SALT LAKE COUNTY

NORTHWEST CANAL AND CREEK STUDY

(HAL Project No.: 014.18.100)

FINAL REPORT

April 2022

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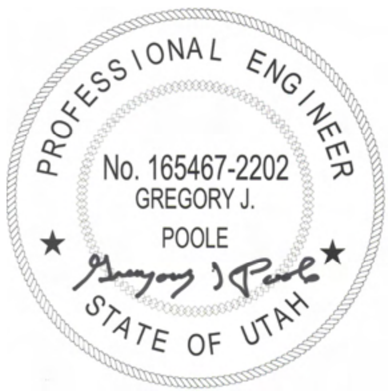

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TABLE OF CONTENTS

LIST OF TABLES

NO. TITLE PAGE

LIST OF FIGURES

NO. TITLE PAGE

GLOSSARY

10-year storm - The storm event that has a 10% (1 in 10) chance of being equaled or exceeded in any given year.

100-year storm - The storm event that has a 1% (1 in 100) chance of being equaled or exceeded in any given year.

Cross drainage structures - Cross drainage structures convey storm drainage flows from one side of the street to the other and normally consist of storm drains or culverts.

Design Rainstorm - A rainfall event, defined by storm frequency and storm duration, that is used to design drainage structures or conveyance systems.

Detention Basin - An impoundment structure designed to reduce peak runoff flow rates by retaining a portion of the runoff during periods of peak flow and then releasing the runoff at lower flow rates.

HEC-HMS - A Hydrologic Modeling System developed by the U.S. Army Corps of Engineers.

Initial storm drainage system - The drainage system which provides for conveyance of the storm runoff from minor storm events. The initial drainage system usually consists of curb and gutter, storm drains, and local detention facilities. The initial drainage system should be designed to reduce street maintenance, control nuisance flooding, help create an orderly urban system, and provide convenience to urban residents.

Major storm drainage system - The drainage system that provides protection from flooding of homes during a major storm event. The major storm drainage system may include streets (including overtopping the curb onto the park strip and sidewalk), large conduits, open channels, and regional detention facilities.

Major storm event - Generally accepted as the 100-year storm. Homes should typically be protected from flooding in storm events up to a 100-year event.

Minor storm event - Storm event which is less than or equal to a 10-year storm.

Probable Maximum Flood - A flood event with a very low probability, usually less than 0.2%, of being exceeded in any given year. This flood event is used as a design storm when failure of the structure could cause loss of life.

Retention Basin - An impoundment structure designed to contain all of the runoff from a design storm event. Retention basins usually contain the runoff until it evaporates or infiltrates into the ground.

Storm Duration - The length of time that defines the rainfall depth or intensity for a given frequency.

Storm Frequency - A measure of the relative risk that the precipitation depth for a particular design storm will be equaled or exceeded in any given year. This risk is usually expressed in years. For example, a storm with a 100-year frequency will have a 1% chance of being equaled or exceeded in a given year.

ABBREVIATIONS

cfs	cubic feet per second (ft^3/s)
E	East
ft	foot or feet
GIS	Geographic Information System
HAL	Hansen, Allen & Luce, Inc.
ID #	identification number
in	inches
N	North
NOAA	National Oceanic and Atmospheric Administration
NRCS	National Resource Conservation Service
RR	railroad
S	South
SCS	Soil Conservation Service
SD	storm drain/drainage
tot	total
TR-55	Technical Release-55
W	West
WSE	water surface elevation

EXECUTIVE SUMMARY

The study area is shown on Figure EX-1 NWCCS Drainage Area Map and includes the following County drainage facilities.

- C-7 Ditch
- Lee Drain & Lee Creek
- Kersey Creek
- Utah & Salt Lake Canal
- Riter Canal
- Coon Canyon Creek and Harker's Canyon Creek
- Goggin Drain
- Kearns-Chesterfield Drain

The Northwest Canal and Creek Study (NWCCS) serves four purposes:

1. Provides storm water runoff models which predict how the NWCCS drainages respond to design storm runoff events.
2. Identifies and describes existing system problems.
3. Identifies alternative mitigation measures to eliminate flooding during design storm runoff events.
4. Provides recommendations for management of the County facilities and provides documentation of the preferred drainage improvements.

Following is a summary of the key study findings and recommendations by drainage facility.

C-7 DITCH

The C-7 Ditch is a major Salt Lake County flood control facility. The following canals and creeks discharge into the C-7 Ditch.

- Utah & Salt Lake Canal
- Coon Canyon Creek and Harker's Canyon Creek
- Riter Canal
- Kersey Creek
- Lee Drain & Lee Creek

The C-7 is formed at the confluence of the Utah & Salt Lake Canal Extension and the Riter Canal and flows northerly into the Great Salt Lake. The banks of the C-7 Ditch are heavily vegetated with phragmites. Phragmites is difficult to control (phragmites Q&A fact sheet.pmd (fws.gov)).

- Based on a comparison of 2020 survey data and 1995 plans "Relocated C-7 Ditch, Tailings Modernization Project" (Morrison Knudsen Corporation for Kennecott Utah Copper), sedimentation has occurred in about the lower 9,000 feet of the C-7 ditch upstream of I-80 with sediment depth of about four feet in the I-80 culverts.

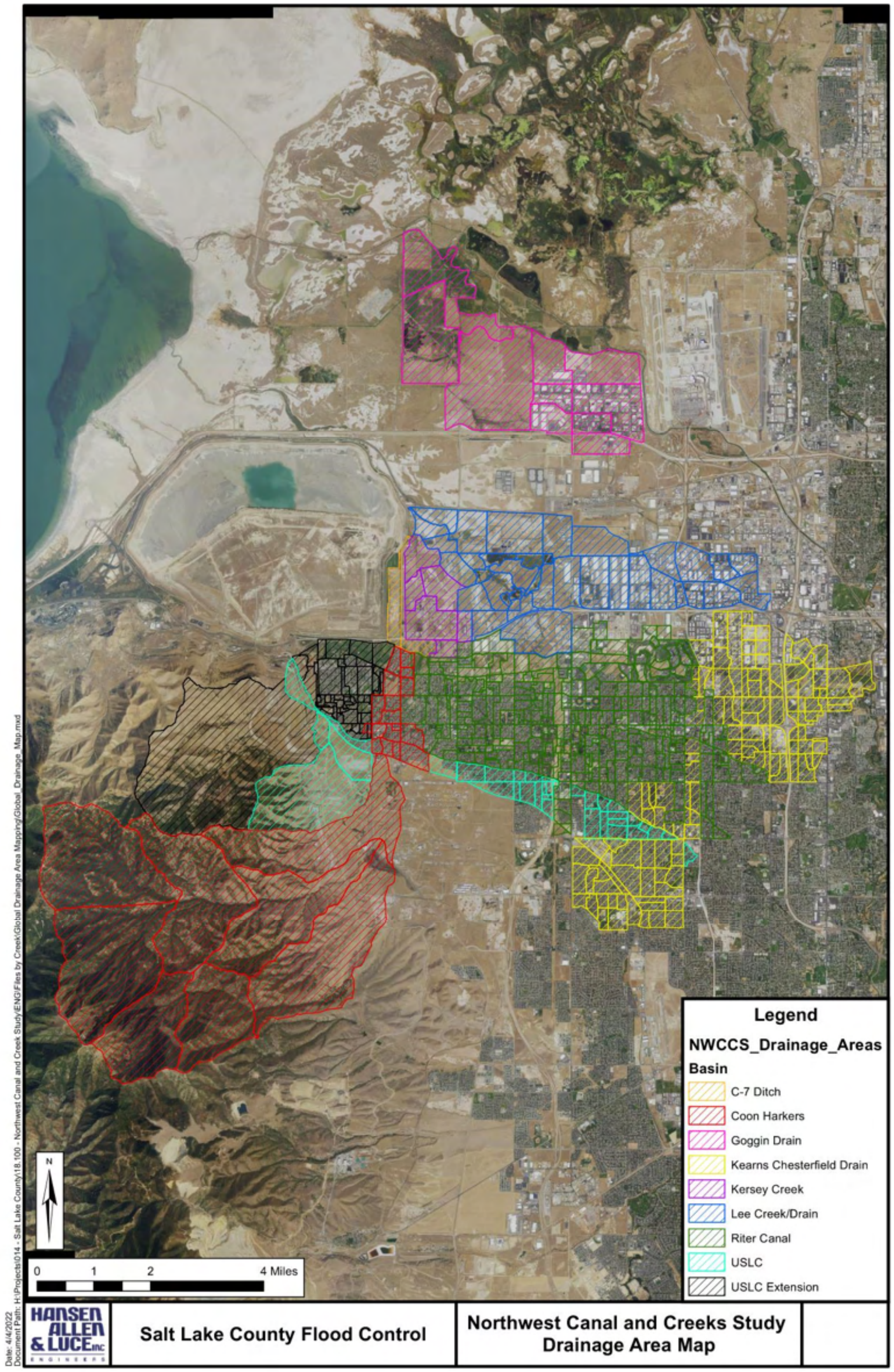


Figure EX-1. Northwest Canal and Creek Study Drainage Area Map

- It is unknown whether the lower C-7 is continuing to aggrade due to sedimentation or if the channel has now reached equilibrium between sediment carried through the C-7 versus the incoming sediment load. It is recommended that a sedimentation monitoring plan be developed for the C-7.

- Areas predicted to be flooded in a 100-year storm runoff event are shown on Figure EX-2. The flooding affected areas are not currently developed. It is recommended that if/when areas shown flooded in a 100-year event develop; the areas be filled to provide a minimum of 1-foot freeboard above the predicted water surface elevations.

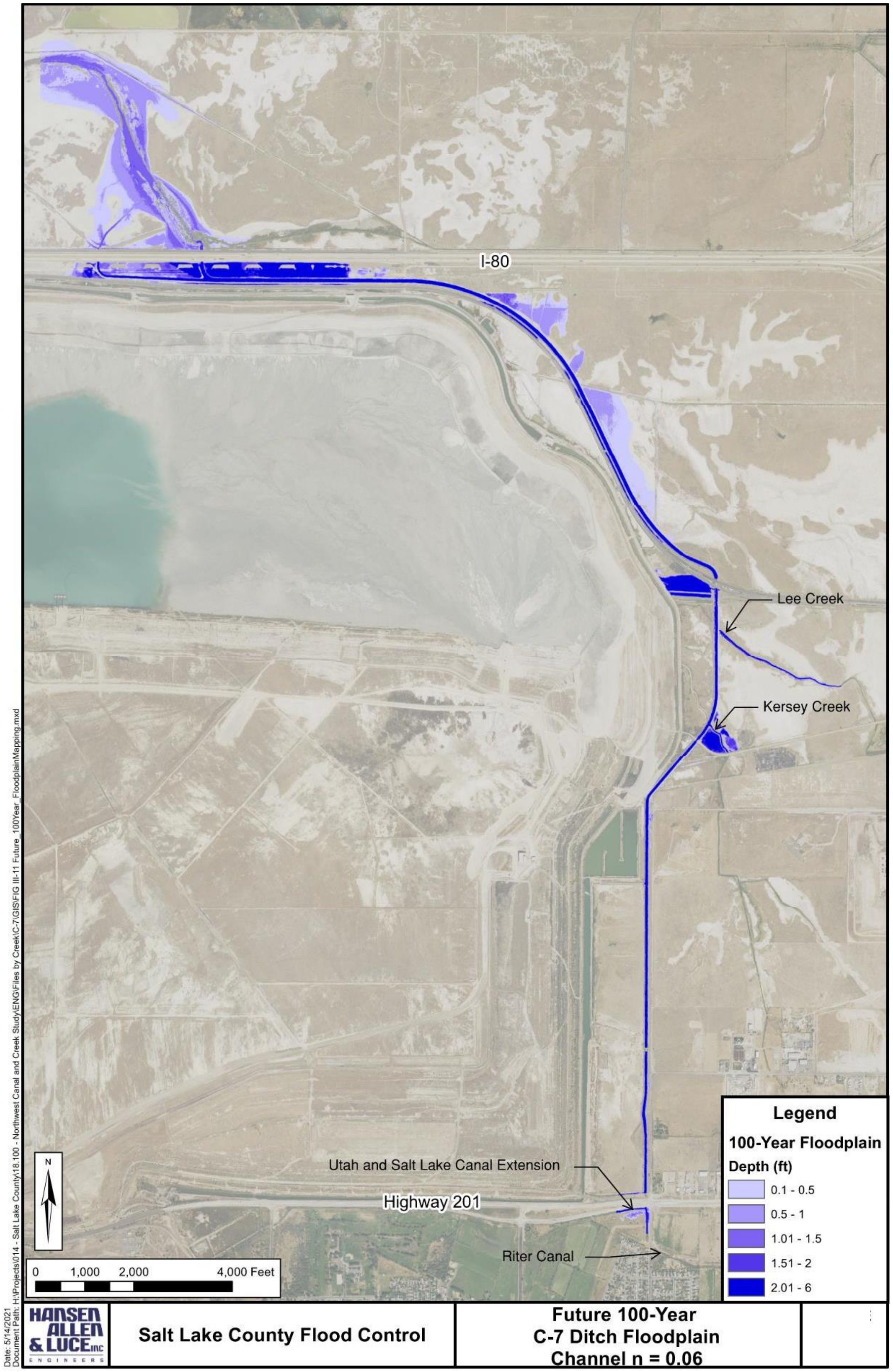


Figure EX-2. C-7 Ditch 100-Year Floodplain

LEE DRAIN & LEE CREEK

Lee Drain and Lee Creek are shown on Figure EX-3. The Salt Lake County portion of the Lee Drain conveyance begins just downstream of a Salt Lake City pump station located at 2955 Andrew Ave. Lee Drain is tributary to Lee Kay Ponds (about 6000 West 1300 South) located south of the Salt Lake County Solid Waste Facility. Near the outlet from the Lee Kay Ponds, Lee Drain flows into Lee Creek. Lee Creek begins at the north side of the Riter Canal (about 2550 South 6000 West) and flows into the C-7 Ditch at about 7700 West 1000 South.

- Most of the area tributary to Lee Drain upstream of the Lee Kay Ponds has already developed with Salt Lake City detention requirements of 0.2 cfs per acre in a 100-year event.
- Flooding is predicted with the existing channel/culvert system during a 100-year event. Table EX-1 summarizes the current Lee Drain culvert capacities, 100-year design flowrates, and the additional needed capacity.

TABLE EX-1. LEE DRAIN EXISTING CROSSING CAPACITY DEFICIENCIES

LOCATION	STA	EXISTING CAPACITY (CFS)	DESIGN FLOWRATE (100-YR) CFS	ADDITIONAL NEEDED CAPACITY (%)
Gladiola Street	18,600	45	73	62%
Brighton & North Point Canal	16,879	45	73	62%
Access Road	16,700	45	94	109%
Rail Road	13,200	62	190	206%
Bangerter HWY	12,900	66	190	188%
Gramercy Road	12,200	100	232	132%
Rail Road	8,400	64	266	316%
4800 West	7,000	105	310	195%
5070 West	5,300	127	351	176%
5500 West	2,500	200	460	130%
5600 West	1,800	215	460	114%

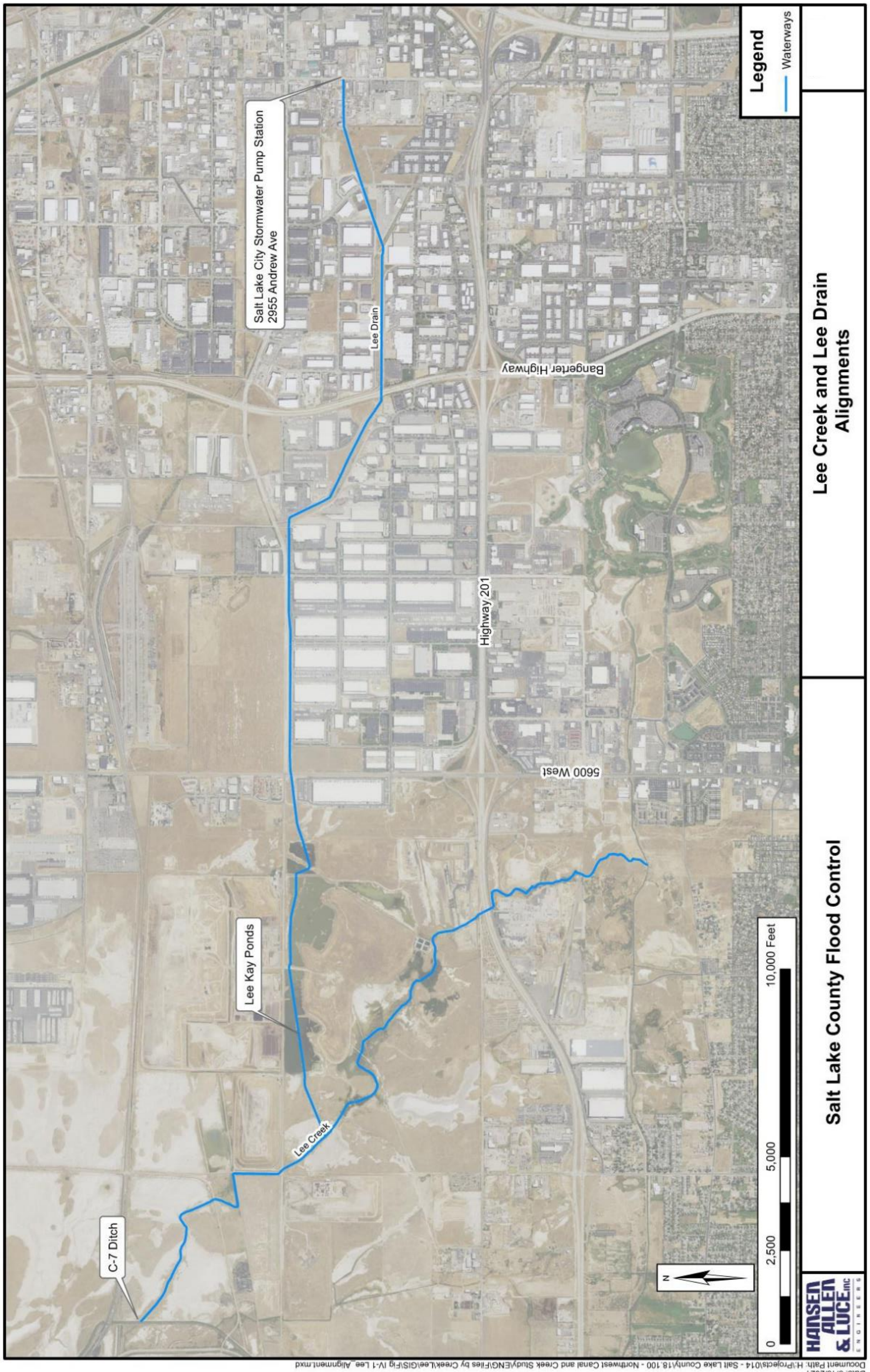


Figure EX- 3. Lee Creek and Lee Drain

The preferred solution for Lee Drain includes culvert and channel improvements.
Lee Drain design peak storm runoff flow rates and conceptual culvert sizes are summarized in Table EX-2.

TABLE EX-2. LEE DRAIN CONCEPTUAL CROSSING SIZES UPSTREAM OF LEE KAY PONDS

LOCATION	STA	DESIGN FLOWRATE (100-YR) CFS	Conceptual Culvert Size Span x Height (feet)
Gladiola Street	18,600	73	5 x 4
Brighton & North Point Canal	16,879	73	5 x 4
Access Road	16,700	94	5 x 4
Rail Road	13,200	190	8 x 5
Bangerter HWY	12,900	190	8 x 5
Gramercy Road	12,200	232	10 x 5
Rail Road	8,400	266	12 x 5
4800 West	7,000	310	12 x 5
5070 West	5,300	351	12 x 6
5500 West	2,500	460	16 x 6
5600 West	1,800	460	16 x 6

A typical proposed channel cross section is shown on Figure EX-4. Estimated excavation quantities are summarized in Table EX-3.

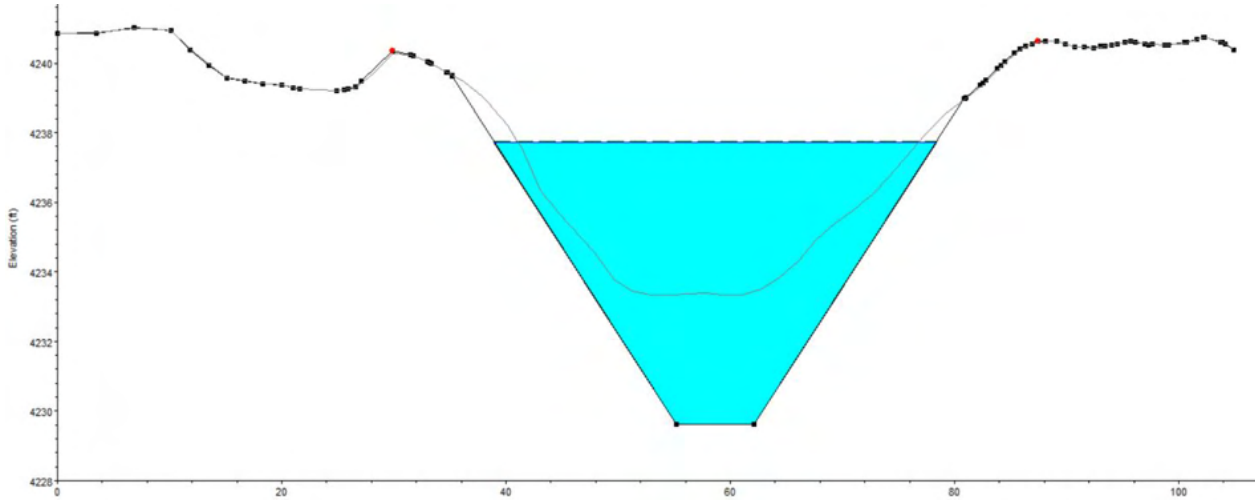


Figure EX- 4. Typical Upper Lee Drain Master Plan Channel Cross Section

TABLE EX-3. LEE DRAIN PROPOSED CHANNEL BOTTOM WIDTHS AND ESTIMATED EXCAVATION QUANTITIES (Trapezoidal Channel with 2 Horizontal to 1 Vertical Side Slopes)

Up STA	Down STA	Bottom Width (ft)	CUT VOLUME (CY)
19,450	18,682	5	280
18,682	16,700	5	3,410
16,700	12,650	7	13,320
12,650	9,962	8	4,890
9,962	7,853	9	5,060
7,853	6,979	10	1,900
6,979	0	0	5,160
		TOTAL	34,020

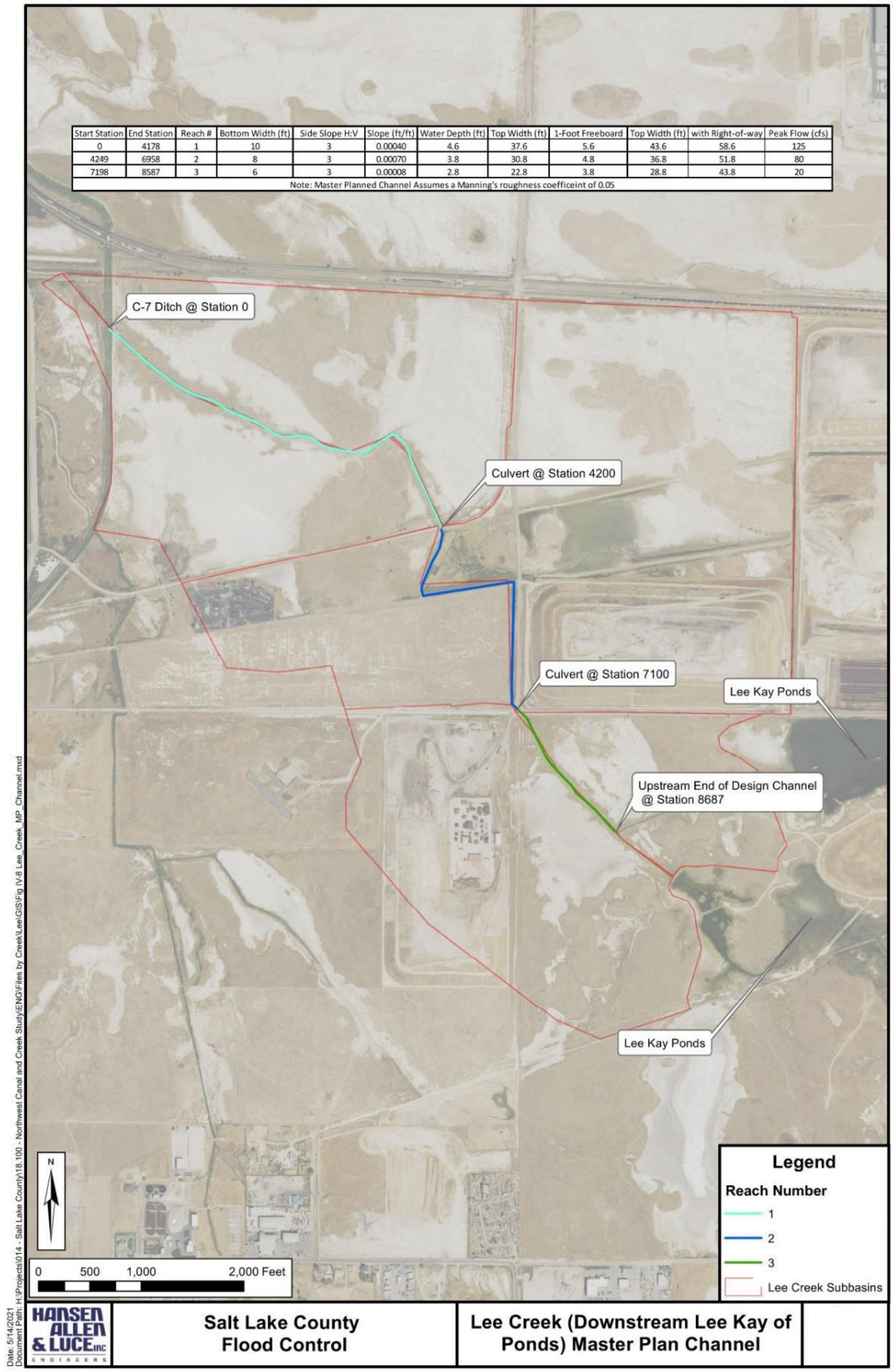
The preferred solution for Lee Creek downstream of the Lee Kay Ponds includes channel and culvert improvements as well as release restrictions from the ponds. Figure EX-5 shows the alignment of Lee Creek and channel configurations by segment. Design peak storm runoff flowrates and conceptual road crossing sizes for Lee Creek downstream of the Lee Kay Ponds are summarized on Table EX-4.

