated in dry land for the purpose of obtaining fill, sand, or gravel unless and until the construction or excavation operation is abandoned and the resulting body of water meets the definition of waters of the United States (as defined in 33 CFR 328.3(a)). The Corps of Engineers generally does not consider these types of aquatic resources waters of the U.S. except on a case-by-case basis and.

In this case, Viele Channel Ditch wetlands W1 (0.5 acre) and Dry Creek Number 2 Ditch wetlands W2 (3.8 acres) are associated with non-tidal irrigation ditches constructed on dry land and would revert tup uplands if irrigation ceased.

Pond 1 (0.05 acre) and associated wetland W3 (0.45 acre), Pond 2 (0.45 acre) and associated wetland W4 (0.42 acre), Pond 3 (2.54 acre) and associated wetland W5 (0.1.34 acre), Pond 4 (0.25 acre) and associated wetland W6 (0.07 acre) are within the footprint of the historic mining operation and appear to be borrow pits for construction of US 36. These ponds now receive stormwater runoff and do not contribute RPW flow to a waters of the US. These bodies of water and associated wetlands do not meet the definition of a waters of the US.

Wetlands W7-W15 are located depressional areas within the footprint of the historic mining operation, associated with irrigation laterals, or located along roadside ditches.

Other waters (O7 through O10) are irrigation laterals and associated open water features.

As such, these aquatic resources are preamble waters, do not meet the definition of waters of the US, and are not considered jurisdictional.

Provide acreage estimates for non-jurisdictional waters in the review area, where the <u>sole</u> potential basis of jurisdiction is the MBR factors (i.e., presence of migratory birds, presence of endangered species, use of water for irrigated agriculture), using best professional judgment (check all that apply):

☐ Non-wetland waters (i.e., rivers, streams): linear feet width (ft).
☐ Lakes/ponds: acres.
☐ Other non-wetland waters: acres. List type of aquatic resource: .
☐ Wetlands: acres.

Provide acreage estimates for non-jurisdictional waters in the review area that do not meet the "Significant Nexus" standard, where such a finding is required for jurisdiction (check all that apply):

☐ Non-wetland waters (i.e., rivers, streams): linear feet, width (ft).
☐ Lakes/ponds: acres.
☐ Other non-wetland waters: acres. List type of aquatic resource: .
☐ Wetlands: acres.

<u>SECTION IV: DATA SOURCES</u>.

A. SUPPORTING DATA. Data reviewed for JD (check all that apply - checked items shall be included in case file and, where checked and requested, appropriately reference sources below):

☒ Maps, plans, plots or plat submitted by or on behalf of the applicant/consultant: Request for Approved Jurisdictional Determination for South Boulder Creek Detention Project - Viele Channel Ditch, Dry Creek Ditch Number 2, Pond 1, Pond 2, Pond 3, Pond 4, Wetlands, and Other Waters, Boulder County, Colorado, ERO Resources Corp., dated February 18, 2022.

☒ Data sheets prepared/submitted by or on behalf of the applicant/consultant.
  ☒ Office concurs with data sheets/delineation report.
  ☐ Office does not concur with data sheets/delineation report.
☐ Data sheets prepared by the Corps: .
☐ Corps navigable waters' study: .
☒ U.S. Geological Survey Hydrologic Atlas: .
  ☒ USGS NHD data.
  ☐ USGS 8 and 12 digit HUC maps.
☐ U.S. Geological Survey map(s). Cite scale & quad name: .
☐ USDA Natural Resources Conservation Service Soil Survey. Citation: .

☐ National wetlands inventory map(s). Cite name: .
☐ State/Local wetland inventory map(s): .
☐ FEMA/FIRM maps: .
☐ 100-year Floodplain Elevation is: (National Geodectic Vertical Datum of 1929)
☒ Photographs: ☒ Aerial (Name & Date):USGS Historic Aerial dated September 9, 1978.
or ☐ Other (Name & Date): .
☐ Previous determination(s). File no. and date of response letter: .
☐ Applicable/supporting case law: .
☐ Applicable/supporting scientific literature: .
☐ Other information (please specify): .