TABLE EX-4. LEE CREEK CONCEPTUAL ROAD CROSSING SIZES (Down Stream of LEE KAY PONDS).

CROSSING	STA (feet)	Crossing Length (ft)	100-YR Design Flows (cfs)	Conceptual Culvert Size Span x Height (feet)
1300 S	7050	180	20	4 x 3
8000 W	4200	40	80	6 x 4

The preferred solution includes a 30 cfs capacity overflow spillway from Riter Canal to Lee Creek.

A critical assumption for Lee Creek upstream of Highway 201 was the availability of storage to attenuate peak flows. Model results indicate that in order to attenuate projected flows to the design flow of the existing culvert (36 cfs) approximately 19.1 acre-ft of storage is required. The property that includes the existing natural detention has a mixture of State and private ownership. We recommend that easements or sufficient property be acquired to maintain at least the minimum required storage for the system to function as designed.



Start Station	End Station	Reach #	Bottom Width (ft)	Side Slope H:V	Slope (ft/ft)	Water Depth (ft)	Top Width (ft)	1-Foot Freeboard	Top Width (ft)	with Right-of-way	Peak Flow (cfs)
0	4178	1	10	3	0.00040	4.6	37.6	5.6	43.6	58.6	125
4249	6958	2	8	3	0.00070	3.8	30.8	4.8	36.8	51.8	80
7198	8587	3	6	3	0.00008	2.8	22.8	3.8	28.8	43.8	20
Note: Master Planned Channel Assumes a Manning's roughness coefficeint of 0.05											

Figure EX- 5. Lee Creek (Downstream of Lee Kay Ponds) Master Plan

KERSEY CREEK

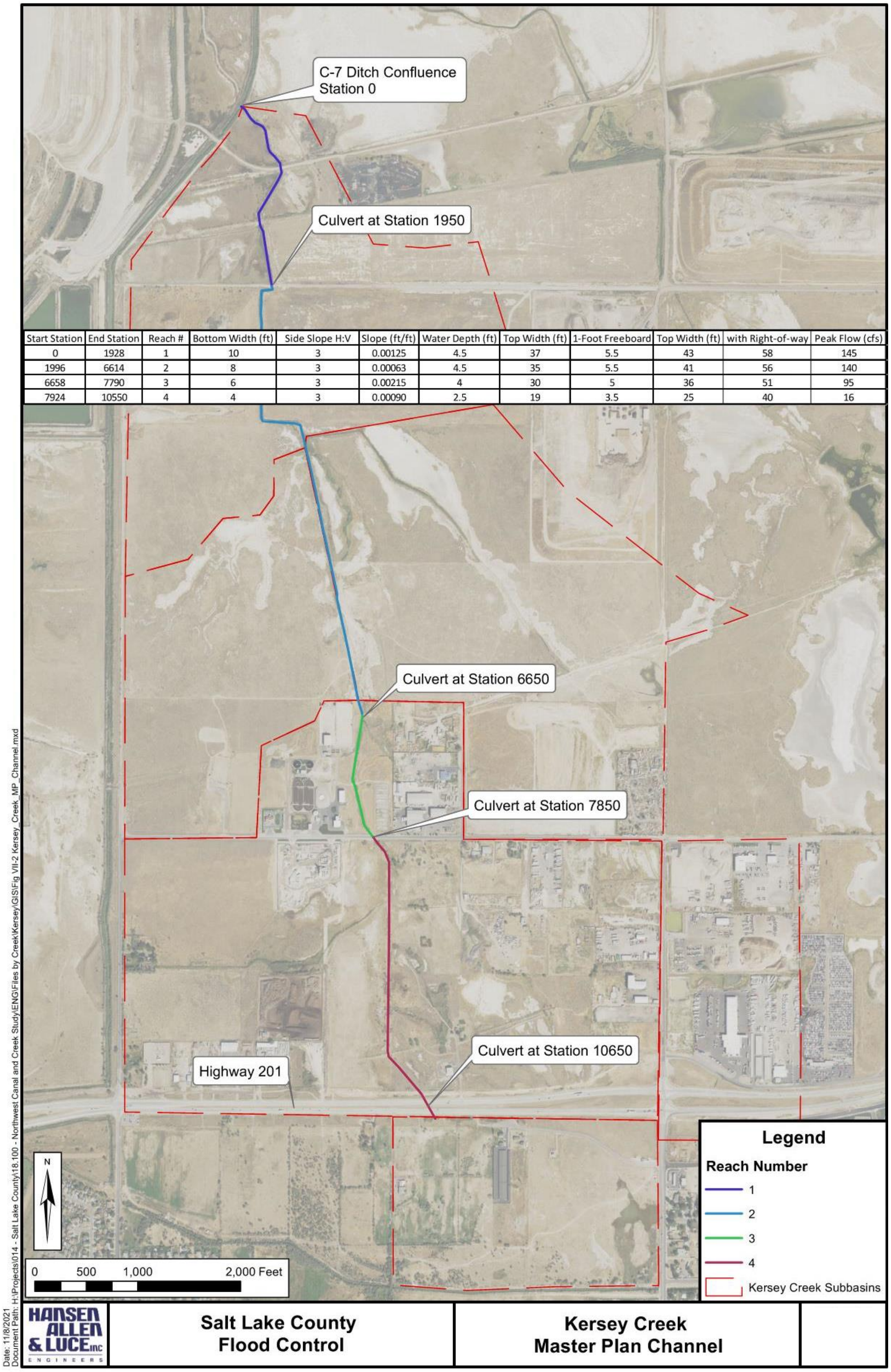
The existing Kersey Creek conveyance is poorly defined. Kersey Creek begins on the south side of Highway 201 at a culvert that goes under the Highway. It flows generally to the north through a few other crossings until it discharges into the C-7 Ditch.

The master plan solution for Kersey Creek includes channel and culvert improvements to be constructed as adjacent properties develop. Figure EX-6 shows the proposed alignment and identifies segments of the reach that will have the same channel configuration. Kersey Creek passes through low lying ground which will need to be filled prior to development to provide a minimum of 1-foot of freeboard above the 100-year water surface elevation.

Proposed road crossings design flowrates and conceptual opening sizes are provided in Table EX-5.

TABLE EX-5. KERSEY CREEK CONCEPTUAL ROAD CROSSING SIZES

CROSSING	STA (feet)	Crossing Length (ft)	Design Flow Rate (cfs)	Conceptual Culvert Size (feet)
Highway 201	10,600	200	16	3’ Dia. (Circular)
Unnamed	10,040	40	16	3’ Dia. (Circular)
2100 S	7,800	110	19	3’ Dia. (Circular)
Unnamed	7,310	20	95	6 x 4
Unnamed	6,630	20	95	6 x 4
1300 S	1,950	40	145	8 x 4
Unnamed	1,340	30	145	8 x 4
8000 W	740	60	145	8 x 4



Start Station	End Station	Reach #	Bottom Width (ft)	Side Slope H:V	Slope (ft/ft)	Water Depth (ft)	Top Width (ft)	1-Foot Freeboard	Top Width (ft)	with Right-of-way	Peak Flow (cfs)
0	1928	1	10	3	0.00125	4.5	37	5.5	43	58	145
1996	6614	2	8	3	0.00063	4.5	35	5.5	41	56	140
6658	7790	3	6	3	0.00215	4	30	5	36	51	95
7924	10550	4	4	3	0.00090	2.5	19	3.5	25	40	16

Figure EX- 6. Kersey Creek Master Plan

UTAH AND SALT LAKE CANAL

The portion of the USLC included in the Northwest Canal and Creek Study (NWCC Study) is shown on Figure EX-7. The USLC study area extends from Bangerter Highway (at about 4900 South) through the USLC Extension to the C-7 Ditch. Even though the USLC Extension is not listed as a Salt Lake County flood control facility, it is included in the study area and is tributary to the C-7 Ditch.

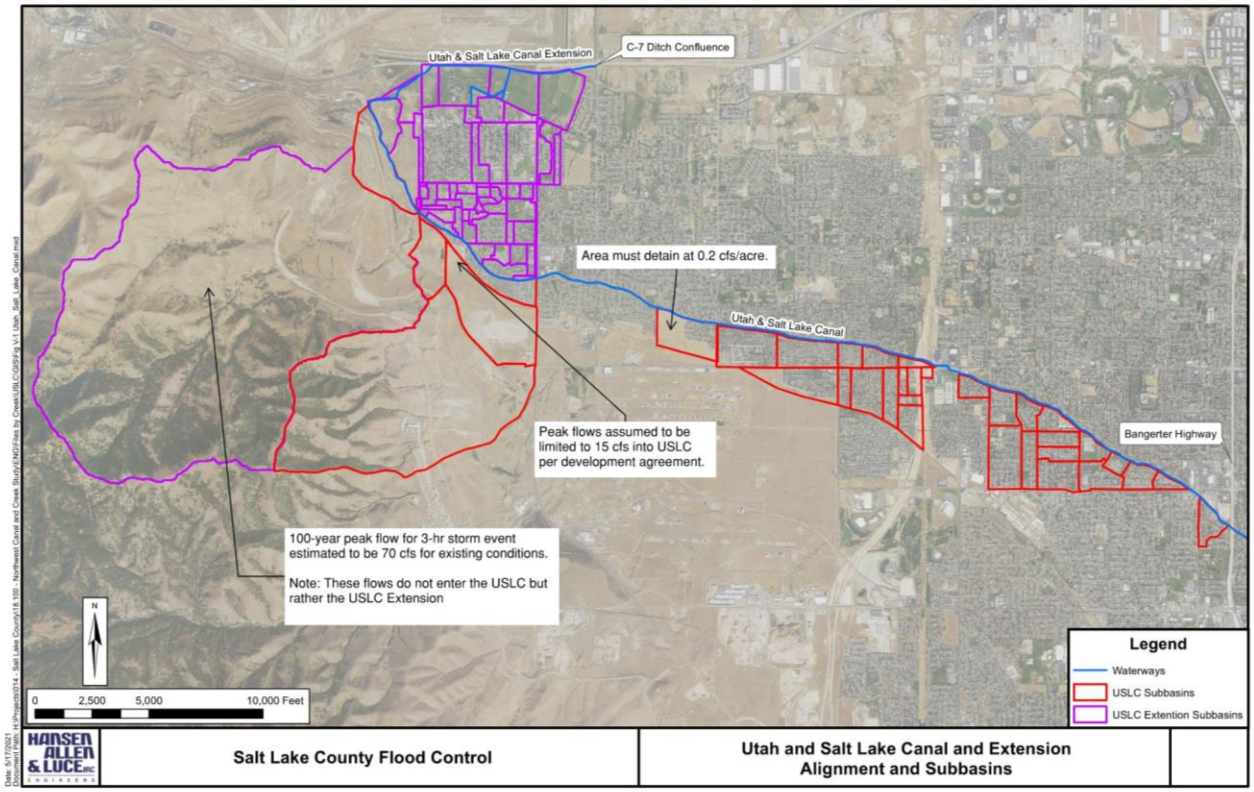


Figure EX- 7. Utah and Salt Lake Canal Study Area

The Utah and Salt Lake Canal was found to have sufficient capacity for the 100-year storm runoff event including a 30 cfs irrigation base flow. The Utah and Salt Lake Canal Extension was found to have the following deficiencies.

- Existing 36-inch culvert at 9180 West shows flow going over the top of the road for the 100-year event.
- Minor flooding along the golf course is predicted for both the 100-year event and 10-year events.

COON CANYON CREEK AND HARKER'S CANYON CREEK

A master plan for the Coon Canyon Creek and Harker's Canyon Creek was completed in 2008 and master planned conveyance improvements were constructed in 2016 up to 3100 South. The master planned parallel 36-inch diameter storm drain from 3100 South to the C-7 Ditch has not been completed. The Coon Canyon Creek and Harker's Canyon Creek drainage area is shown on Figure EX-8. The 4100 South Coon Harker Detention Basin provides about 90 acre-feet of storage and is key to flood control and should be maintained on a regular basis.

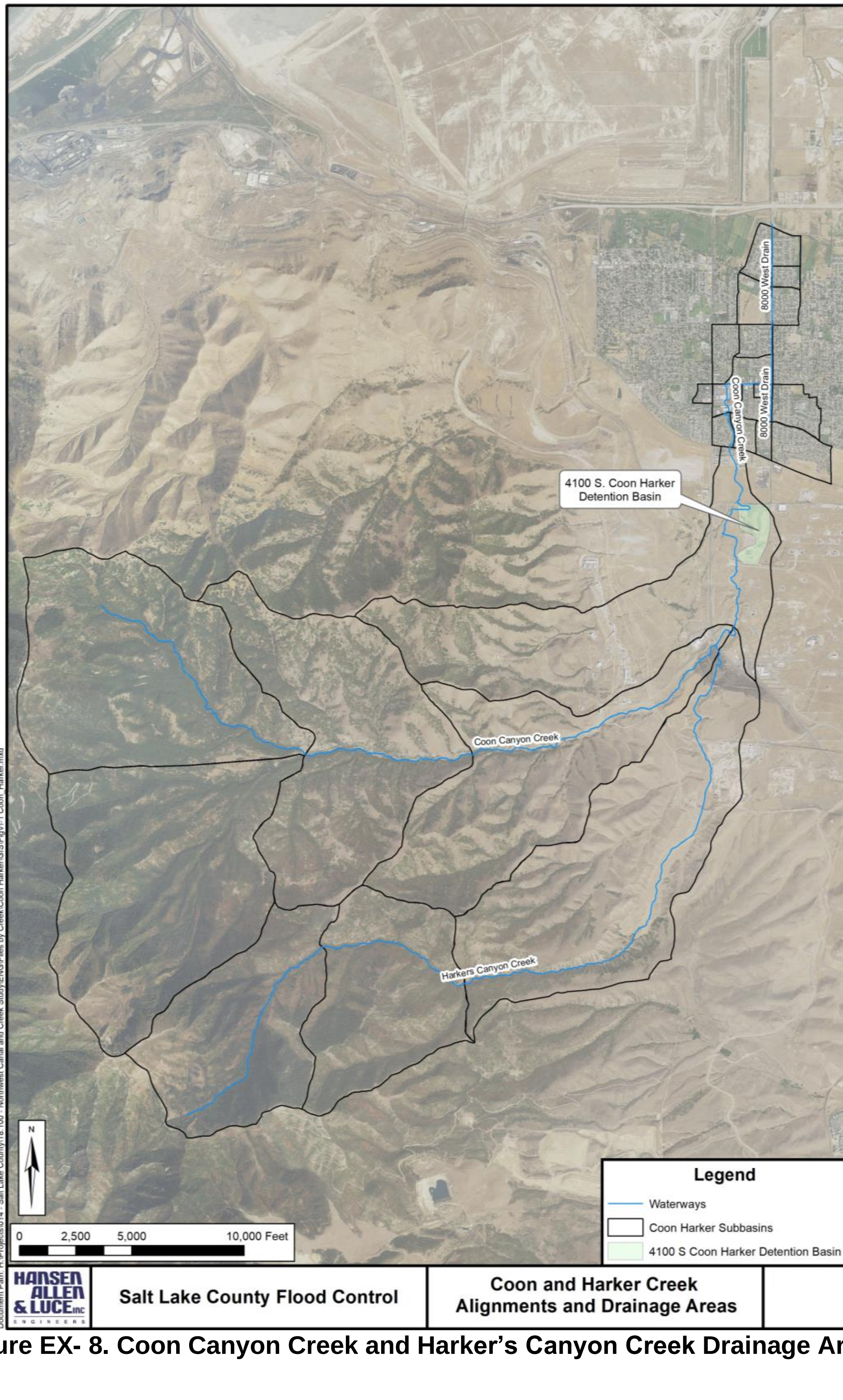


Figure EX- 8. Coon Canyon Creek and Harker's Canyon Creek Drainage Areas

RITER CANAL

A significant portion of the storm drainage from West Valley City discharges into the ponds that surround Stonebridge Golf Course. These ponds are the headwaters of the Riter Canal. There is a control structure just east of 5370 West that can be used to control releases from these ponds. The Riter Canal flows west approximately 3.5 miles from the control structure to the confluence with C-7 Ditch. Figure EX-9 shows the alignment of the Riter Canal and its tributary area.

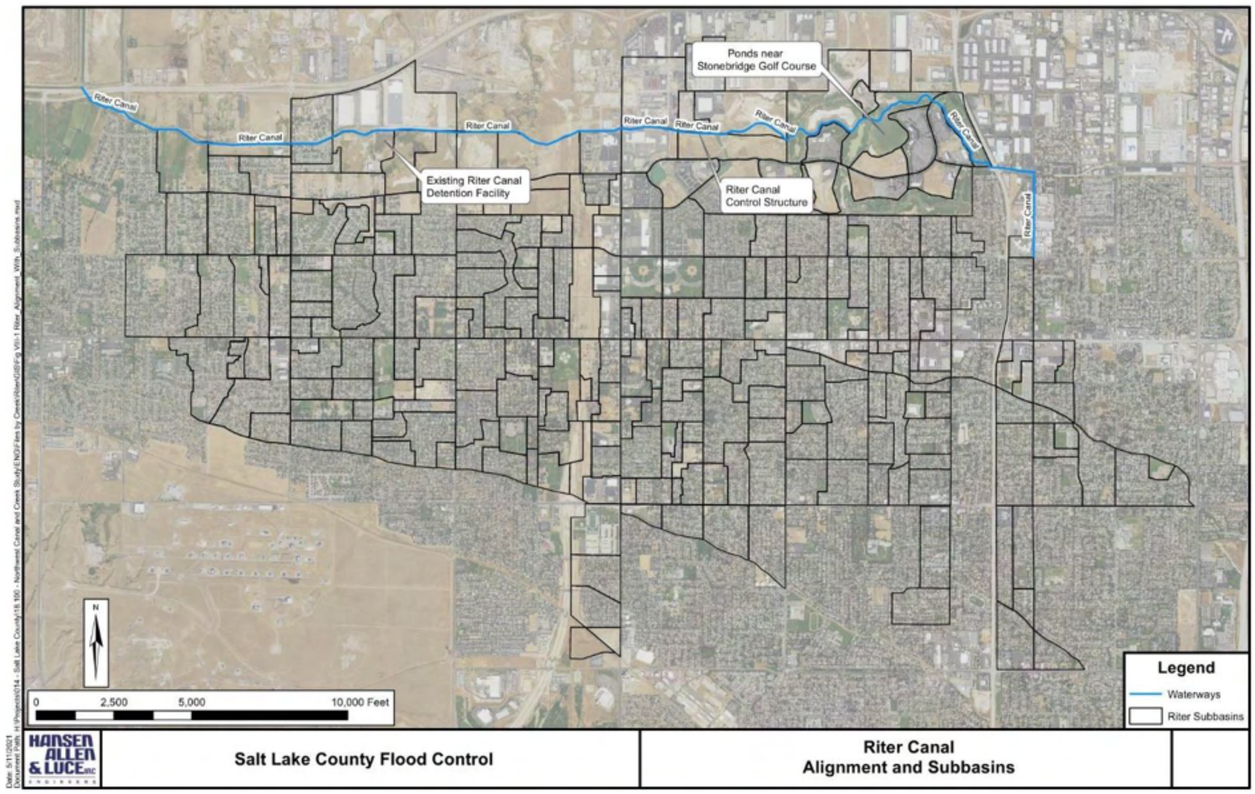


Figure EX- 9 – Riter Canal

The 10 and 100-year flows for existing conditions both show overtopping the banks of the Riter Canal in low bank areas. For the 10-year events most of the areas showing inundation are due to the low-lying areas along the channel that were undeveloped at the time of the LIDAR mapping (2013).

The preferred Solution involves a combination of controlling vegetation such that the channel roughness Mannings N is 0.05 or less and raising the banks at key locations to achieve 1-foot of freeboard for the design flow events.

The preferred solution includes controlling peak flow releases from the ponds near Stonebridge Golf Course, detention in two low lying areas on the south side of the Riter Canal (about 6700 West and 6200 West), and an overflow structure to Lee Creek at about 6200 West.

GOGGIN DRAIN

Goggin Drain begins on the west side of the Salt Lake International Airport at approximately 5100 West and 1000 North receiving flows from the Surplus Canal. The Goggin Drain flows generally to the west and passes under the North Point Canal on its way to the Great Salt Lake. The extent of the Goggin Drain and the areas directly tributary to it are shown in Figure EX-10.

The Goggin Drain receives flood waters from the Jordan River via the Surplus Canal. The flood flows in the Jordan River are managed in accordance with the Utah Lake Compromise Agreement.

Storm runoff and Goggin Drain hydraulic models prepared by HAL in 2011 were updated using the Northwest Quadrant Master Plan, which was adopted in 2016. Specifically, the Future Land Use Map was used to predict what development will look like in this area at buildout.

The estimated maximum historic peak flow in the Goggin Drain is 2,500 cfs. This was added to the predicted storm runoff flows from the tributary area to estimate total flood flows under various detention scenarios. The 2011 Goggin Drain hydraulic model was combined with 2013 Lidar data and used to predict the floodplain extents for the various detention scenarios.

The predicted Goggin Drain floodplain with 2,500 cfs from the Surplus Canal added to about 560 cfs (100-year storm runoff event with 0.2 cfs/acre detention) and a Great Salt Lake level of 4214.0 is shown on Figure EX-11. The 0.2 cfs/acre detention alternative results in a predicted increase in flooding of 26 acres (see light blue shaded areas on Figure EX-11).

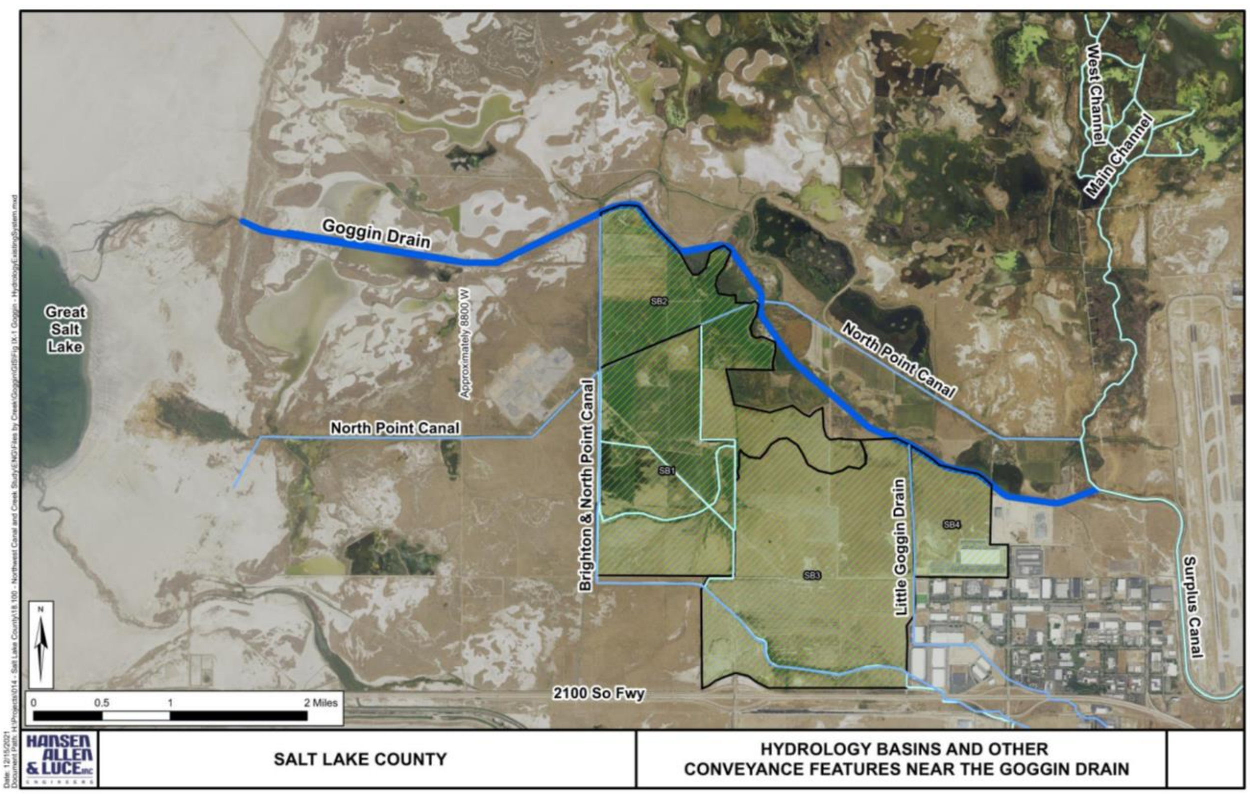


Figure EX- 10. Goggin Drain

The preferred solution requires development to detain storm water runoff to 0.2 cfs/acre in a 100-year event consistent with current Salt Lake City detention requirements.

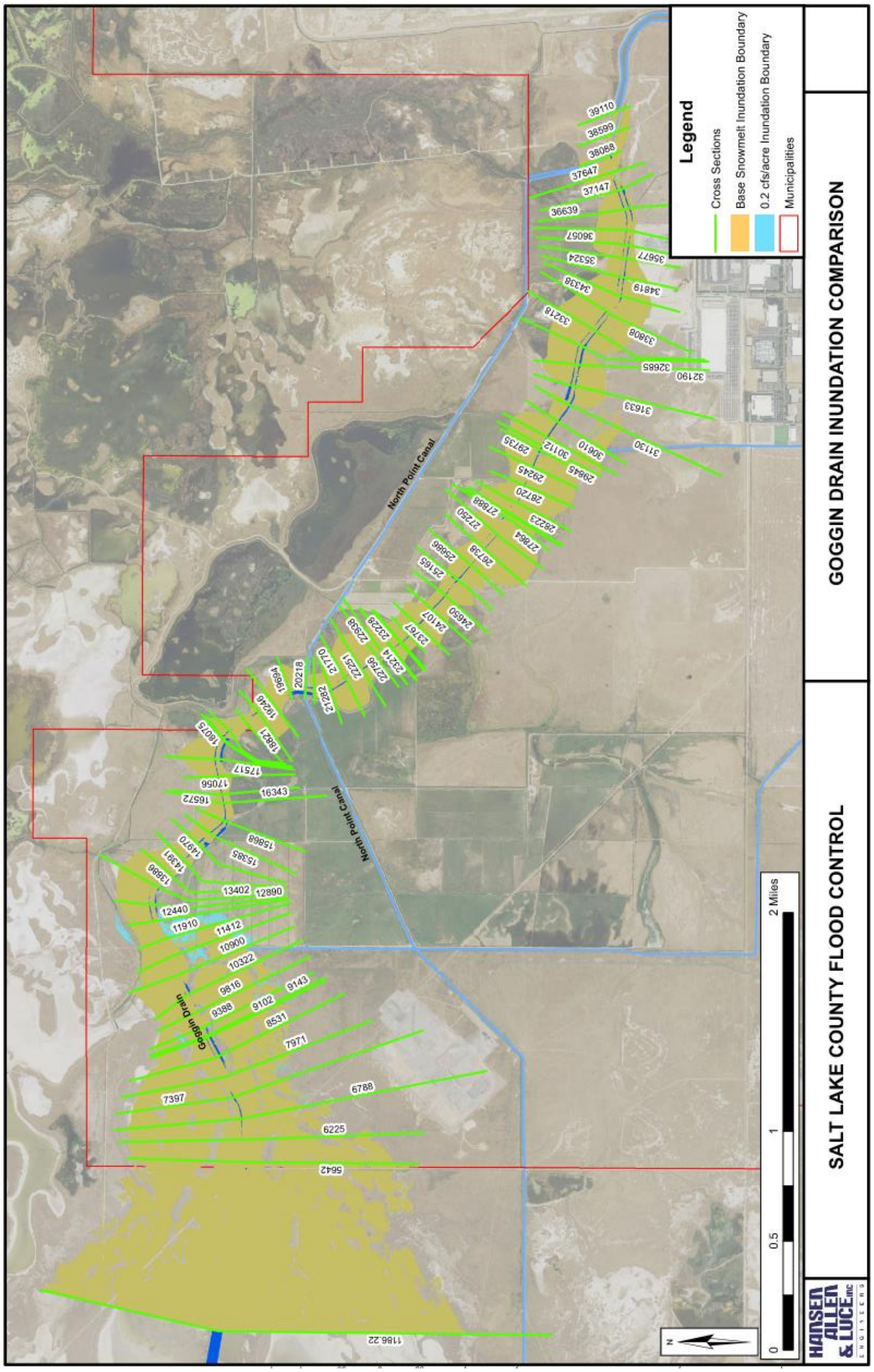


Figure EX-11. Goggin Drain Floodplain Comparison

KEARNS-CHESTERFIELD DRAIN

The Kearns-Chesterfield Drain and tributary area are shown on Figure EX-12.

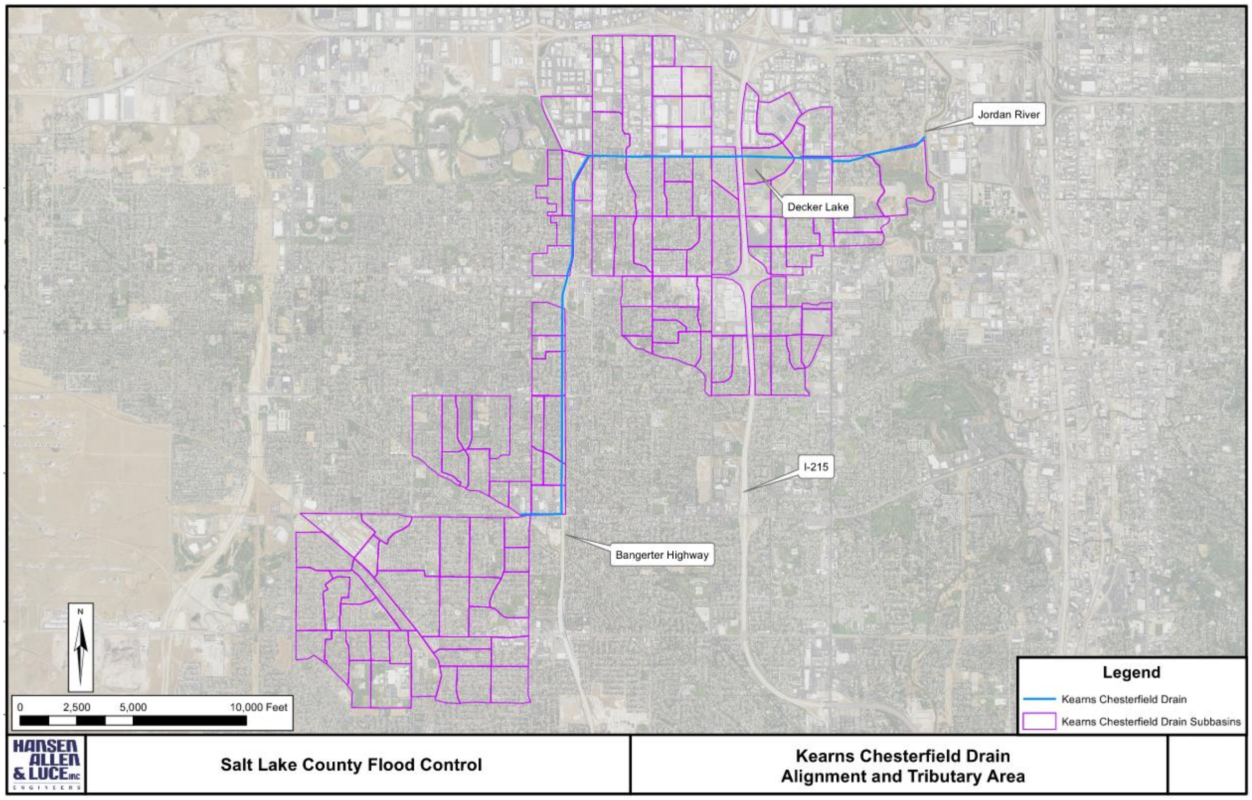


Figure EX- 12. Kearns-Chesterfield Drain

The predicted 10-year storm peak runoff flowrates are shown on Figure EX-13. The hydraulic models included both a SWMM model for the long conduit in Bangerter Highway and a HEC-RAS model for the mostly open channel segment from Bangerter Highway to the Jordan River.

The models predict that the Kearns-Chesterfield Drain can convey the runoff from a 10-year storm event without flooding.

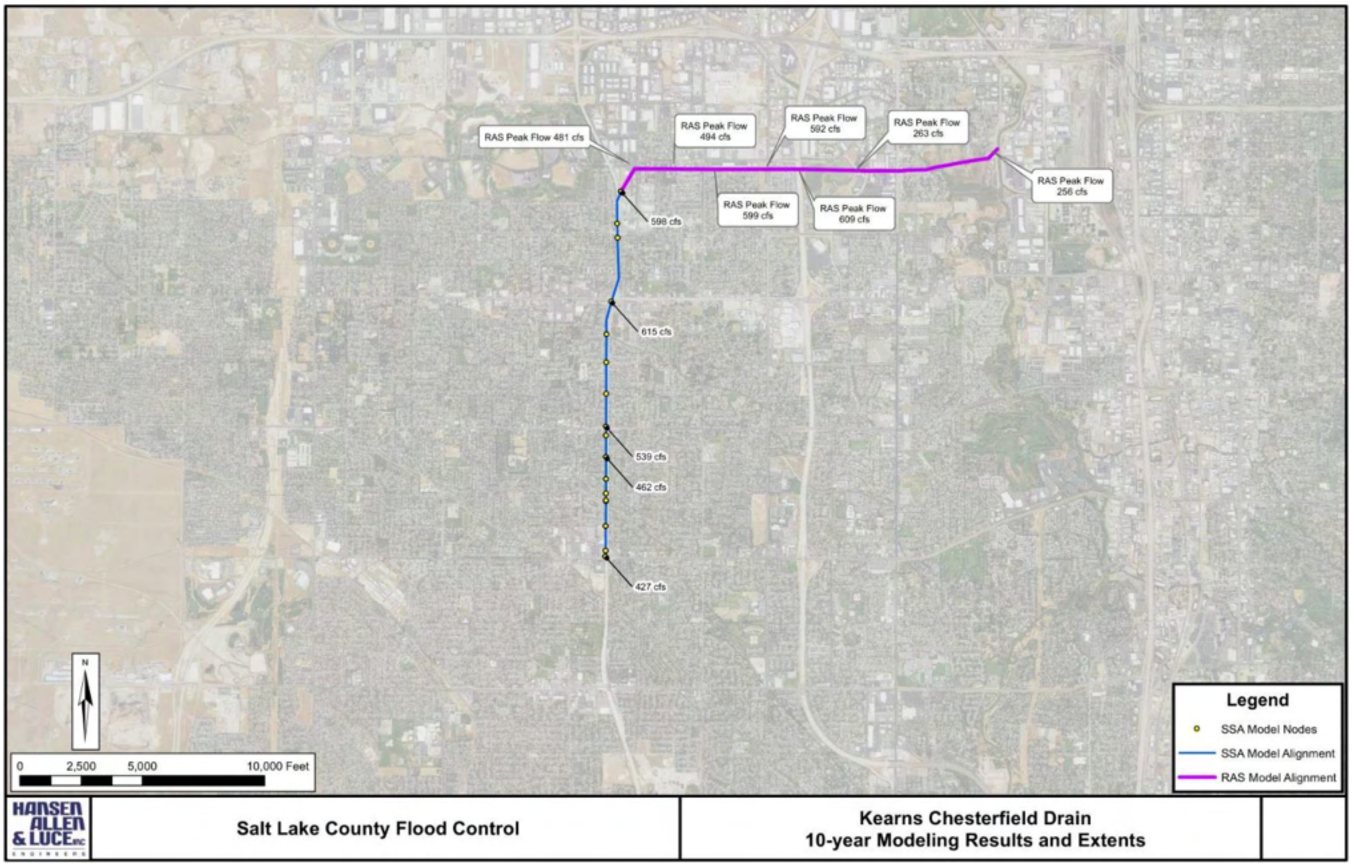


Figure EX- 13. Kearns-Chesterfield Drain Modeling Extents

TABLE EX-6. NORTHWEST CANAL AND CREEK STUDY SUMMARY

Conveyance	Existing Deficiencies/Comments	Master Plan Recommendation
C-7 Ditch	Some flooding of undeveloped land. There has been significant sediment deposition in lower reaches.	Development of a sedimentation monitoring plan is recommended. New developments should be constructed to provide a minimum of two feet of freeboard from building first floor elevations to the predicted 1% chance flood elevations.
Lee Creek	The existing UDOT Highway 201 culvert was sized based on upstream detention. Some of the existing storage is provided on private land. The Lee Creek channel is poorly defined between Lee Kay Ponds and C-7 Ditch and does not have sufficient capacity.	It is recommended that the storage area upstream of 201 be formalized with easements or public ownership. A plan including channel and culvert improvements downstream of Lee Kay Ponds is included to provide the 100-year conveyance capacity. New developments should be constructed to provide a minimum of two feet of freeboard from building first floor elevations to the predicted 1% chance flood elevations.
Lee Drain	Existing conveyance capacity is generally less than half of the predicted 100-year storm runoff peak flows. Most of the tributary area to Lee Drain has already developed as commercial/industrial.	Channel and culvert improvements are needed to provide the 100-year conveyance capacity. Coordination is needed between Salt Lake County and Salt Lake City concerning sharing construction costs and long-term maintenance.
Kersey Creek	The existing Kersey Creek conveyance is poorly defined and has limited capacity.	A master plan including proposed channel and culvert improvements for the 100-year event is included. New developments should be constructed to provide a minimum of two feet of freeboard from building first floor elevations to the predicted 1% chance flood elevations.
Utah & Salt Lake Canal	The Utah & Salt Lake Canal (USLC)has capacity for predicted 100-year storm runoff flows assuming detention of new developments. Development detention assumptions are shown on Figure V-1. The 9180 West culvert on the USLC Extension does not have capacity for the 100-year event. Some flooding from the USLC Extension is predicted in the golf course.	The USLC Extension is not a county facility. To provide 100-year capacity, the USLC Extension 9180 West Culvert capacity needs to be increased by either replacing the existing culvert or adding a parallel culvert. New developments in the golf course area should be constructed to provide a minimum of two feet of freeboard from building first floor elevations to the predicted 1% chance flood elevations.

Conveyance	Existing Deficiencies/Comments	Master Plan Recommendation
Riter Canal	Flooding is predicted in areas of low bank elevation along the Riter Canal. West Valley City owns/ controls the low-lying area at 6700 West and plans to preserve the detention storage.	The plan includes a combination of controlling vegetation such that the channel roughness Mannings N is 0.05 or less and requiring adjacent new development to construct such as to provide fill a minimum of one foot above the 1% chance flood elevations. The plan includes controlling peak flow releases from the ponds near Stonebridge Golf Course, detention in two low lying areas on the south side of the Riter Canal (about 6700 West and 6200 West), and an overflow structure to Lee Creek at about 6200 West. The plan includes purchase of the 6200 West detention area.
Coon and Harker Creek	The 4100 South Coon Harker Detention Basin is critical to providing flood control for Magna. Storm drain conveyance in 8000 West north of 3100 South is not adequate for the 10-year event.	The 4100 South Coon/Harker Detention Basin should be maintained on a regular basis including periodic inspection of the outlet works and vegetation control. Increase capacity of storm drain in 80th West north of 3100 South by adding a parallel 36" dia. storm drain from 3100 South to the C-7 Ditch. Future developments within the Coon/Harker's canyons are to detain storm runoff flows to pre-development conditions for the 2-year, 10-year, and 100-year runoff events.
Goggin Drain	Some flooding of undeveloped areas predicted for both existing and future developed conditions in the low areas near the Great Salt Lake.	The selected master plan solution requires development to detain storm water runoff to 0.2 cfs/acre in a 100-year event consistent with current Salt Lake City detention requirements. New developments should be constructed to provide a minimum of two feet of freeboard from building first floor elevations to the predicted 1% chance flood elevations.
Kerns-Chesterfield Drain	No deficiencies were identified in a 10-year storm event.	

CHAPTER I - INTRODUCTION

BACKGROUND

The Northwest Canal and Creek Study area (see Figure I-1) has unique drainage challenges and current high development pressures. The study area includes the following County drainage facilities.

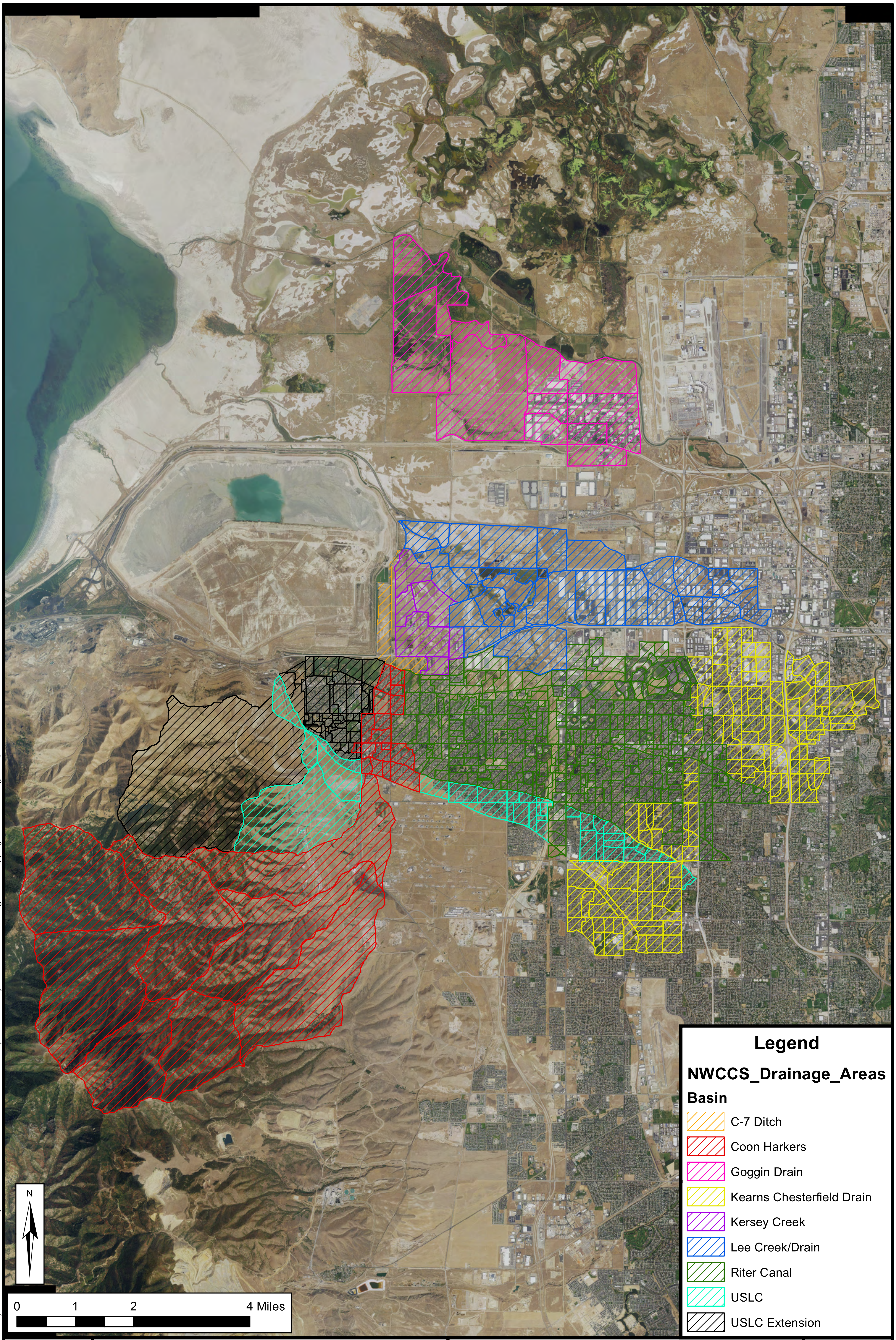
- C-7 Ditch
- Lee Creek
- Lee Drain
- Kersey Creek
- Utah & Salt Lake Canal
- Riter Canal
- Coon Canyon Creek and Harker's Canyon Creek
- Goggin Drain
- Kearns-Chesterfield Drain

Most of these drainages have interconnections. While previous studies have been completed of most of the individual conveyances, a study including all of these drainages had not been completed. Current high development pressures affect or have the potential to affect all of these drainages. Salt Lake County recognized the need for a planning effort which included analysis of the interconnections and provides guidance for management.

PURPOSE

The Northwest Canal and Creek Study (NWCCS) serves four purposes:

1. Provides storm water runoff models which predict how the NWCCS drainages respond to design storm runoff events.
2. Identifies and describes existing system problems.
3. Identifies alternative mitigation measures to eliminate flooding during design storm runoff events.
4. Provides recommendations for management of the County facilities and provides documentation of the preferred drainage improvements for each of the conveyances.



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HANSEN ALLEN & LUCE INC ENGINEERS

Salt Lake County Flood Control

Northwest Canal and Creeks Study
Drainage Area Map

FIGURE
I-1

CHAPTER II - HYDROLOGY METHODS

This section describes the hydrologic methods used to perform the storm runoff analysis for the study area, which includes a discussion of point precipitation data, depth area reduction factors, design storm distribution, and drainage basin characteristics.

POINT PRECIPITATION DATA

The National Oceanic and Atmospheric Administration point precipitation depth frequency estimates data server (NOAA 14)[1] was used to define design point rainfall storm depths for each of the study drainages. The 10-year 3-hour precipitation depth was approximately 1.0 inches while the 100-year 3-hour depth was approximately 1.85 inches.

DEPTH AREA REDUCTION FACTORS

A key assumption in most hydrologic analyses includes the utilization of the Depth Area Reduction Factor (DARF). DARFs are utilized to convert point rainfall depths of a particular return period to an area-averaged precipitation depth for the same return period. The area is often assumed to be the size of the drainage area being analyzed, but should be representative of the area the storm cell covers for the drainage area of interest. The US Weather Bureau developed DARF curves that have been commonly used for many years shown in Figure II-1.