B. ADDITIONAL COMMENTS TO SUPPORT JD:

USGS Historic Aerial, photo date: September 9, 1978

NOTIFICATION OF ADMINISTRATIVE APPEAL OPTIONS AND PROCESS AND REQUEST FOR APPEAL

Applicant: Brandon Coleman, City of Boulder	File Number: NWO-2021-01995-DEN	Date: May 20, 2022

Attached is:		See Section below
	INITIAL PROFFERED PERMIT (Standard Permit or Letter of permission)	A
	PROFFERED PERMIT (Standard Permit or Letter of permission)	B
	PERMIT DENIAL	C
X	APPROVED JURISDICTIONAL DETERMINATION	D
	PRELIMINARY JURISDICTIONAL DETERMINATION	E

SECTION I - The following identifies your rights and options regarding an administrative appeal of the above decision. Additional information may be found in Corps regulations at 33 CFR Part 331, or at http://www.usace.army.mil/Missions/CivilWorks/RegulatoryProgramandPermits/FederalRegulation.aspx

A: INITIAL PROFFERED PERMIT: You may accept or object to the permit.

- ACCEPT: If you received a Standard Permit, you may sign the permit document and return it to the district engineer for final authorization. If you received a Letter of Permission (LOP), you may accept the LOP and your work is authorized. Your signature on the Standard Permit or acceptance of the LOP means that you accept the permit in its entirety, and waive all rights to appeal the permit, including its terms and conditions, and approved jurisdictional determinations associated with the permit.
- OBJECT: If you object to the permit (Standard or LOP) because of certain terms and conditions therein, you may request that the permit be modified accordingly. You must complete Section II of this form and return the form to the district engineer. Your objections must be received by the district engineer within 60 days of the date of this notice, or you will forfeit your right to appeal the permit in the future. Upon receipt of your letter, the district engineer will evaluate your objections and may: (a) modify the permit to address all of your concerns, (b) modify the permit to address some of your objections, or (c) not modify the permit having determined that the permit should be issued as previously written. After evaluating your objections, the district engineer will send you a proffered permit for your reconsideration, as indicated in Section B below.

B: PROFFERED PERMIT: You may accept or appeal the permit

- ACCEPT: If you received a Standard Permit, you may sign the permit document and return it to the district engineer for final authorization. If you received a Letter of Permission (LOP), you may accept the LOP and your work is authorized. Your signature on the Standard Permit or acceptance of the LOP means that you accept the permit in its entirety, and waive all rights to appeal the permit, including its terms and conditions, and approved jurisdictional determinations associated with the permit.
- APPEAL: If you choose to decline the proffered permit (Standard or LOP) because of certain terms and conditions therein, you may appeal the declined permit under the Corps of Engineers Administrative Appeal Process by completing Section II of this form and sending the form to the division engineer. This form must be received by the division engineer within 60 days of the date of this notice.

C: PERMIT DENIAL: You may appeal the denial of a permit under the Corps of Engineers Administrative Appeal Process by completing Section II of this form and sending the form to the division engineer. This form must be received by the division engineer within 60 days of the date of this notice.

D: APPROVED JURISDICTIONAL DETERMINATION: You may accept or appeal the approved JD or provide new information.

- ACCEPT: You do not need to notify the Corps to accept an approved JD. Failure to notify the Corps within 60 days of the date of this notice, means that you accept the approved JD in its entirety, and waive all rights to appeal the approved JD.
- APPEAL: If you disagree with the approved JD, you may appeal the approved JD under the Corps of Engineers Administrative Appeal Process by completing Section II of this form and sending the form to the division engineer. This form must be received by the division engineer within 60 days of the date of this notice.

E: PRELIMINARY JURISDICTIONAL DETERMINATION: You do not need to respond to the Corps regarding the preliminary JD. The Preliminary JD is not appealable. If you wish, you may request an approved JD (which may be appealed), by contacting the Corps district for further instruction. Also, you may provide new information for further consideration by the Corps to reevaluate the JD.

SECTION II - REQUEST FOR APPEAL or OBJECTIONS TO AN INITIAL PROFFERED PERMIT

REASONS FOR APPEAL OR OBJECTIONS: (Describe your reasons for appealing the decision or your objections to an initial proffered permit in clear concise statements. You may attach additional information to this form to clarify where your reasons or objections are addressed in the administrative record.)

ADDITIONAL INFORMATION: The appeal is limited to a review of the administrative record, the Corps memorandum for the record of the appeal conference or meeting, and any supplemental information that the review officer has determined is needed to clarify the administrative record. Neither the appellant nor the Corps may add new information or analyses to the record. However, you may provide additional information to clarify the location of information that is already in the administrative record.

POINT OF CONTACT FOR QUESTIONS OR INFORMATION:

If you have questions regarding this decision and/or the appeal process you may contact: US Army Corps of Engineers, Denver Regulatory Office Attn: Matt Montgomery 9307 S. Wadsworth Blvd Littleton, CO 80128 Telephone (720) 922-3852 Matthew.R.Montgomery@usace.army.mil	If you only have questions regarding the appeal process you may also contact: US Army Corps of Engineers, Northwestern Division Attn: Melinda Larsen, Regulatory Appeals Review Officer 1201 NE Lloyd Blvd Ste 400 Portland, OR 97232-1257 Telephone (503) 808-3888 Melinda.M.Larsen@usace.army.mil

RIGHT OF ENTRY: Your signature below grants the right of entry to Corps of Engineers personnel, and any government consultants, to conduct investigations of the project site during the course of the appeal process. You will be provided a 15-day notice of any site investigation and will have the opportunity to participate in all site investigations.