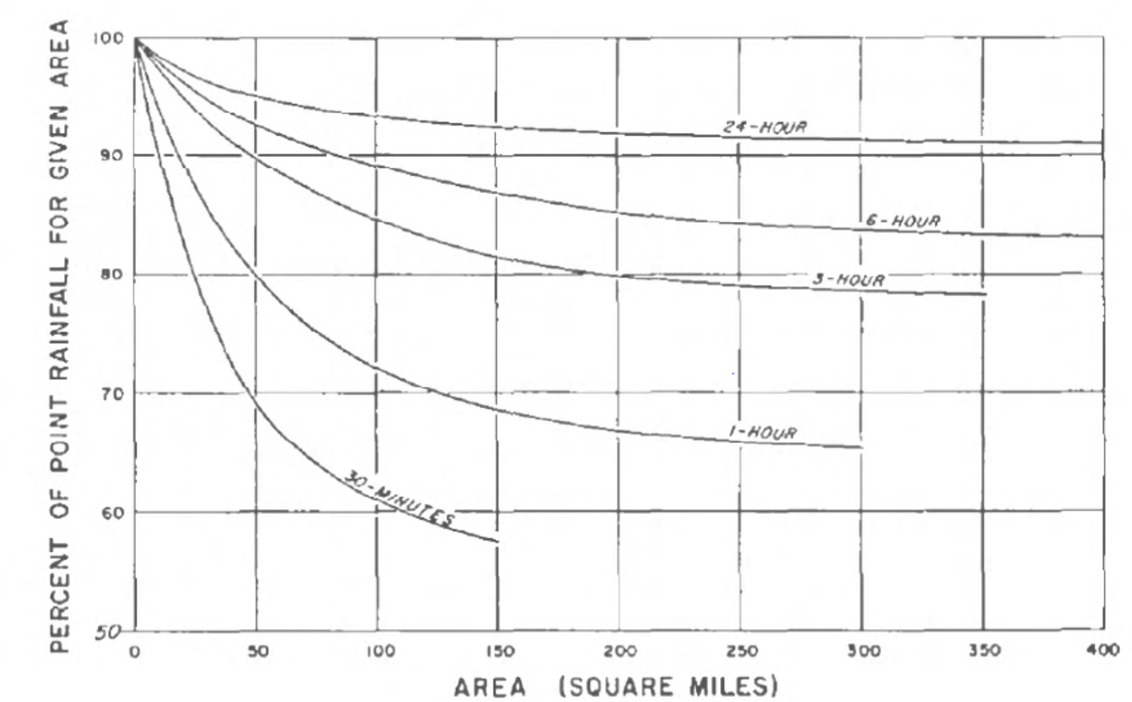

Figure II-1. Depth Area Reduction Factor Comparison (US Weather Bureau, 1961, Figure 15)[2].

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[1] Bonnin, G., D. Martin, B. Lin, T. Parzybok, M. Yekta, and D. Riley (2004, revised 2011). NOAA Atlas 14 Volume 1, Precipitation-Frequency Atlas of the United States, Semiarid Southwest. NOAA, National Weather Service, Silver Spring, MD.

[2] Hershfield, D. M. (1961). Technical Paper No. 40, Rainfall Frequency Atlas of the United States for

As higher resolution rainfall datasets (i.e. gage adjusted radar rainfall and radar rainfall data) have become more available in recent years several studies have shown that Depth Area Reduction Factors tend to decay more rapidly than values presented in the commonly used USWB. Results from a site-specific study performed in Colorado Springs is shown in Figure II-2 (Fountain Creek Watershed Rainfall Characterization Study, Carlton Engineering, 2011)

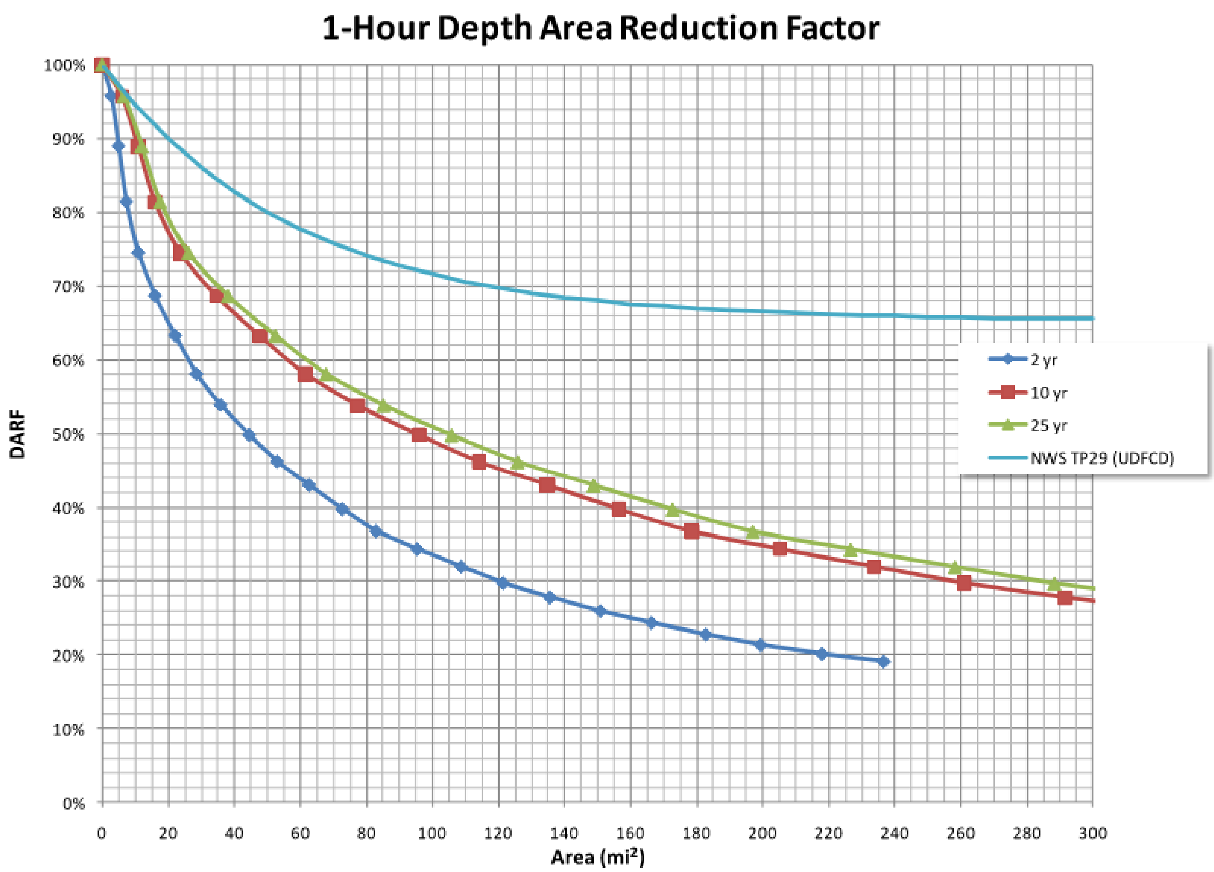

Figure II-2. Colorado Springs Site Specific One Hour DARF Curves (source Colorado Springs Study)[3].

For the purposes of this study a hypothetical elliptical storm shape that is presented in HMR-52[4] (ellipse with a 2:1 length to width ratio) was assumed to be a representative storm cell shape for the purposes of developing design storms for each of the areas analyzed. Several of the drainage areas included in this study are linear and somewhat fragmented. For this reason, the design storm shape was overlaid on top of the delineated drainage areas so an estimate could be made for the storm cell coverage area that was required to cover all the drainage area that was being analyzed. The representative storm cell area required was utilized in the selection of the DARF factor.

HAL presented the County with results from some recent DARF factor studies that have utilized

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Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years. U.S. Weather Bureau, Washington D.C.

[3] Carlton Engineering (2011). Fountain Creek Watershed Rainfall Characterization Study. Colorado Springs, Colorado.

[4] USACE (1984). HMR52 Probable Maximum Storm (Eastern United States) User's Manual.

radar rainfall data and high-density rain gage networks to develop site specific DARF factors for locations in the western United States. The results of two specific studies that were reviewed and presented to the County as part of this study were from Walnut Gulch, AZ and Colorado Springs, CO. These studies were compared with the USWB DARF factor curves as well as the curves from a Salt Lake City Cloudburst study performed by the Corps of Engineers in 1976[5] Figure II-3 shows a comparison of the DARF curves for various return periods, storm durations, and the studies that were reviewed.

At the direction from the County the SLC 3-HR DARF factor curves (see Figure II-3) from the Salt Lake City Cloudburst study were utilized for this project.

DESIGN STORM DISTRIBUTION

The distribution of rainfall through the duration of a design storm is used to define design storm runoff hydrographs using the Army Corps of Engineers HEC-HMS computer model. The design storm selected by Salt Lake County is a three-hour duration storm which incorporates a Farmer-Fletcher[6] 1-hour first quartile storm event as the middle hour of the three-hour design storm. The design distribution is shown on Table II-1 and on Figure II-4.

TABLE II-1. Design Three Hour Storm Distribution.

TIME (minutes)	Cumulative Precipitation (unitized)
0	0
5	0.002
10	0.006
15	0.012
20	0.019
25	0.029
30	0.040
35	0.054
40	0.069
45	0.086
50	0.105
55	0.127
60	0.150
65	0.365
70	0.540
75	0.659
80	0.735
85	0.786
90	0.822

[5] USACE (1976). Project Cloudburst. Salt Lake City Utah. Sacramento, California.
[6] Farmer, E. E. and Joel E. Fletcher. 1972. *Distribution of Precipitation in Mountainous Areas*. Geilo Symposium, Norway

TIME (minutes)	Cumulative Precipitation (unitized)
95	0.847
100	0.866
105	0.880
110	0.893
115	0.905
120	0.920
125	0.932
130	0.943
135	0.954
140	0.963
145	0.971
150	0.978
155	0.985
160	0.990
165	0.994
170	0.997
175	0.999
180	1

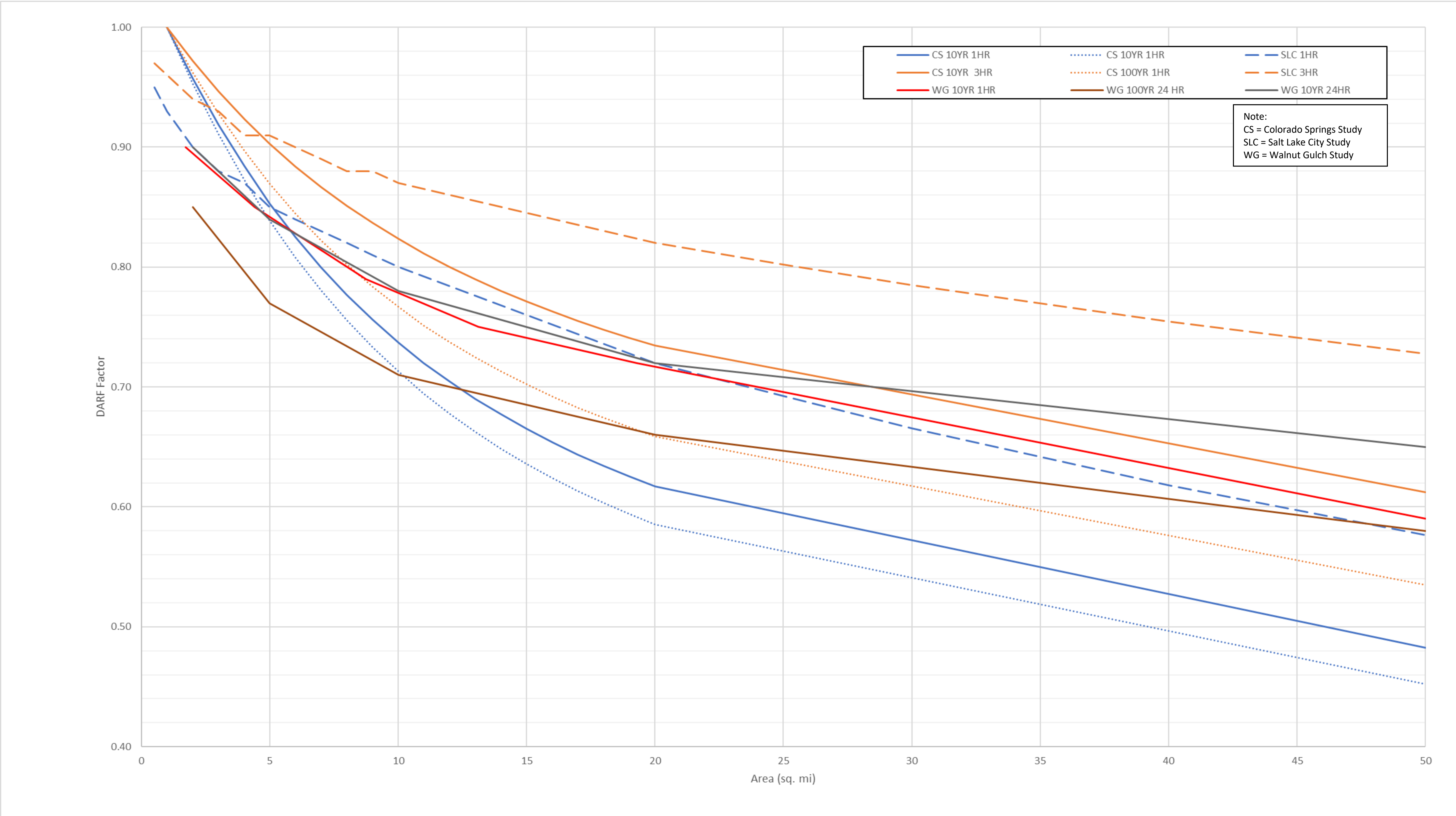


FIGURE II-3. DARF Curve Comparisons.

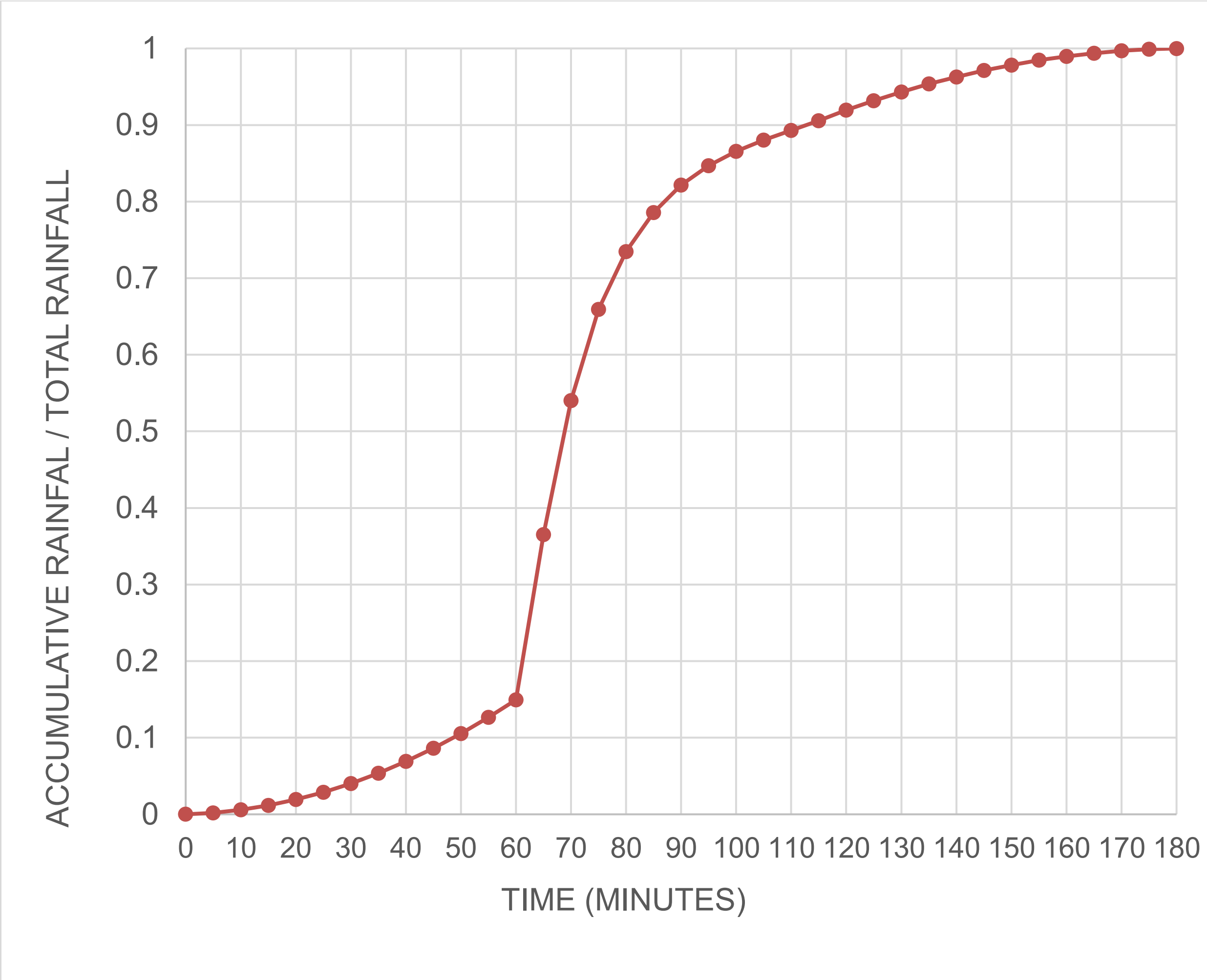


Figure II-4. Design Three Hour Storm Distribution.

DRAINAGE BASIN CHARACTERISTICS

A drainage basin is an area where all rainfall or snowmelt runoff within it will collect to a common point. Drainage basins may also be referred to as watersheds or catchments. Subbasins are smaller drainage basins located within a larger drainage basin. Drainage subbasin boundaries depend upon both the topography and the location of storm drainage facilities. The drainage subbasin boundaries delineated for the existing conditions models are shown on Figure I-1.

Subbasin characteristics were developed based on aerial mapping, 2013 Lidar data, and soils coverage from the state GIS site which comes from the Natural Resource Conservation Service database (NRCS, 2010). Existing hydrologic models were used when available. Subbasin characteristics include:

- Subbasin area
- Hydrologic Soil Type
- Percentage of impervious area
- SCS curve number
- Conveyance characteristics

Subbasin characteristics for future conditions were estimated using available land use planning information, zoning maps, and current detention requirements.

Impervious areas were estimated based on the National Agriculture Imagery Program (NAIP) imagery. This particular dataset includes infrared images which can be combined with the typical RGB images to identify healthy vegetation. This process is known as the Normalized Difference Vegetation Index (NDVI) approach. While this data is often used to identify healthy vegetation, the same process can be used to isolate areas that are impervious by using parts of the spectrum that are opposite the growing vegetation areas. An example of the impervious grid that was produced using this approach is shown in Figure II-5.

A residential neighborhood in West Valley City (shown on Figure II-5) was selected as a test sample to determine the percent of impervious area that is either directly connected or unconnected in residential neighborhoods.

Residential properties were visually inspected using satellite imagery and Google Street View to determine which portions of each home were classified as directly connected impervious area and unconnected impervious area. Drainage from sheds, garages, patios, and other impervious surfaces were also analyzed in the same way.

The two shape files (impervious area by image processing and digitized unconnected impervious area) were merged into a single shape file through a union. The polygons generated from overlapping areas represent the portion of the impervious area determined to be unconnected. The remaining impervious area from the image processing step is considered directly connected impervious area.

The results from this sample area indicated the following:

- Of the 112.65 acre residential sample area:
    - 37.46 acres were identified as impervious through image processing
- Of the 37.46 acres identified as impervious
    - 11.61 acres were determined to be unconnected. (30.98% of impervious area is unconnected)

Therefore, we will assume 31% of residential impervious area in the study is unconnected, and 69% of impervious area is directly connected.

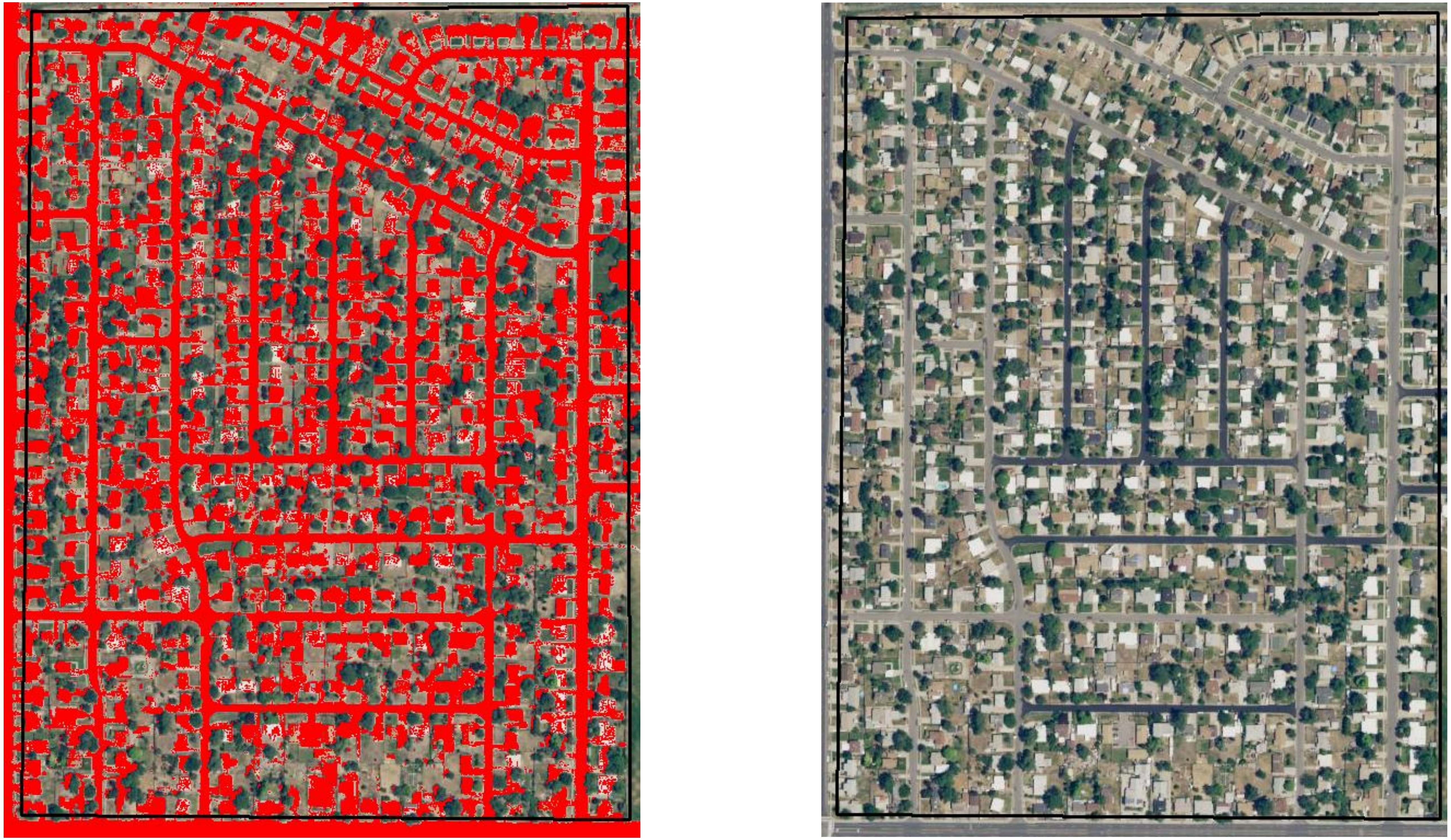


FIGURE II-5. Example Impervious Layer Generated with NAIP Imagery Data.

CHAPTER III – C-7 DITCH

EXISTING SYSTEM DESCRIPTION

The C-7 Ditch is a major Salt Lake County flood control facility. The following canals and creeks discharge into the C-7 Ditch.

- Utah & Salt Lake Canal
- Coon Canyon Creek and Harker's Canyon Creek
- Riter Canal
- Kersey Creek
- Lee Drain & Lee Creek

The C-7 is formed at the confluence of the Utah & Salt Lake Canal and the Riter Canal. It generally flows northward with a bend to the west as it approaches I-80. The C-7 passes under I-80 through large box culverts and then continues north until it discharges into the Great Salt Lake. It has been realigned since it was originally constructed, but has had its current alignment since the late 1990's according to the design drawings. This alignment is shown in Figure III-1.

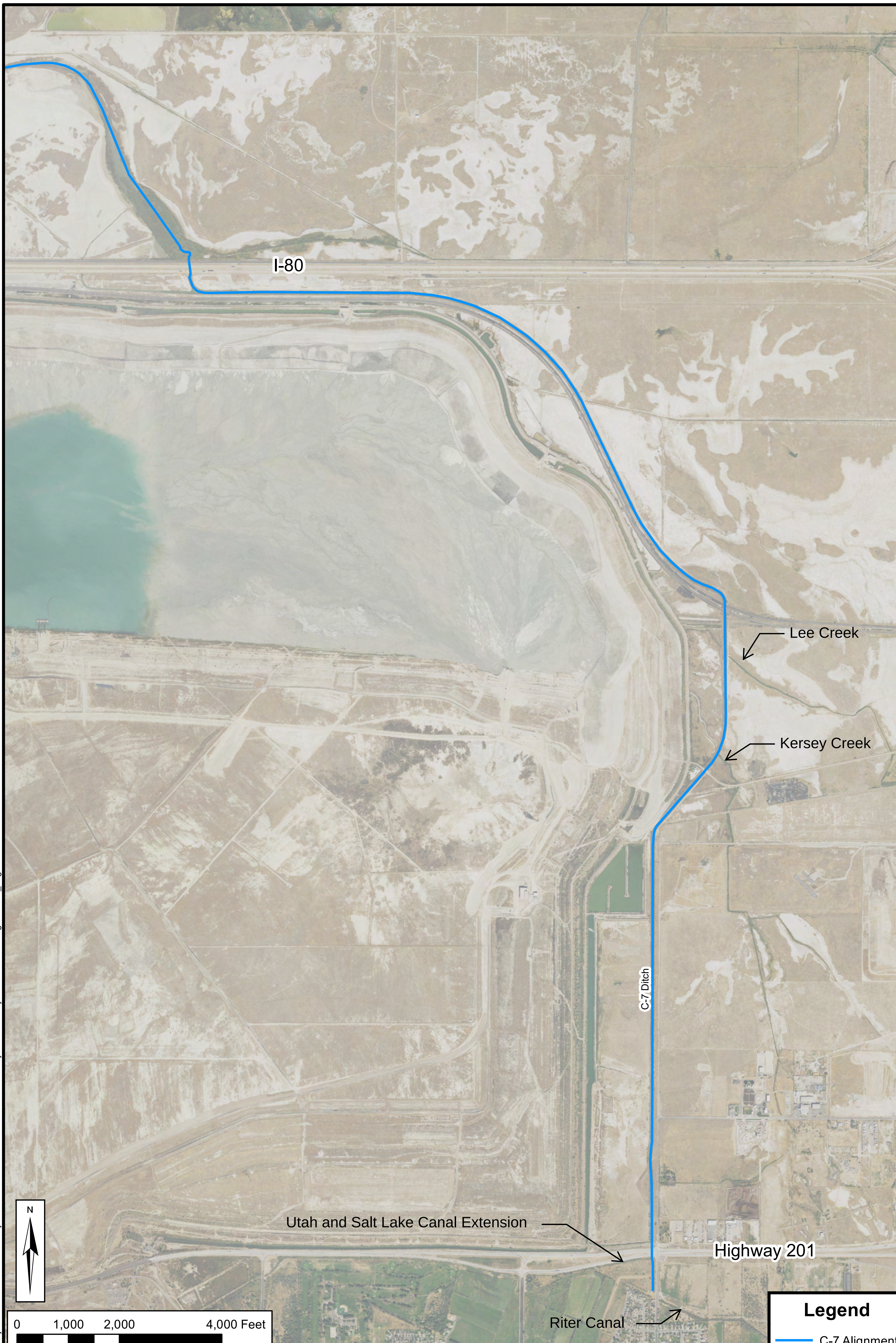
PREDICTED DESIGN STORM RUNOFF FLOWRATES

The majority of the C-7 Ditch runoff is dependent on flows from upstream flood control facilities. However, it is not realistic to assume that the 100-year peak flowrates from each of the upstream flood control facilities occur at the same time. A separate DARF factor was calculated for the C-7 Ditch tributary area and the hydrologic models for each of the upstream facilities were rerun utilizing the C-7-Ditch DARF factor. The DARF factor assumed for the C-7 was 0.67 which correlates to a storm coverage area of 75 sq miles as shown in Figure III-2. The calculated hydrographs were then used as input to the C-7 Ditch hydraulic model to evaluate capacity and flood potential.

EXISTING SYSTEM CAPACITY

HAL collected survey data along the bottom of the channel to verify actual bed elevations so an accurate estimate of capacity could be produced. Once the data was collected it was found there are some significant discrepancies between the surveyed channel bed elevations as compared to the design drawings we obtained as shown in Figure III-3.

The data suggests that there has been significant sedimentation at the downstream end of C-7. The area of sedimentation includes the existing culverts that go underneath I-80 and extends approximately 9,000 feet upstream. The survey data was used as a basis for modifying the Lidar dataset so that a 2D model of the C-7 Ditch could be developed. The new modification layer tool from HEC-RAS 6.0 Beta version was used to adjust invert elevations and cross section shape to match the survey data and design cross section as shown in Figure III-4.



Salt Lake County Flood Control

C-7 Ditch Alignment

FIGURE III-1

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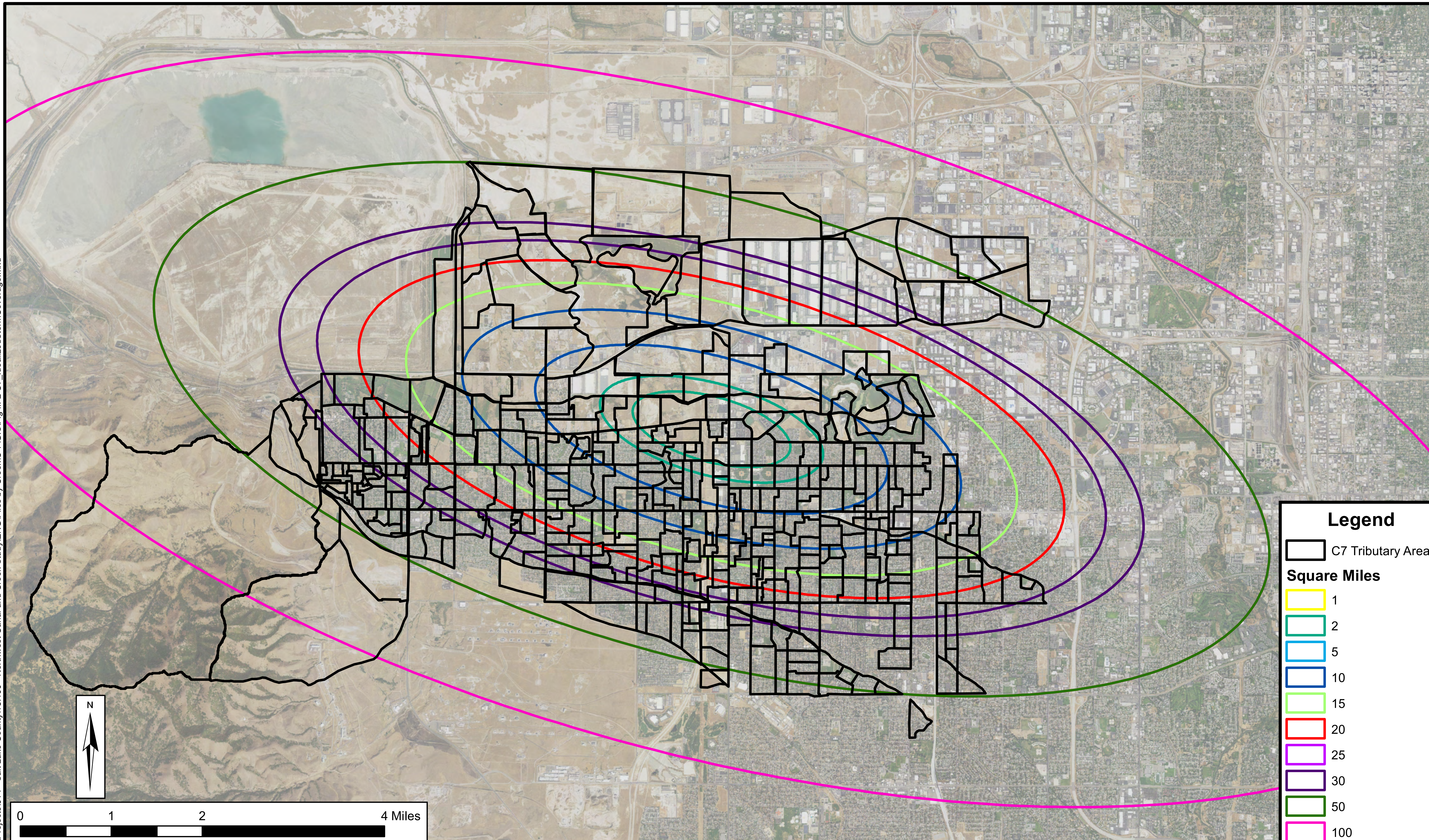

HANSEN ALLEN & LUCE INC ENGINEERS

Salt Lake County Flood Control

C-7 Ditch Drainage Area
Idealized Storm Area

FIGURE
III-2

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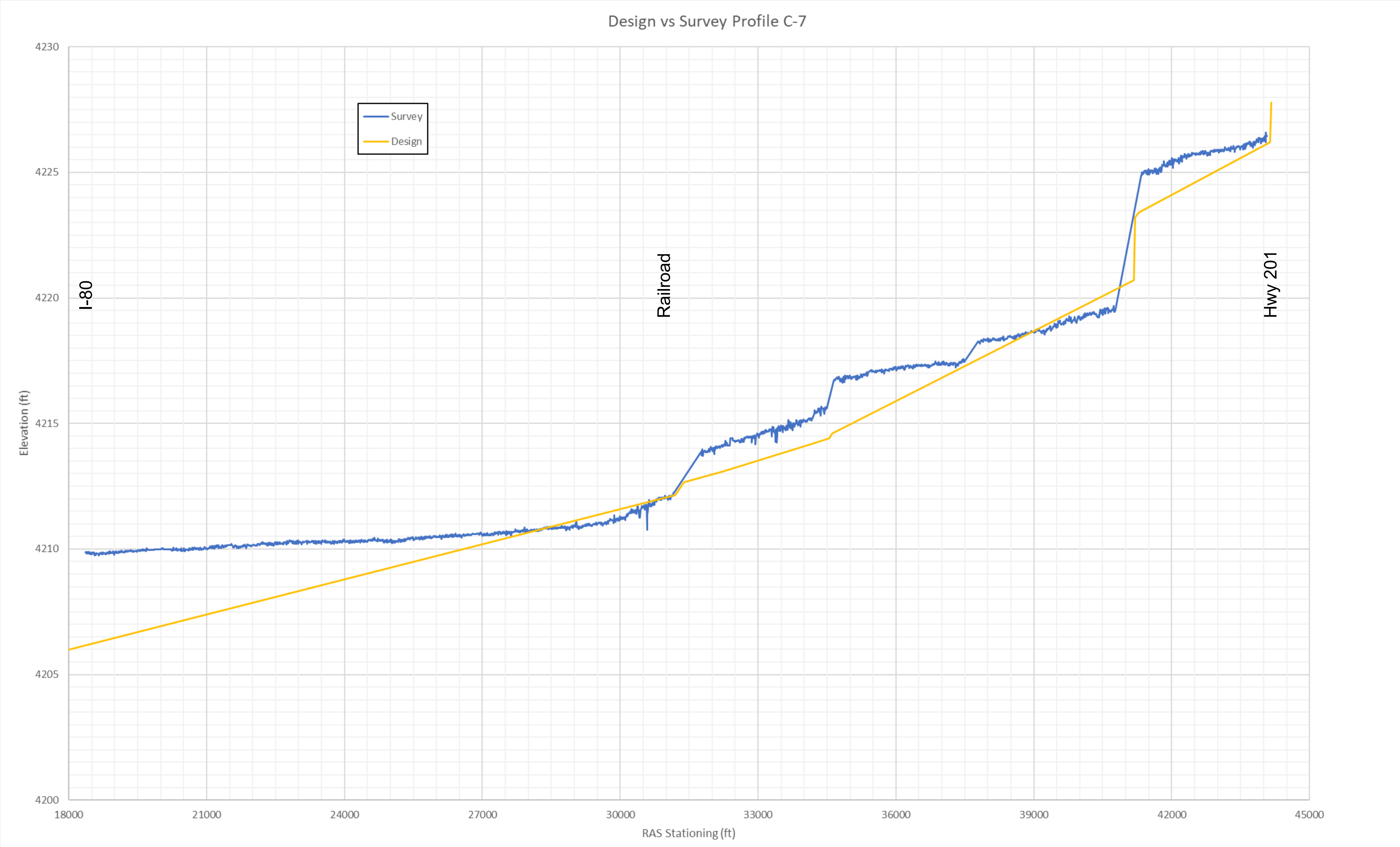


FIGURE III-3. C-7 Design vs. Survey Profiles.

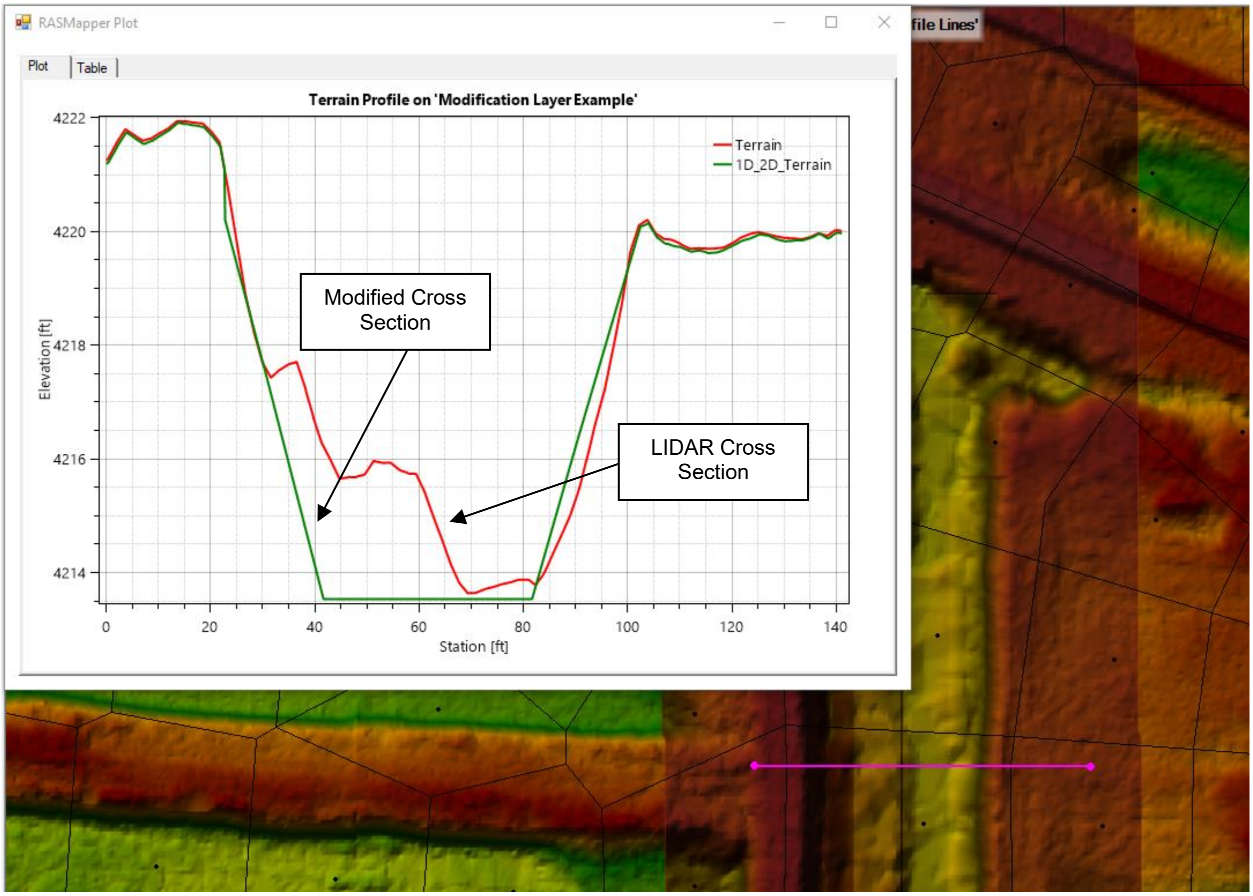


FIGURE III-4. C-7 Terrain Modification Example.

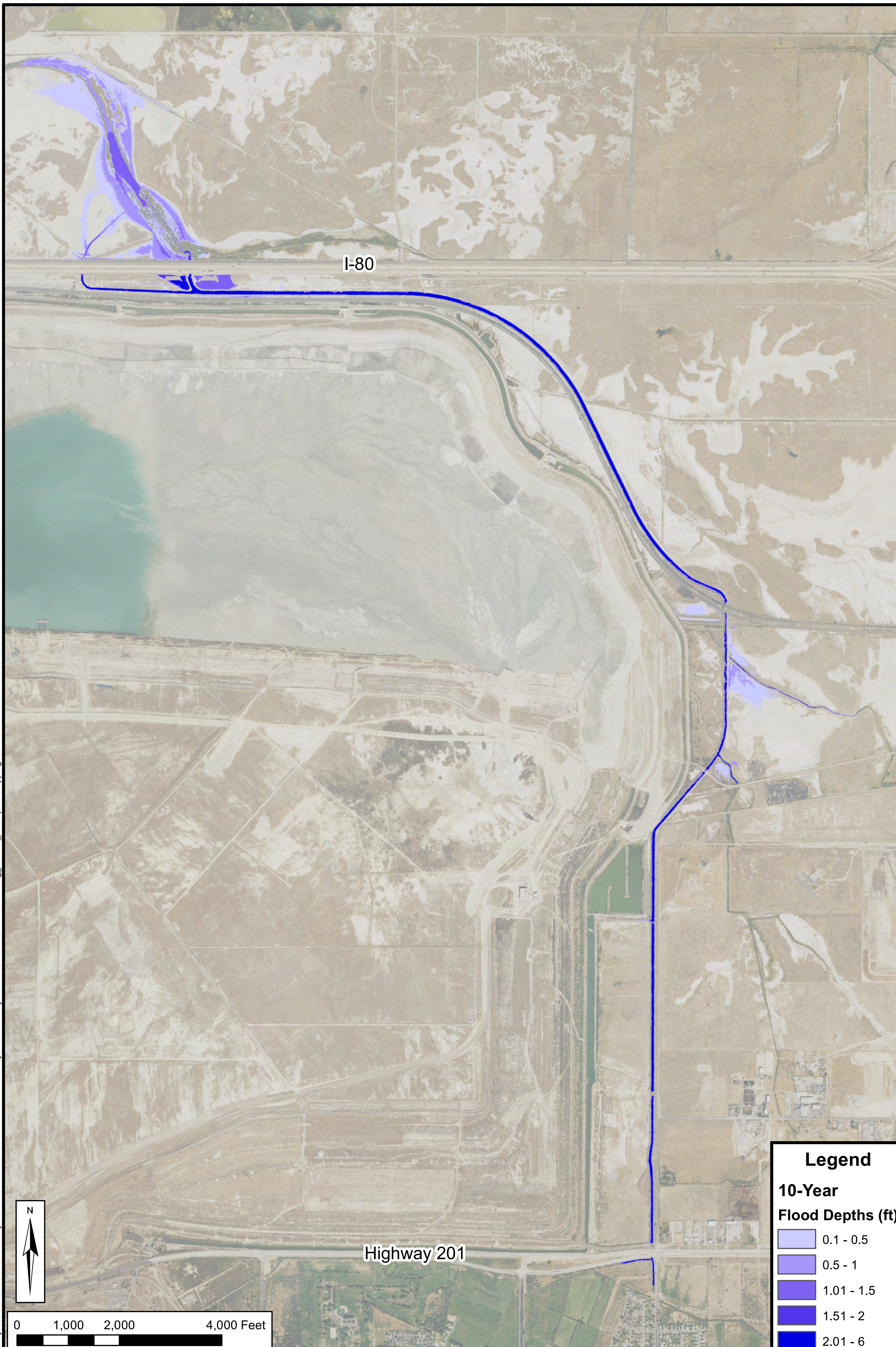
For both the 10-year and 100-year events there are some flooded areas. Particularly at the locations where Lee Creek and Kersey Creek enter the system. Figure III-5 shows the flooding extents for the existing conditions 10-year event and Figure III-6 shows the flooding extents for the existing conditions 100-year event.

C-7 HYDRAULIC ANALYSIS

The C-7 hydrologic/hydraulic analysis predicts minor flooding along the C-7. The flooding generally occurs in undeveloped areas. If and when development pressures come to this area the inundation maps that have been prepared will assist the County in guiding development in these areas.

The future flows for the C-7 increase when considering future developments along Lee Creek and Kersey Creek as well as implementing improvements along the Riter Canal.

The C-7 is heavily vegetated along its banks. Figure III-7 through Figure III-10 shows how thick the vegetation along the C-7 was at the time we performed our survey. Clearing the vegetation along the banks of the C-7 would improve its conveyance capacity.



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Document Path: H:\Projects\014 - Salt Lake County\18.100 - Northwest Canal and Creek Study\ENG\Files by Creek\C-7\GIS\FIG III-5 Existing_10Year_FloodplainMapping.mxd

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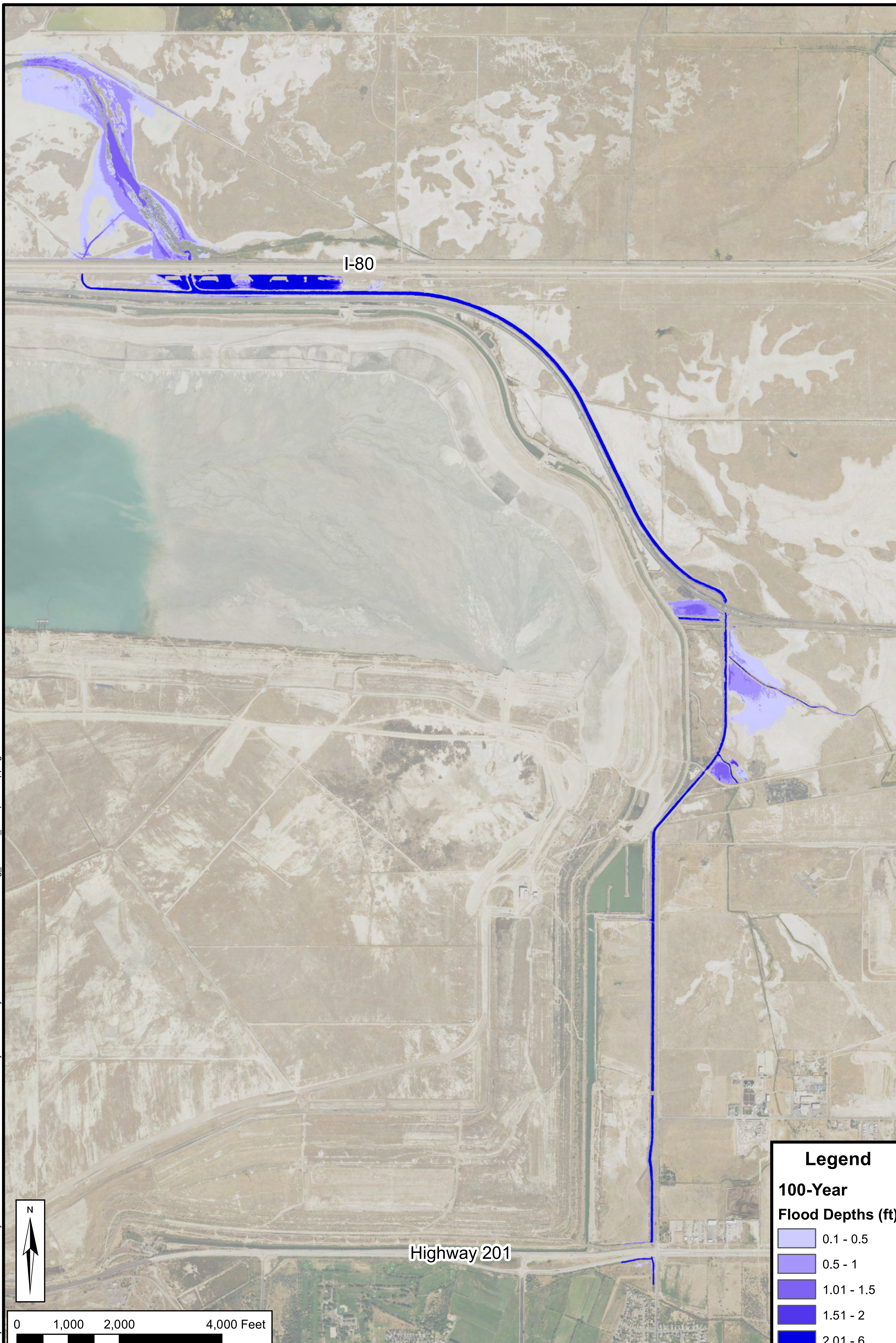
Salt Lake County Flood Control

Existing 10-Year
C-7 Ditch Floodplain

FIGURE
III-5



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Salt Lake County Flood Control

Existing 100-Year C-7 Ditch Floodplain

FIGURE III-6

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FIGURE III-7. C-7 Vegetation Thickness Example 1.