______________________________ Signature of appellant or agent.	Date:	Telephone number:

APPENDIX I

COST OPINION INFORMATION

Bid Schedule
South Boulder Creek Regional Detention Concept Design

April 2022

Item No.	Item Description	Unit	Estimated Quantity	Unit Price	Extension
General Items					
1	Mobilization, Demobilization & Preparatory Work	1	Lump Sum	$ 1,620,000.00	$ 1,620,000.00
2	Diversion and Control of Surface Water	1	Lump Sum	$ 35,000.00	$ 35,000.00
3	Dewatering	1	Lump Sum	$ 1,100,000.00	$ 1,100,000.00
4	Stripping and Stockpiling Topsoil	70,000	Cubic Yard	$ 4.00	$ 280,000.00
5	Clearing and Grubbing	1	Lump Sum	$ 150,000.00	$ 150,000.00
6	Demolition and Disposal	1	Lump Sum	$ 300,000.00	$ 300,000.00
7	Sediment and Erosion Control	1	Lump Sum	$ 260,000.00	$ 260,000.00
8	Temporary Site Security & Fencing	1	Lump Sum	$ 100,000.00	$ 100,000.00
9	Permanent Fencing	1	Lump Sum	$ 25,000.00	$ 25,000.00
10	Reclamation	100	Acre	$ 3,800.00	$ 380,000.00
11	Ecological Restoration	25	Acre	$ 140,000.00	$ 3,500,000.00
12	Outlet Works Discharge Structure Access Improvements	1	Lump Sum	$ 1,000,000.00	$ 1,000,000.00
13	Spillway Access Road	1	Lump Sum	$ 65,000.00	$ 65,000.00
14	All Other Items of Work (2.5%)	1	Lump Sum	$ 791,000.00	$ 791,000.00
General Earthwork					
15	Embankment Excavation	64,500	Cubic Yard	$ 6.00	$ 387,000.00
16	Excavating and Stockpiling of CU Levee	85,000	Cubic Yard	$ 6.00	$ 510,000.00
17	Excavating Detention Basin and Shaping Slopes	120,000	Cubic Yard	$ 7.00	$ 840,000.00
18	Placing Detention Excavation Fill	40,000	Cubic Yard	$ 7.00	$ 280,000.00
19	Furnishing and Placing CU Earthfill - Import	70,000	Cubic Yard	$ 25.00	$ 1,750,000.00
20	Furnishing and Placing CU Earthfill - From Excavations	80,000	Cubic Yard	$ 7.00	$ 560,000.00
21	Aggregate Surfacing for CU Earthfill	4,050	Cubic Yard	$ 55.00	$ 222,750.00
22	Asphalt Paving for CU Earthfill	10,000	Square Yard	$ 18.00	$ 180,000.00
Embankment Dam					
23	Foundation Preparation	41,000	Square Yards	$ -	$ -
24	Furnishing and Placing Embankment Core	13,500	Cubic Yard	$ 15.00	$ 202,500.00
25	Furnishing and Placing Embankment Fill	135,000	Cubic Yard	$ 7.00	$ 945,000.00
26	Furnishing and Placing Filter Sand	9,500	Cubic Yard	$ 85.00	$ 807,500.00
27	Furnishing and Placing Drain Gravel	1,100	Cubic Yard	$ 90.00	$ 99,000.00
28	Furnishing and Installing 8-inch PVC Drain Pipe	2,405	Linear Feet	$ 100.00	$ 240,500.00
29	Furnishing and Placing Aggregate Surfacing	960	Cubic Yard	$ 55.00	$ 52,800.00
30	Furnishing and Placing Upstream Erosion Control Blanket	17,000	Square Yards	$ 4.50	$ 76,500.00
Spillway					
31	Excavation to Working Platform	28,000	Cubic Yard	$ 5.00	$ 140,000.00
32	Secant Pile Wall	14,000	Vertical Linear Feet	$ 500.00	$ 7,000,000.00
33	Pile Cap	1,200	Cubic Yard	$ 500.00	$ 600,000.00
34	Spillway Wall Reinforced Concrete	1,520	Cubic Yard	$ 1,000.00	$ 1,520,000.00
35	Backfill Above Working Platform	26,100	Cubic Yard	$ 7.00	$ 182,700.00
36	Spillway Apron Reinforced Concrete	1,050	Cubic Yard	$ 450.00	$ 472,500.00