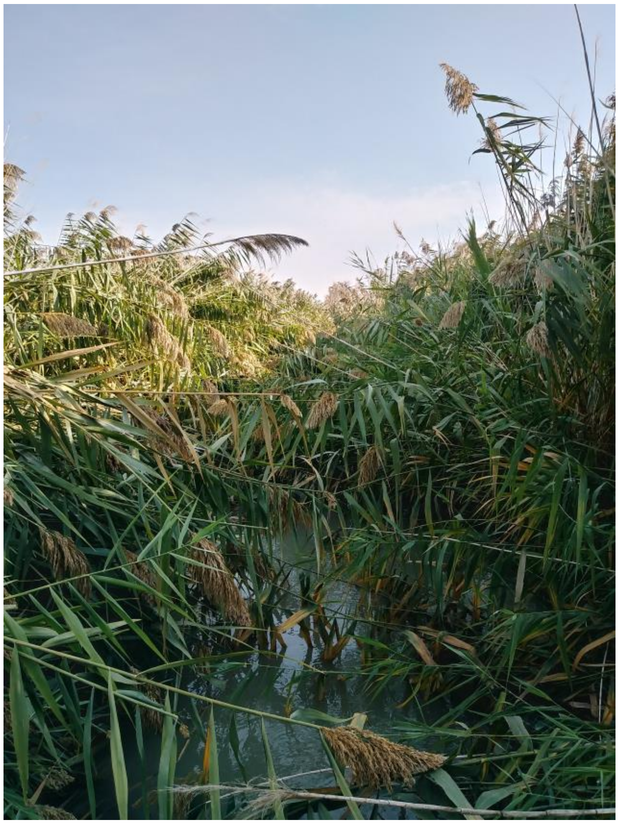


FIGURE III-8. C-7 Vegetation Thickness Example 2.

FIGURE III-9. C-7 Vegetation Thickness Example 3.

FIGURE III-10. C-7 Near I-80 Crossing.

When comparing the design drawing profile and the surveyed profile of the C-7 we find a significant amount of sedimentation has occurred at the downstream end of the reach. The box culverts that go under I-80 (based on the design drawings) appear to be approximately half full of sediment. Clearing out the sediment that has accumulated would also increase the capacity of C-7.

The floodplain maps for the future 100-year flows are shown in Figure III-11. A profile of the 100-year water surface elevations for existing roughness conditions and reduced channel roughness with the stationing starting at the south side of the I-80 culverts (assumes existing channel roughness n= 0.06 and reduced channel roughness n = 0.035) are shown in Figure III-12. The differences are based on the same inflow hydrographs, but the reduced roughness scenario allows for higher velocities and higher peak flows due to reduced travel times. Therefore, the comparison of max water surface elevation is not necessarily comparing the same peak flow rates. The backwater from the I-80 culverts reduces the benefit of the reduced roughness for the first few thousand feet of the C-7 as shown on the profile plot.

The computed peak flow rates at a few discrete locations along the C-7 for the future 100-year flows are provided in Table III-1. Additional capacity and reduced water surface elevations (WSEs) can be achieved by reducing roughness along the banks and dredging the channel where sedimentation has occurred. It should be noted that the Lidar elevations on the upstream and downstream side of the culverts at I-80 are approximately equivalent and the gradients going towards the Great Salt Lake are very gradual. Dredging downstream of the I-80 culverts would also likely be required to get the full benefits of dredging the channel upstream. Restoring the channel profile to what is shown on the design drawings would require significant dredging ranging up to 4 feet over a stretch of approximately 3 miles. Areas prone to sedimentation typically remain prone to sedimentation even after dredging has occurred. It is unknown whether the C-7 is continuing to aggrade due to sedimentation. It is recommended that a sedimentation monitoring plan be developed for the C-7.

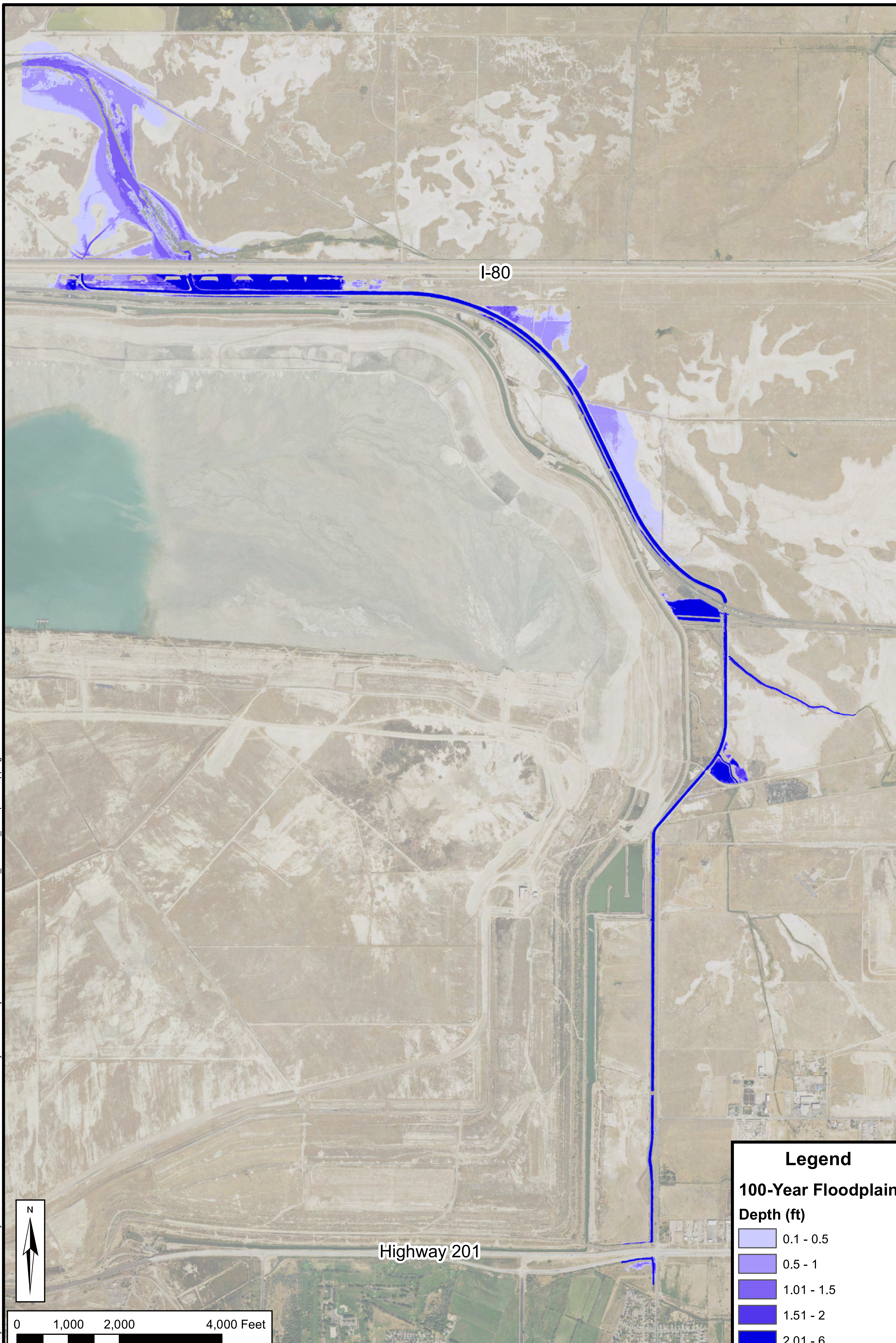
TABLE III-1. C-7 Design Peak Flowrates.

LOCATION	STA*	DESIGN FLOWRATE (100-YR) CFS Channel n = 0.06	DESIGN FLOWRATE (100-YR) CFS Channel n = 0.035
Downstream HWY 201	26,450	347	352
Upstream Kersey Creek	17,400	349	368
Downstream Kersey Creek	16,300	415	483
Upstream Lee Creek	14,800	405	477
Downstream Lee Creek	14,300	508	586
Bend Before I-80	7,850	349	449
Upstream of I-80	1,600	246	338

*STA is the distance in feet upstream from I-80.

RECOMMENDATIONS FOR C-7

The predicted minor flooding is currently of low consequence along the C-7 Ditch. Development plans along the C-7 are largely unknown at this point in time. Rather than presenting a single preferred solution it was determined in consultation with the County that the flood risk be mapped along the C-7 so if and when development occurs the flood risks are known and computed WSEs



HANSEN ALLEN & LUCE inc ENGINEERS	Salt Lake County Flood Control	Future 100-Year C-7 Ditch Floodplain Channel n = 0.06	FIGURE III-11

Date: 5/14/2021
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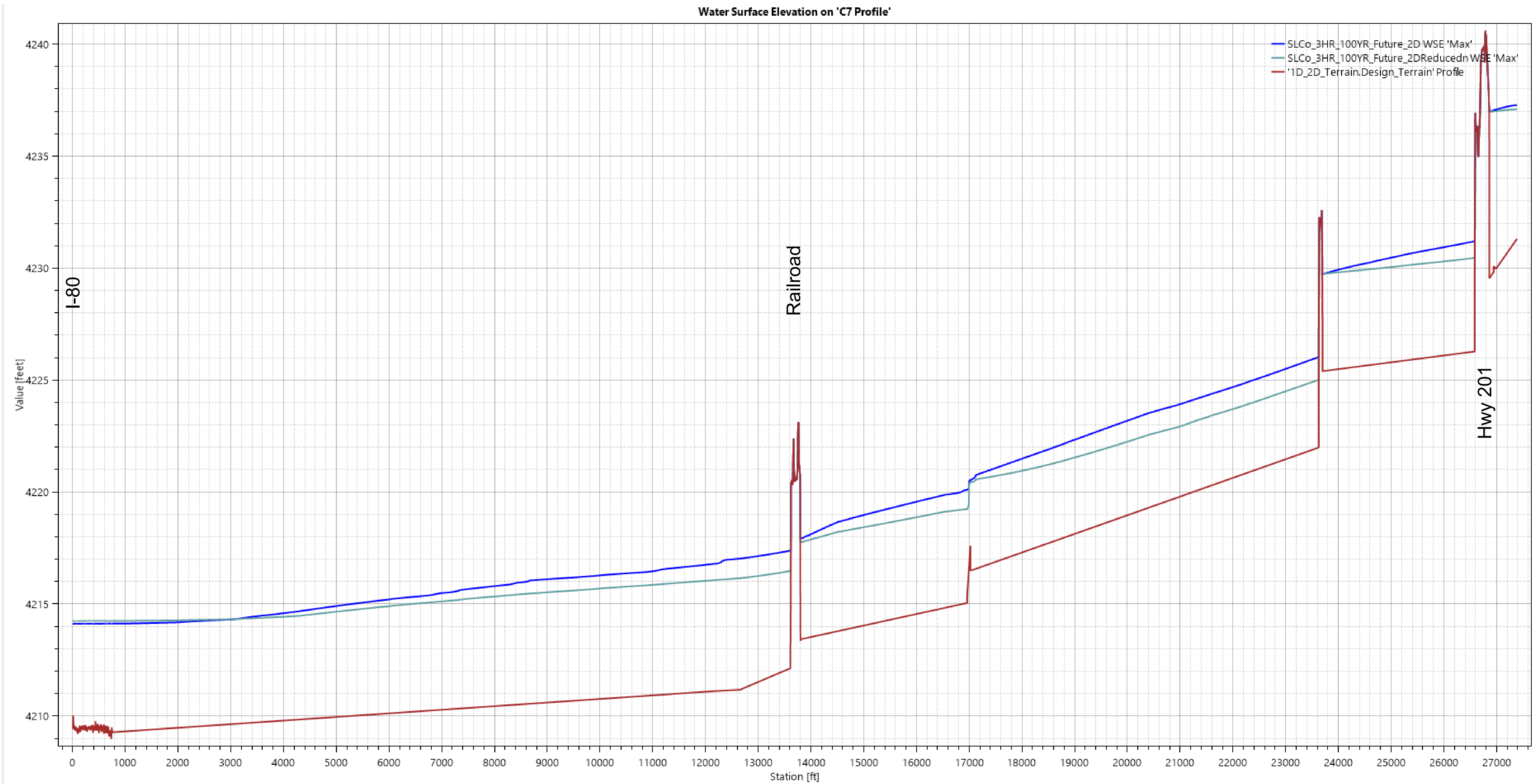


FIGURE III-12. C-7 Design Flow Profiles for Existing Channel Roughness (n= 0.06) and Maintained Channel Roughness (n= 0.035).

are also known so the County can advise future developers of how development can occur while protecting against flood risks.

The following are recommendations for guiding development along the C-7 Ditch

- Follow through with the preferred solutions for the conveyances that are tributary to the C-7.
- Control the vegetation along the channel banks.
- Require fill for development along the C-7 for surrounding ground elevations that are less than 1 foot above the computed WSE.
    - The required elevation can be extracted from the profile plot provided in Figure III-12, one foot should be added to the water surface elevations shown on the profile to achieve 1-foot of freeboard.
- Monitor sedimentation levels along the C-7 Ditch by collecting thalweg elevation data every 2-5 years. If sedimentation continues and further reduces conveyance capacity, develop a dredging schedule to maintain the desired capacity.

CHAPTER IV

CHAPTER IV – LEE DRAIN AND LEE CREEK

EXISTING SYSTEM DESCRIPTION

The Salt Lake County conveyance Lee Drain begins just downstream of a Salt Lake City pump station located at 2955 Andrew Ave (see Figure IV-1).

Lee Drain is tributary to Lee Kay Ponds (about 6000 West 1300 South) located south of the Salt Lake County Solid Waste Facility. Near the outlet from the Lee Kay Ponds, Lee Drain flows into Lee Creek. Lee Creek begins at the north side of the Riter Canal (about 2550 South 6000 West) and flows into the C-7 Ditch at about 7700 West 1000 South).

PREDICTED DESIGN STORM RUNOFF FLOWRATES

Aerial photographic mapping and 2013 Lidar data were used with land use mapping to define subbasins for the drainages directly tributary to the Lee Drain and Lee Creek system. Subbasins are shown on Figure IV-2 and the HEC-HMS model schematic is shown on Figure IV-3.

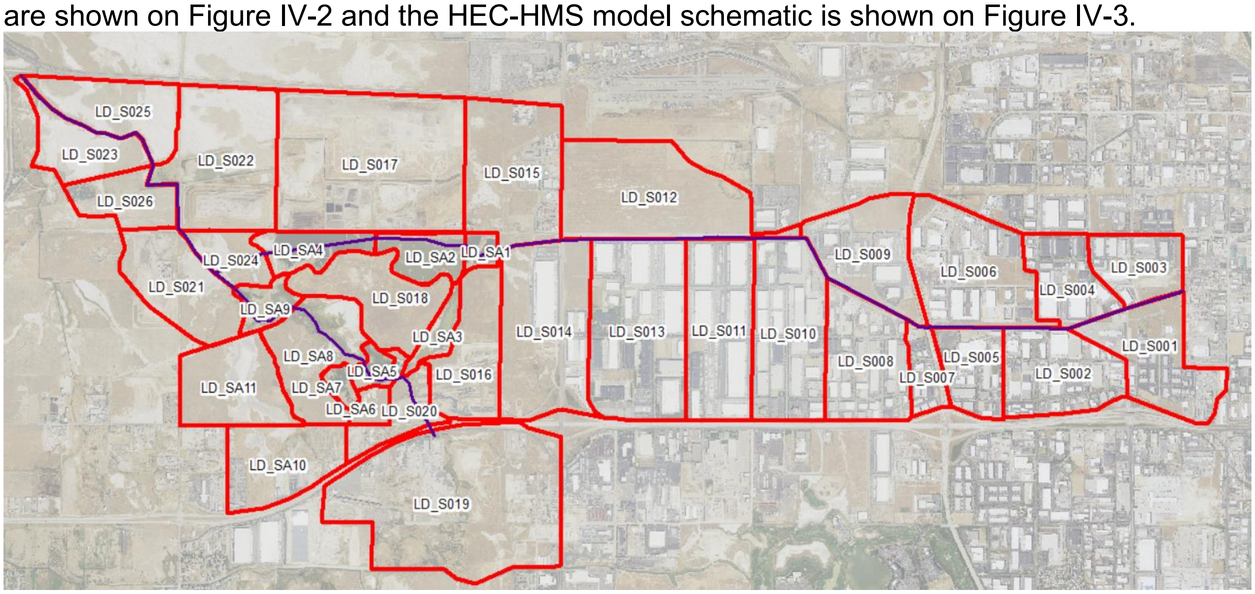
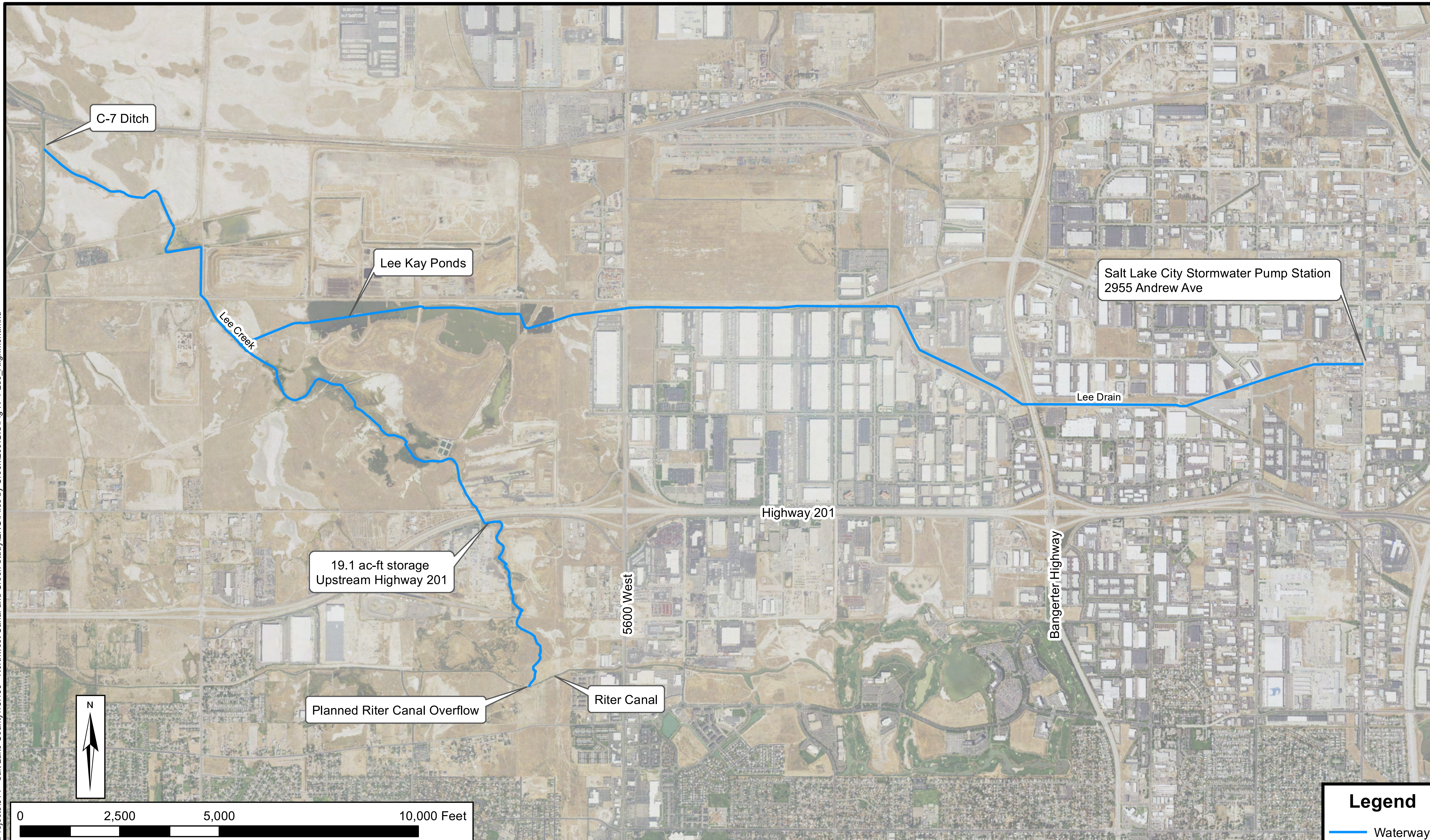


Figure IV-2. Lee Drain and Lee Creek Subbasins.



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HANSEN ALLEN & LUCE INC ENGINEERS

Salt Lake County Flood Control

Lee Creek and Lee Drain Alignments

FIGURE IV-1

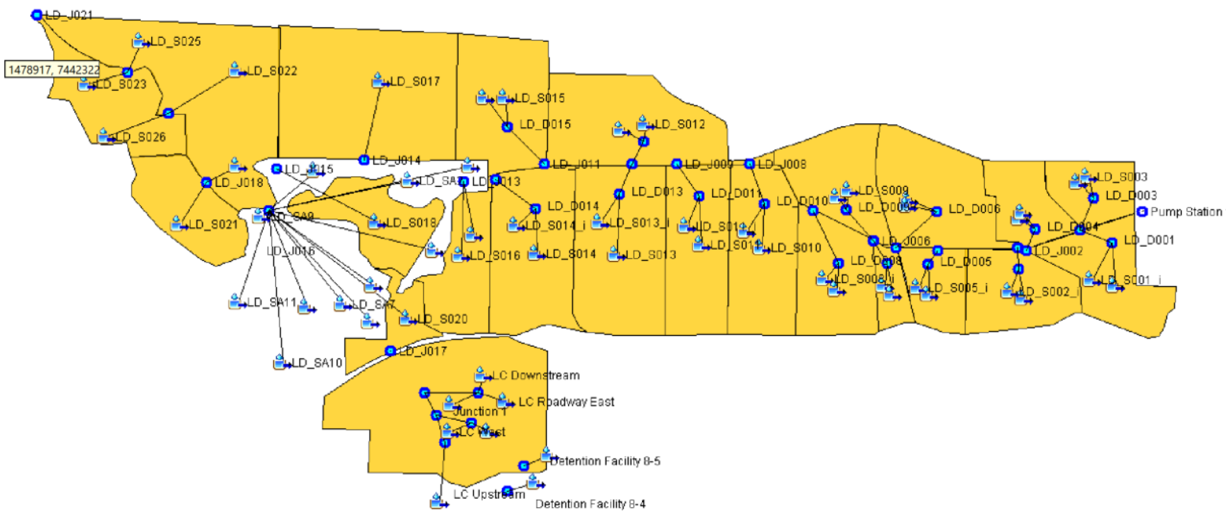


Figure IV-3 – Lee Drain and Lee Creek HEC-HMS Model Schematic.

The Salt Lake City pump station at the head of Lee Drain has a design capacity of 22 cfs. A summary of the predicted peak storm runoff flowrates is provided in Table IV-1 for Lee Drain above the Lee Kay Ponds.

Most of the area tributary to Lee Drain upstream of the Lee Kay Ponds has already developed. Existing and future developments are assumed to implement Salt Lake City's detention requirement which is a maximum allowable release rate of 0.2 cfs per acre in a 100-year event.

About five square miles of urban area is tributary to Lee Drain upstream of the Lee Kay Ponds. The hydrographs produced by the HEC-HMS model for discrete locations along Lee Drain are used as inputs to a HEC-RAS model (see Figure IV-4).

Figure IV-4. Lee Drain and Lee Creek HEC-RAS Model Schematic.

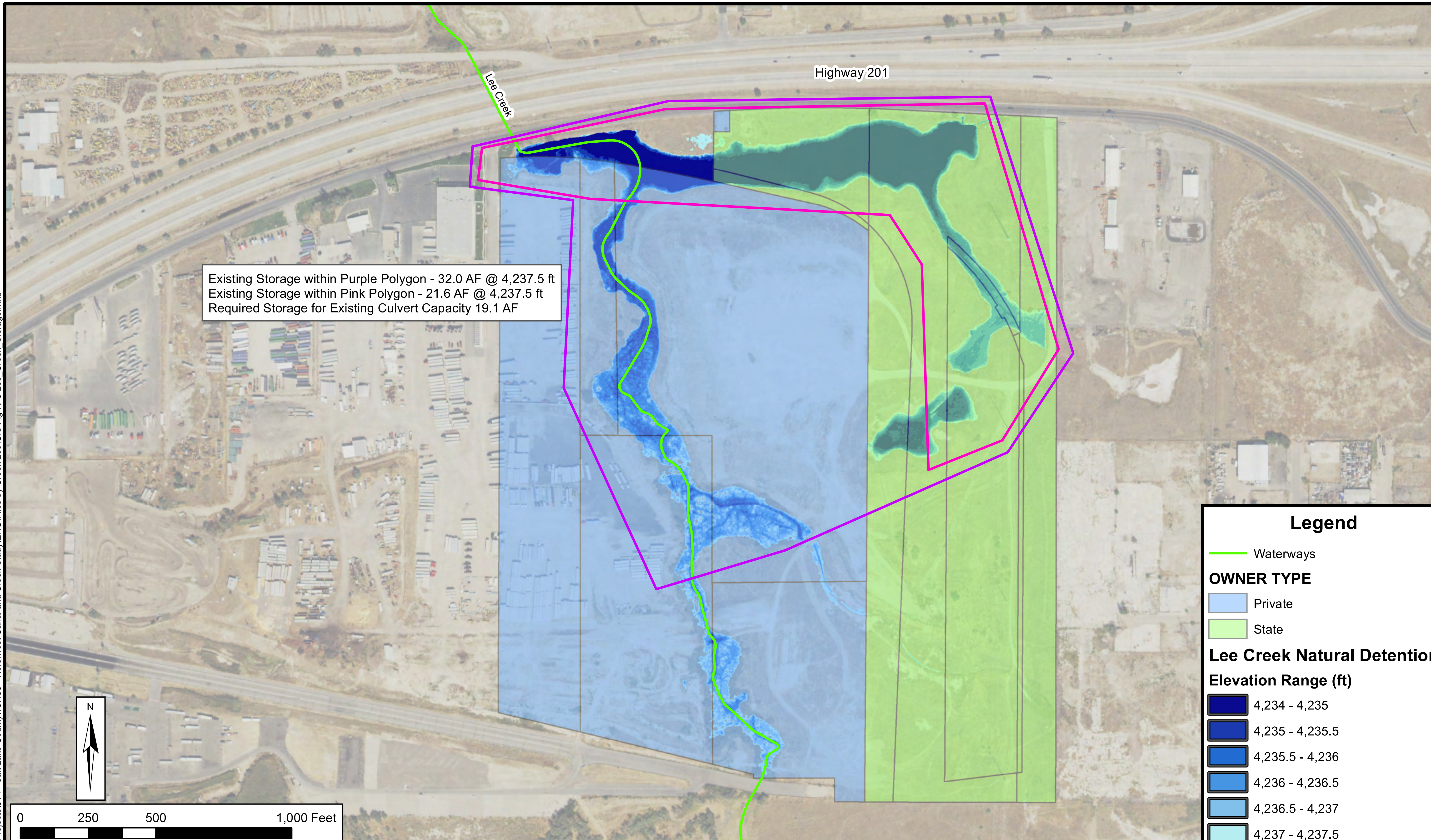
The County provided HAL with a design report along with its accompanying hydrologic and hydraulic models used to design the culvert that goes under Highway 201 and enters the Lee Kay ponds from the south[7]. The computed flows from this study were used as input into a 2D grid that would route those flows to the downstream Lee Kay ponds. A critical assumption of the HDR study was the availability of storage on the upstream side of Highway 201. Model results indicate that in order to attenuate projected flows to the design flow of the existing culvert (36 cfs) approximately 19.1 acre-ft of storage is required. The property that includes the existing natural detention has a mixture of State and private ownership. The majority of the area critical to maintaining the required detention volume is owned by UDOT. We recommend that the easements or sufficient property be acquired to maintain at least the minimum required storage for the system to function as designed. Figure IV-5 shows parcel ownership and elevations for the existing natural detention along with estimated existing storage volumes for two distinct footprints based on Lidar elevations. It is estimated that an additional 2.5 acres of private property will need to be acquired to secure the storage volume required for Lee Creek. A cost estimate for acquiring this land is provided in the Appendix.

The Lee Kay ponds were modeled as storage areas that were connected by pipes between high ground. The storage elevation curves were extracted by the HEC-RAS RAS mapper tool based on Lidar elevations. It is likely that there is some storage below the water surface elevation when the Lidar was taken, but for the purposes of this study it is adequate to assume no storage below the water surface elevation when the Lidar data was obtained. The water passes through the storage areas based on the hydrographs that enter the ponds and the difference in head between the connected storage areas. The upstream end of the ponds includes inflows from the 1D modeled reach of the Lee Drain and the 2D grid that includes the flows from the culvert that goes under Highway 201. The most downstream storage areas of the Lee Kay ponds are connected to a 2D grid that routes the releases downstream to the C-7 Ditch.

The existing Lee Creek conveyance between the Lee Kay Ponds and the C-7 Ditch is poorly defined and has limited capacity. The existing conditions model indicated that only about 25 cfs peak flow from a 100-year event makes it to the C-7 Ditch because of the flat gradients and general lack of a defined channel. It was determined in consultation with the County that for the purposes of this study a master planning effort would be conducted to provide a design channel with sufficient capacity to convey future 100-year flows.

Design peak storm runoff flowrates for selected locations along Lee Drain upstream of Lee Kay Ponds are provided in Table IV-1. Design peak storm runoff flowrates for Lee Creek are provided in Table IV-2. Lee Drain is considered a major drainage system with the goal of providing capacity for a 100-year design storm runoff event.

[7] HDR (2018). SR-85 MVC; 4100 South to SR-201. Salt Lake City, Utah.



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Salt Lake County Flood Control

Lee Creek Detention Plan
Upstream of Highway 201

FIGURE IV-5

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TABLE IV-1. Lee Drain Design Peak Flowrates.

LOCATION	STA*	DESIGN FLOWRATE (100-YR) CFS
Gladiola Street	18,600	73
Brighton & North Point Canal	16,879	73
Access Road	16,700	94
Rail Road	13,200	190
Bangerter HWY	12,900	190
Gramercy Road	12,200	232
Rail Road	8,400	266
4800 West	7,000	310
5070 West	5,300	351
5500 West	2,500	460
5600 West	1,800	460

*STA is the distance in feet upstream from Lee Kay Ponds.

TABLE IV-2. Lee Creek Design Peak Flowrates

LOCATION	STA*	DESIGN FLOWRATE (100-YR) CFS
Riter Canal Overflow	22,200	30
Hwy 201	16700	36
Flow Into Lee Kay Pond	14,750	105
Lee Kay Pond Outflow	8,600	10
1300 South	7,050	20
8000 West	4,200	80
C-7 Ditch	0	125

*STA is the distance in feet upstream from C-7 Ditch.

EXISTING SYSTEM CAPACITY DEFICIENCIES

The design peak flowrates (Table IV-1) assume that the Lee Drain channel and culvert capacity are sufficient to pass the 100-year runoff flows. The HEC-RAS hydraulic model of the existing system predicts that the existing upper Lee Drain system is inadequate for the design flows. A hydraulic profile showing locations where roads are overtopped and where predicted water surface elevations exceed the existing top of bank are shown on Figure IV-6. Flooding is predicted upstream of about 4000 West in a 10-year event. Salt Lake County personnel confirm that flooding has occurred in this area.

The existing culvert capacities (including tailwater effects) are provided in Table IV-3.

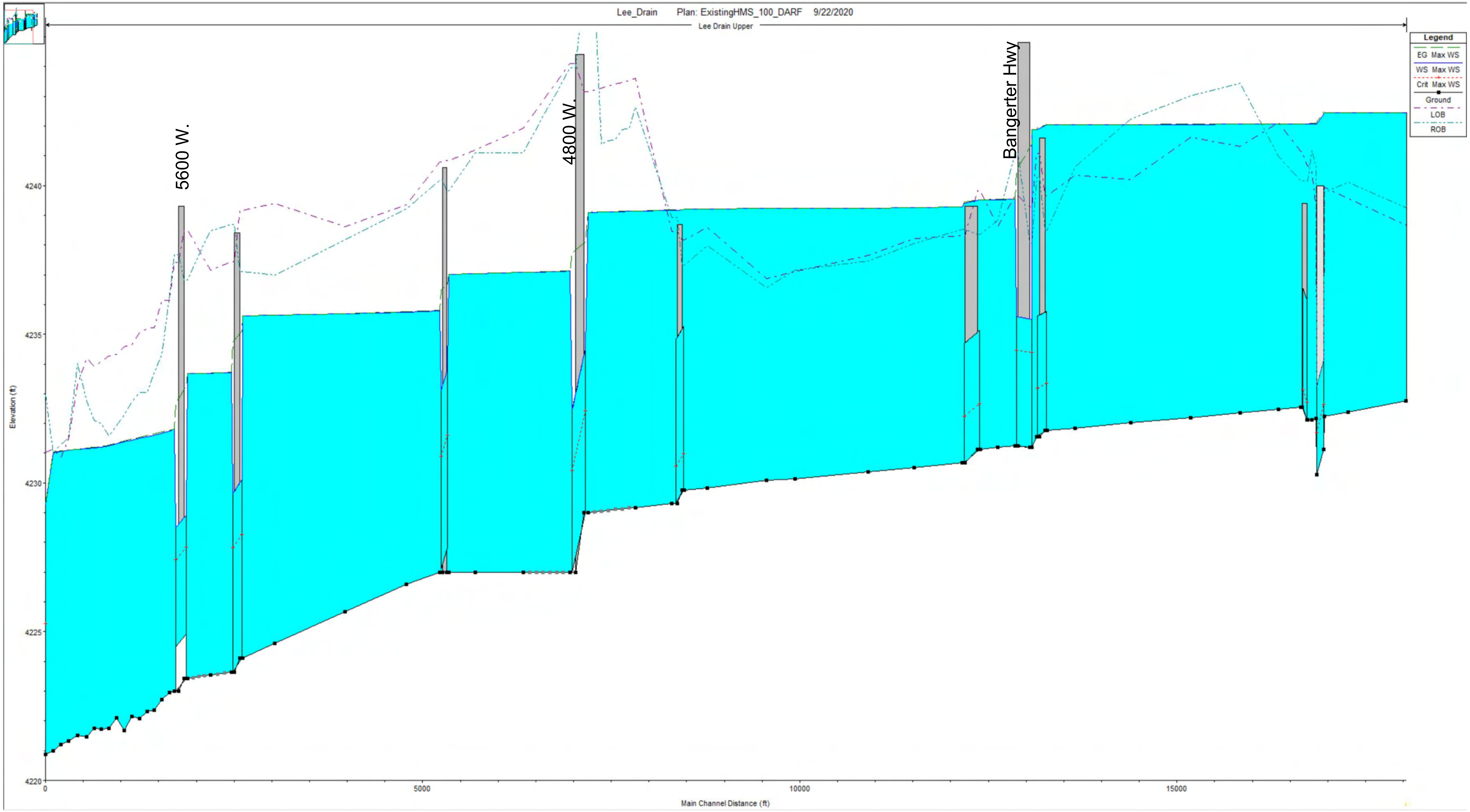


Figure IV-6. Lee Drain – Predicted Water Surface Profile with Existing Conditions and 100-year Event.

TABLE IV-3. Lee Drain Existing Crossing Capacity Deficiencies.

LOCATION	STA	EXISTING CAPACITY (CFS)	DESIGN FLOWRATE (100-YR) CFS	ADDITIONAL NEEDED CAPACITY (%)
Gladiola Street	18,600	45	73	62%
Brighton & North Point Canal	16,879	45	73	62%
Access Road	16,700	45	94	109%
Rail Road	13,200	62	190	206%
Bangerter HWY	12,900	66	190	188%
Gramercy Road	12,200	100	232	132%
Rail Road	8,400	64	266	316%
4800 West	7,000	105	310	195%
5070 West	5,300	127	351	176%
5500 West	2,500	200	460	130%
5600 West	1,800	215	460	114%

The capacities in Table IV-3 include the downstream backwater effects. The culverts and channel system do not have capacity to pass the 100-year design storm runoff event.

The Lee Creek conveyance between the C-7 Ditch and the Lee Kay Ponds is poorly defined and lacks capacity along the entire stretch. A master plan level design channel is provided in the "Preferred Solution" Section of this Chapter.

ALTERNATIVES CONSIDERED

A levee system was briefly considered and then rejected due to feasibility issues and maintenance concerns. For example, several of the road crossings would need to be raised and the existing channel right of way would need to be widened to accommodate the levees and levee maintenance access. Interior drainage systems would be affected as well.

Two conveyance improvement alternatives were considered for providing the 100-year design capacity: 1) Increase channel and culvert capacity while maintaining the existing channel profile; and 2) excavate a new channel profile providing a continuous channel slope with increased channel and culvert capacity.

1. Maintain existing channel profile and Increase channel and culvert capacity while maintaining the existing channel profile. This alternative was discarded due to maintenance concerns and large culvert widths. Existing overall channel gradients are very small (on the order of 0.0002 feet per foot) upstream from Bangerter Highway (about 4000 West). This very small slope would require replacing the existing 4-foot diameter culvert under the railroad (STA 8,400) with an opening on the order of 20 feet wide by 4 feet high.

2. Change channel profile. Excavate a new channel profile providing a continuous channel slope with increased channel capacity and culvert capacity sufficient to pass the design

storm runoff. The proposed new channel profile is shown on Figure IV-7. For comparison, the railroad culvert (STA 8,400) with this alternative would need to be an 8 feet wide by 5 feet deep box culvert.

PREFERRED SOLUTION

The preferred solution includes channel and culvert improvements. Channel improvements include excavating the channel to the profile shown on Figure IV-7 and assumes a resulting trapezoidal channel with 2 horizontal to 1 vertical side slopes and bottom widths as shown in Table IV-4. A typical cross section is shown on Figure IV-8. The proposed cross section stays within the existing channel right of way. Project cost estimates are provided in the Appendix.

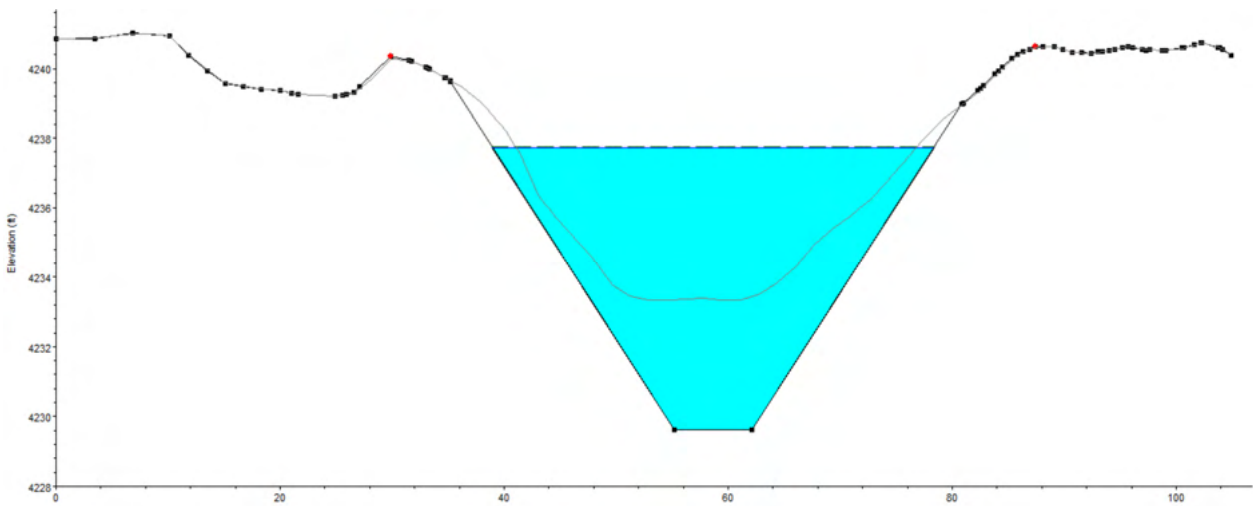


Figure IV-8. Typical Upper Lee Drain Master Plan Channel Cross Section.

TABLE IV-4 – Lee Drain Proposed Channel Bottom Widths and Estimated Excavation Quantities.

Up STA	Down STA	Bottom Width (ft)	CUT VOLUME (CY)
19,450	18,682	5	280
18,682	16,700	5	3,410
16,700	12,650	7	13,320
12,650	9,962	8	4,890
9,962	7,853	9	5,060
7,853	6,979	10	1,900
6,979	0	0	5,160
		TOTAL	34,020

Conceptual road crossing opening sizes are provided in Table IV-5. The Lee Drain Profile with proposed master plan improvements is shown on Figure IV-7.

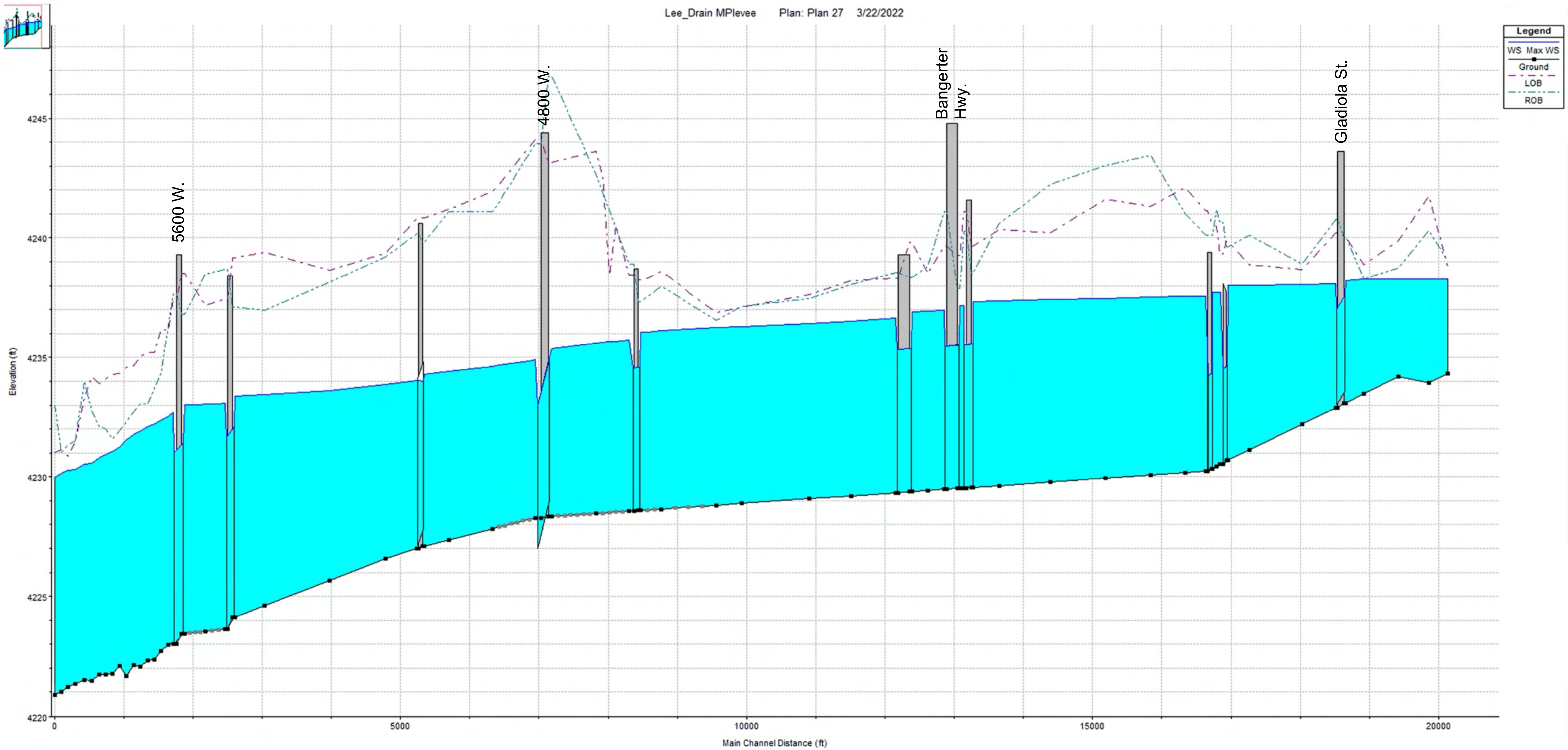


Figure IV-7 Lee Drain Profile with Master Planned Improvements

TABLE IV-5 – Lee Drain Conceptual Crossing Opening Sizes.

CROSSING	STA (feet)	Crossing Length (ft)	Span x Height (feet)
Gladiola Street	18,600	117	5 x 4
Brighton & North Point Canal	16,879	58	5 x 4
Access Road	16,700	58	5 x 4
Rail Road	13,200	120	8 x 5
Bangerter Highway	12,900	200	8 x 5
Gramercy Road	12,200	200	10 x 5
Rail Road	8,400	97	12 x 5
4800 West	7,000	170	12 x 5
5070 West	5,300	170	12 x 6
5500 West	2,500	170	16 x 6
5600 West	1,800	170	16 x 6

The preferred solution for Lee Creek downstream of the Lee Kay Ponds includes channel and culvert improvements as well as release restrictions from the ponds. Figure IV-9 shows the proposed alignment and identifies segments of the reach that have the same channel configuration. The releases from the ponds were restricted in the model by utilizing a 1-foot diameter low level outlet to restrict release from the ponds to 10 cfs.

The proposed channel is trapezoidal with the dimensions and slope for each of these segments shown on Figure IV-9. Channel improvements will require both cut and fill to develop the proposed design channel with the overall profile as shown in Figure IV-10. Figure IV-11 shows the minimum required ground elevations along the banks of the design channel to provide 1-foot of freeboard. An example design cross section is shown in Figure IV-12.

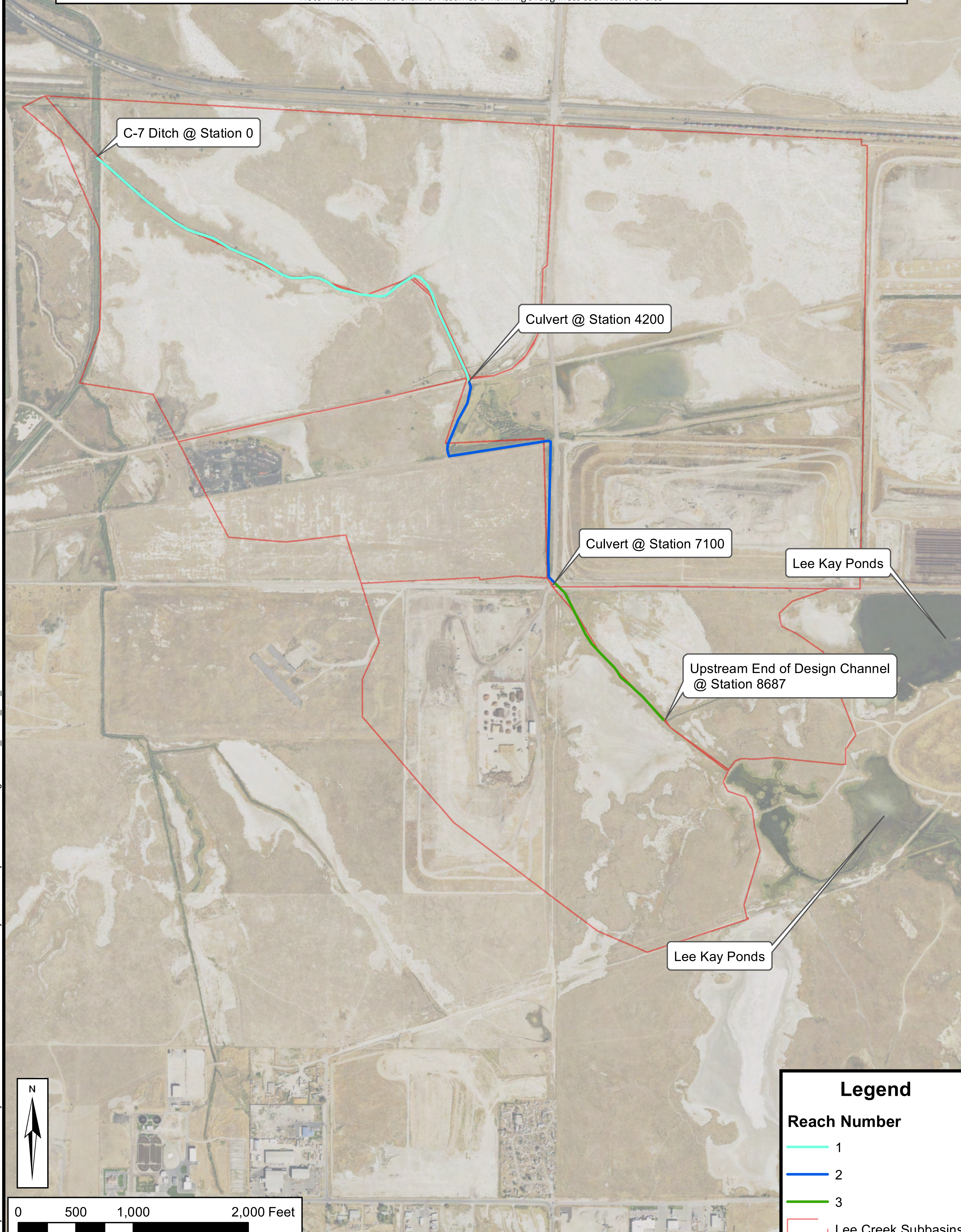
The channel was designed to reflect similar vegetation growth as is seen in other nearby flood control channels. This allows for a channel that will convey future 100-year flows and not rely on a smooth channel assumption that overtime becomes difficult to sustain because of the complications associated with maintaining vegetation growth along the channel banks.