37	Soil Bentonite/Secant Wall Tie-in	2	Each	$ 50,000.00	$ 100,000.00
38	Groundwater Drain Excavation	2,400	Cubic Yard	$ 9.50	$ 22,800.00
39	Groundwater Drain Filter Gravel	2,350	Cubic Yard	$ 110.00	$ 258,500.00
40	Groundwater Drain Backfill Plug	16	Each	$ 1,500.00	$ 24,000.00
41	Groundwater Drain 10-inch PVC Slotted Pipe	3,600	Linear Feet	$ 105.00	$ 378,000.00
42	Groundwater Drain 10-inch PVC Solid Pipe	700	Linear Feet	$ 100.00	$ 70,000.00
43	Groundwater Drain Connector Pipe	8	Each	$ 2,600.00	$ 20,800.00
44	Groundwater Drain 6-foot-diameter Manhole	22	Each	$ 12,000.00	$ 264,000.00
45	Groundwater Drain Flow Regulation Gate	16	Each	$ 10,000.00	$ 160,000.00
Instrumentation					
46	Piezometers	10	Each	$ 7,000.00	$ 70,000.00
47	Structure Monitoring Points	9	Each	$ 2,000.00	$ 18,000.00
48	Weir Boxes	5	Each	$ 18,000.00	$ 90,000.00
Barrier Wall					
49	Mobilization	1	Lump Sum	$ 200,000.00	$ 200,000.00
50	Mix Design	1	Lump Sum	$ 20,000.00	$ 20,000.00
51	Barrier Wall Working Platform	40,000	Square Foot	$ 1.00	$ 40,000.00
52	Soil-Bentonite Wall - Embankment Station 28+02 to 57+00	45,000	Vertical Square Foot	$ 11.00	$ 495,000.00
53	Soil-Bentonite Wall - Detention Pond	59,000	Vertical Square Foot	$ 11.00	$ 649,000.00
Outlet Works					
54	Tunneling Mobilization and Demobilization	1	Lump Sum	$ 120,000.00	$ 120,000.00
55	Furnish and Install Casing Pipe and Conduit in Excavated Tunnel	235	Linear Foot	$ 4,500.00	$ 1,057,500.00
56	Tunneling Launch Shaft	1	Lump Sum	$ 180,000.00	$ 180,000.00
57	Tunneling Recovery Shaft	1	Lump Sum	$ 100,000.00	$ 100,000.00
58	Furnish and Install Reinforced Concrete Encased Conduit	465	Linear Foot	$ 4,500.00	$ 2,092,500.00
59	Intake Structure Reinforced Concrete	1	Lump Sum	$ 80,000.00	$ 80,000.00
60	Outlet Structure Reinforced Concrete	1	Lump Sum	$ 175,000.00	$ 175,000.00
61	Furnishing and Installing Trashracks	1	Lump Sum	$ 124,000.00	$ 124,000.00
62	Furnishing and Placing Riprap Bedding	11	Cubic Yard	$ 75.00	$ 825.00
63	Furnishing and Placing Riprap	32	Cubic Yard	$ 160.00	$ 5,120.00
Site Drainage					
64	Furnishing and Installing Spillway Drainage Penetrations	5	Each	$ 65,000.00	$ 325,000.00
65	Dry Creek Reinforced Concrete Outlet Basin	1	Lump Sum	$ 50,000.00	$ 50,000.00
US 36 Multi-Use Trail					
66	Temporary Signage and Traffic Control	1	Lump Sum	$ 115,000.00	$ 115,000.00
67	Trail Concrete Paving	3,700	Square Foot	$ 15.00	$ 55,500.00
68	Permanent Trail Signing and Markings	1	Lump Sum	$ 10,000.00	$ 10,000.00
Bonds and Insurance					$510,679
Base Construction Subtotal (BCS)					**$34,555,974**
Contingencies (25% of BCS)					$8,511,324
Direct Construction Subtotal (DCS)					**$43,067,298**

Design Engineering (9% of BCS)	$3,110,038
CLOMR/LOMR (1% of BCS)	$345,560
Construction Engineering and Management (12% of BCS)	$4,146,717
Permitting (1% of BCS)	$345,560
Opinion of Probable Project Cost (OPPC)	**$51,015,172**

Notes
1. Costs are in April 2022 Dollars (ENR CCI factor of 12898.96)
2. Costs for real estate and easements are not included.

APPENDIX J

30 PERCENT DESIGN DRAWINGS

Provided separately