Design flowrates and conceptual road crossing opening sizes are provided in Table IV-6 for Lee Creek downstream of the Lee Kay Ponds.

**TABLE IV-6. Lee Creek Conceptual Crossing Sizes
(Down Stream of Lee Kay Ponds).**

CROSSING	STA (feet)	Crossing Length (ft)	100-YR Design Flows (cfs)	Conceptual Culvert Size Span x Height (feet)
1300 S	7050	180	20	4 x 3
8000 W	4200	40	80	6 x 4

Start Station	End Station	Reach #	Bottom Width (ft)	Side Slope H:V	Slope (ft/ft)	Water Depth (ft)	Top Width (ft)	1-Foot Freeboard	Top Width (ft)	with Right-of-way	Peak Flow (cfs)
0	4178	1	10	3	0.00040	4.6	37.6	5.6	43.6	58.6	125
4249	6958	2	8	3	0.00070	3.8	30.8	4.8	36.8	51.8	80
7198	8587	3	6	3	0.00008	2.8	22.8	3.8	28.8	43.8	20
Note: Master Planned Channel Assumes a Manning's roughness coefficeint of 0.05											



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Lee Creek Downstream Lee Kay of Ponds

FIGURE IV-9

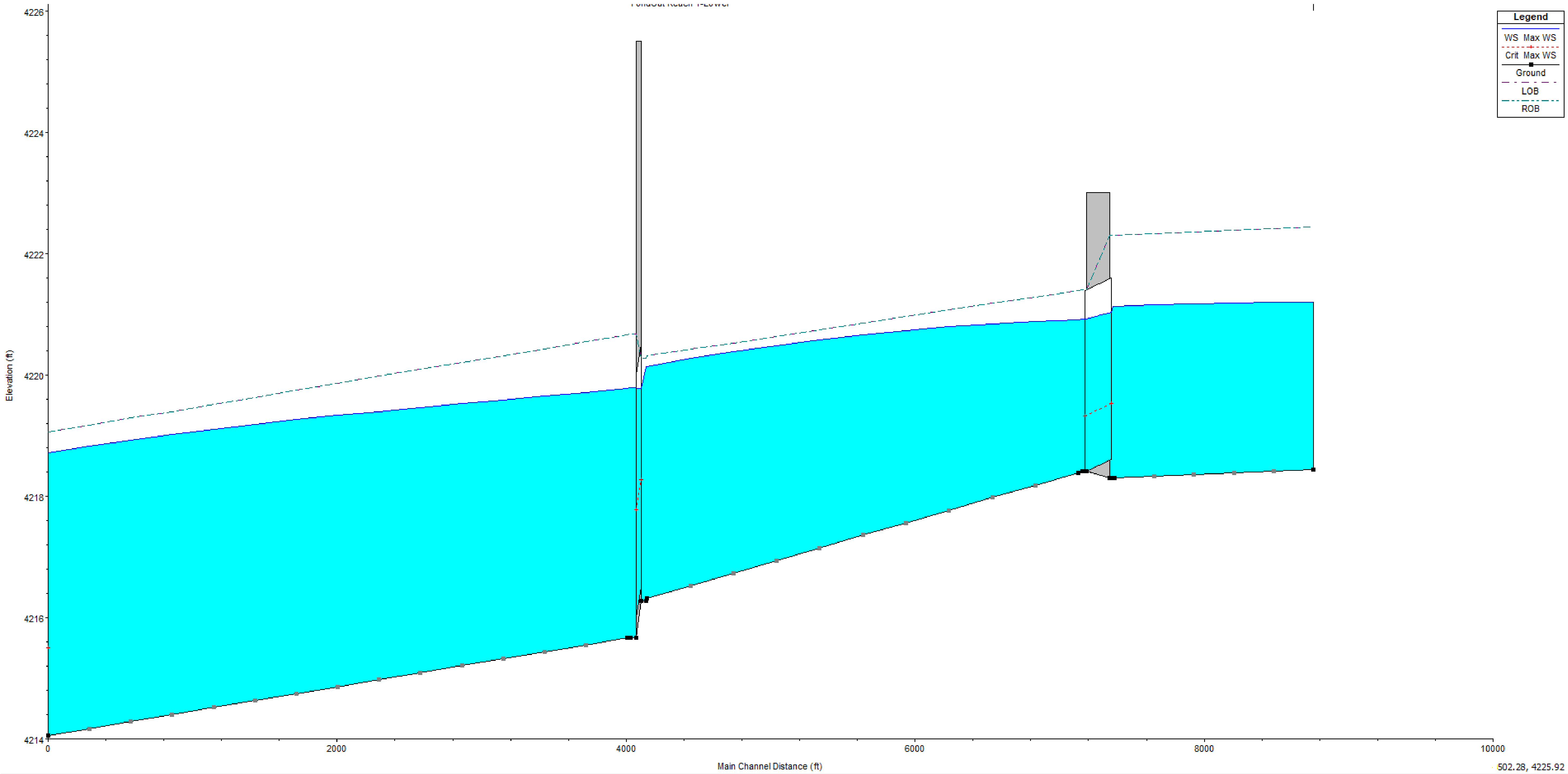


Figure IV-10. Lee Creek Design Channel Profile (C-7 to Lee Kay Ponds).

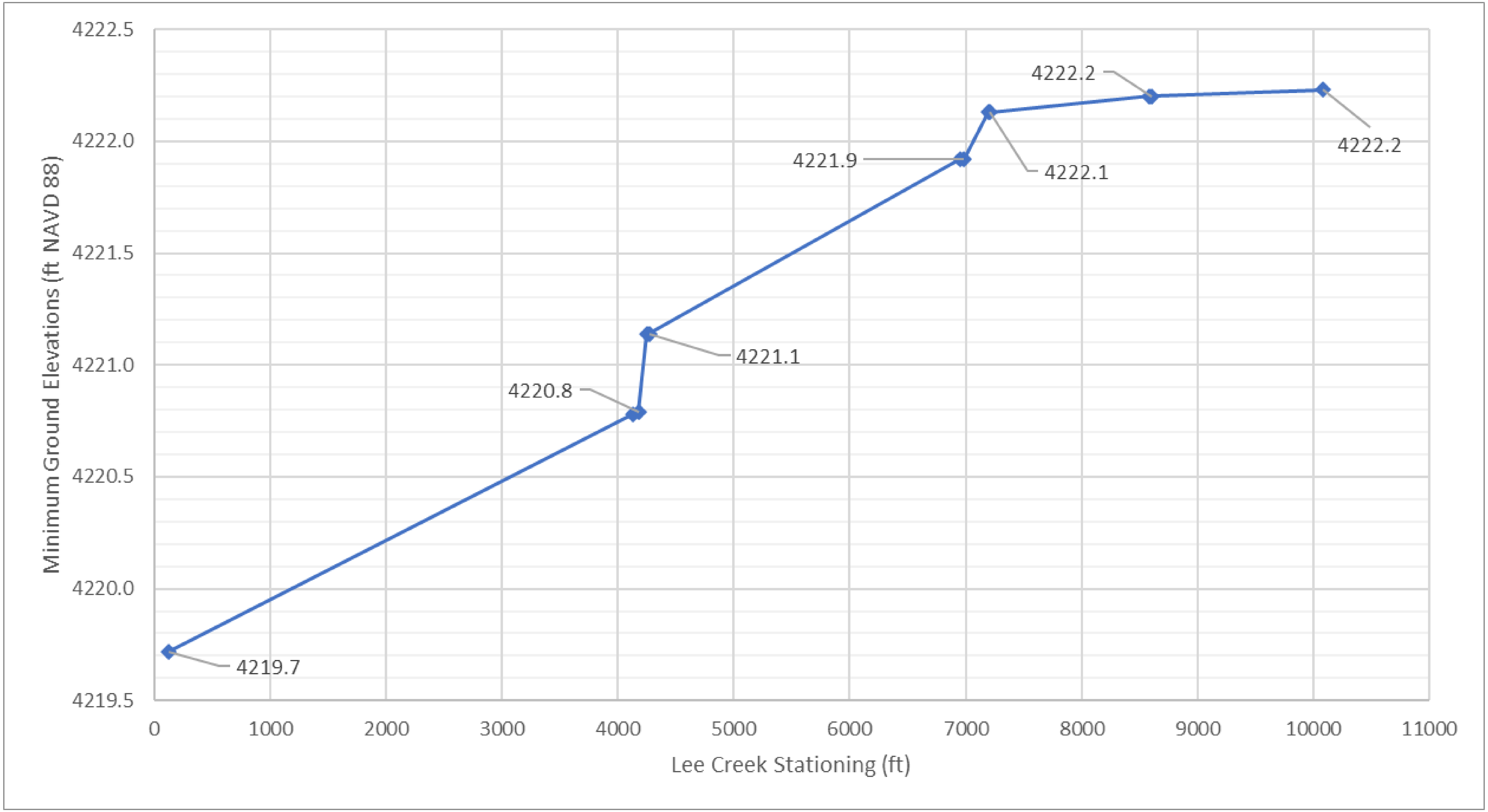


Figure IV-11. Lee Creek Design Channel Minimum Ground Elevations (C-7 to Lee Kay Ponds).

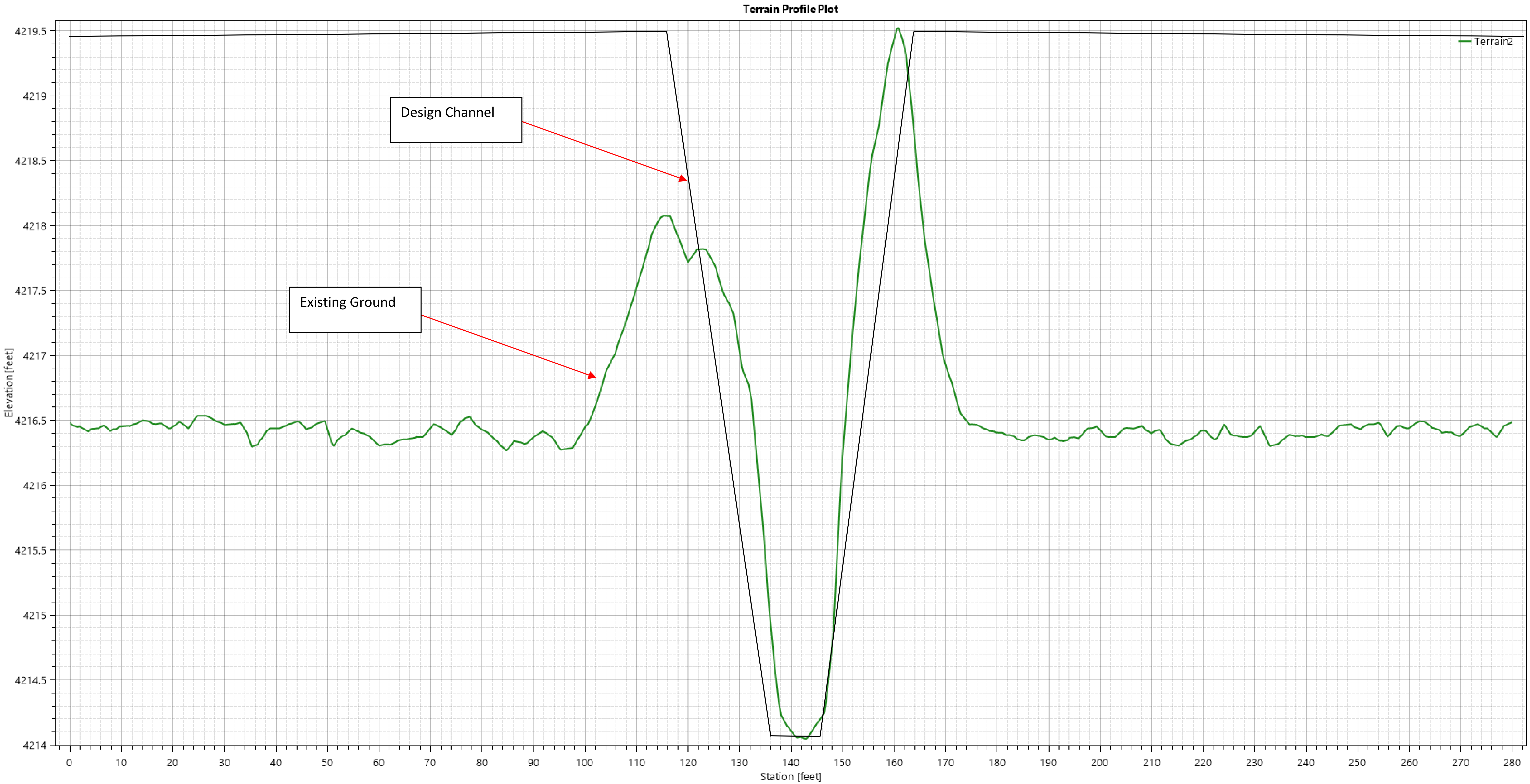


Figure IV-12. Typical Lee Creek Master Plan Channel Cross Section (Near C-7 Confluence).

CHAPTER V

CHAPTER V – UTAH & SALT LAKE CANAL

EXISTING SYSTEM DESCRIPTION

The Utah and Salt Lake Canal (USLC) begins at Turner dam and extends to the Kennecott property just west of Magna. The canal continues on Kennecott property to the "diving board" which leads to the Utah & Salt Lake Canal Extension (a Kennecott maintained facility). The Utah & Salt Lake Extension discharges to the C-7 Ditch. The portion of the USLC included in the Northwest Canal and Creek Study (NWCC Study) extends from Bangerter Highway (at about 4900 South) through the USLC Extension to the C-7 Ditch (see Figure V-1). The USLC upstream of the Bangerter Highway crossing is included in the study area for the Southwest Canal and Creek Study.

PREDICTED 10-YR AND 100-YR FLOWS

The area that is tributary to the USLC for the extents of the NWCCS was delineated to determine incremental hydrographs at discrete locations along the creek. A map of the subbasins tributary to the USLC and the Extension as well as their respective alignments for the extents of this study are shown in Figure V-1. A key assumption in the analysis is that the runoff from Little Valley will be detained to 15 cfs in a 3-hr 100-year storm runoff event consistent with current development planning. The flows and detention requirements shown on Figure V-1 are based on the 3-hr 100-year storm event.

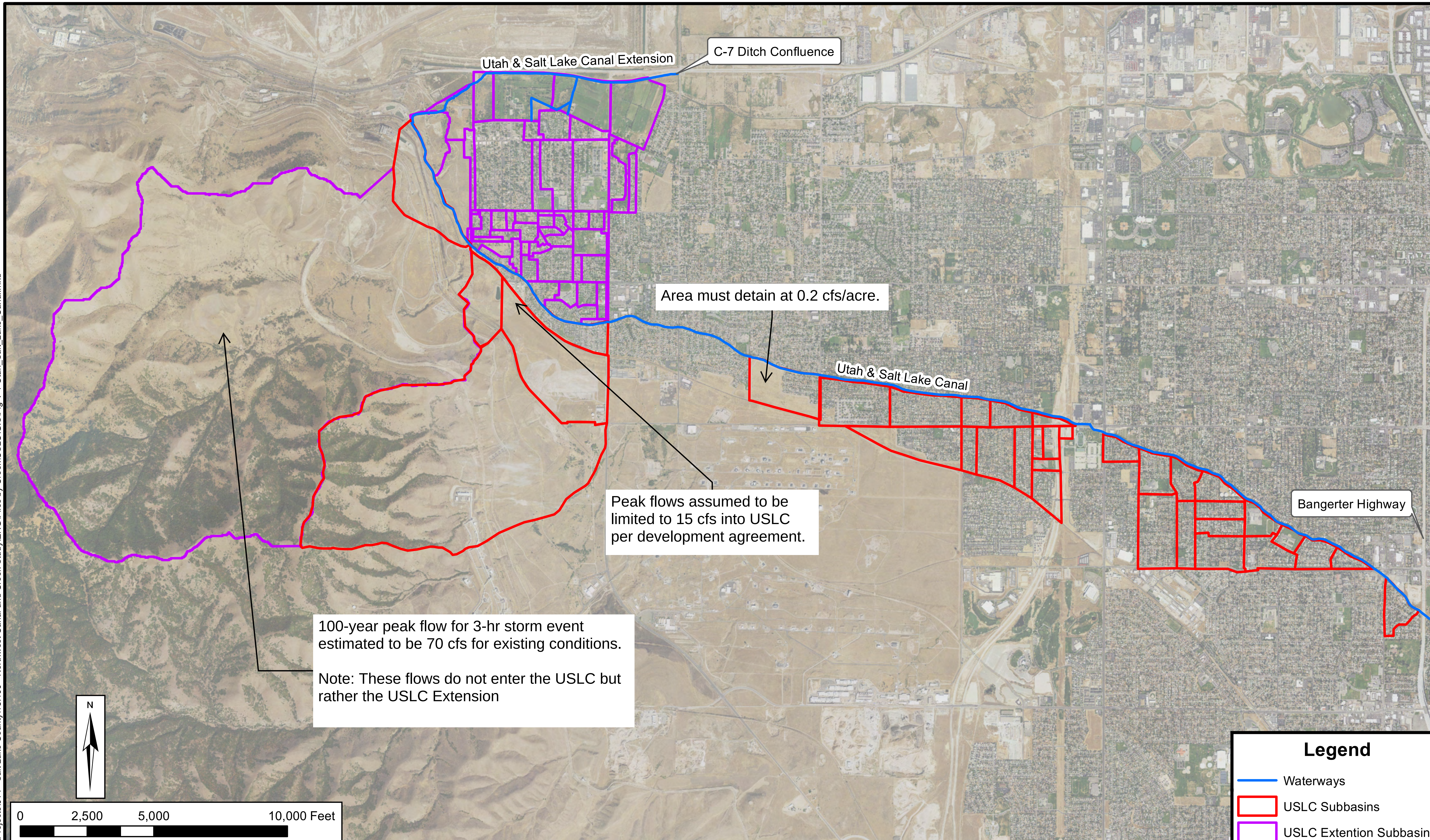
Design storm considerations were given to the elongated and discontinuous nature of the areas that are tributary to the USLC. The approach for this study was to apply a Depth Area Reduction Factor (DARF) based on the area of a hypothetical storm shape covering the areas tributary to the USLC simultaneously. Figure V-2 shows a hypothetical storm centered over the USLC drainage area. The required hypothetical storm size to cover the drainage areas for the extents of the NWCCS was approximately 30 square miles. Based on the SLC DARF curves the appropriate DARF factor for this area would be 0.78.

The flows for each subbasin were predicted utilizing the techniques described in Chapter II. The resulting hydrographs were inserted into the hydraulic model at the relative location they would expect to enter the canal. A baseflow of 30 cfs was utilized in the model to account for irrigation flows per direction from the County. The flows were then routed through the canal to predict anticipated peak flowrates and stages in the canal. Figure V-3 shows how the peak flow varies over the extent of the USLC starting at the beginning of the "diving board" upstream to approximately Bangerter Highway.

EXISTING SYSTEM CAPACITY

In evaluating capacity along the USLC it was necessary to split the canal into two general segments.

The first segment is from the upstream end of the NWCCS project extents (just west of Bangerter Highway at about 4900 South) to the headworks of the "diving board". The downstream boundary condition of the hydraulic model for this segment was a rating curve that was developed to simulate the stage-flow conditions at the diving board. This rating curve is shown in Figure V-4. The maximum anticipated flow through the "diving board" structure with six inches of freeboard is



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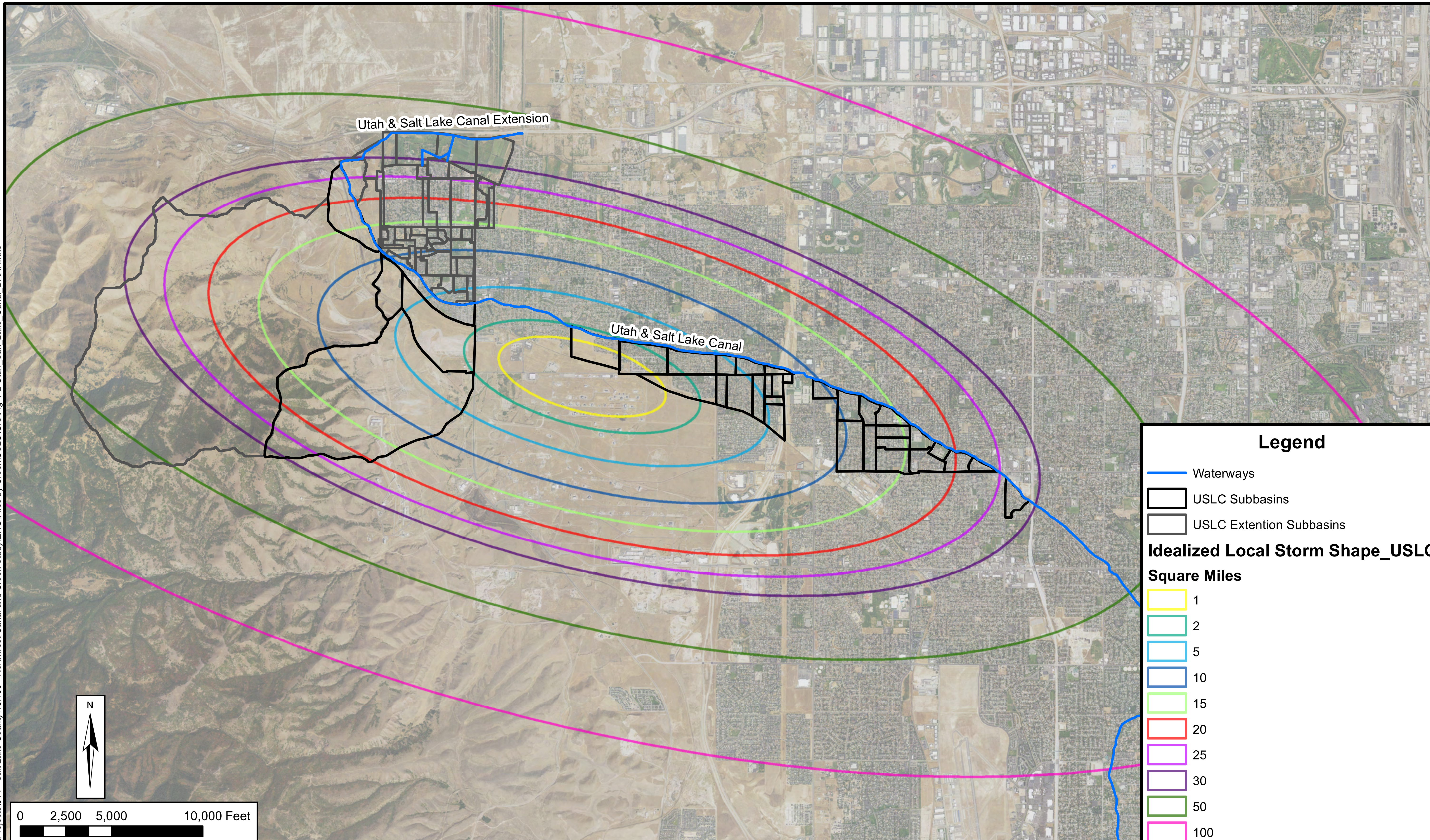
Utah and Salt Lake Canal and Extension
Alignment and Subbasins

FIGURE
V-1



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Utah and Salt Lake Canal and Extension
Idealized Storm Coverage

FIGURE V-2

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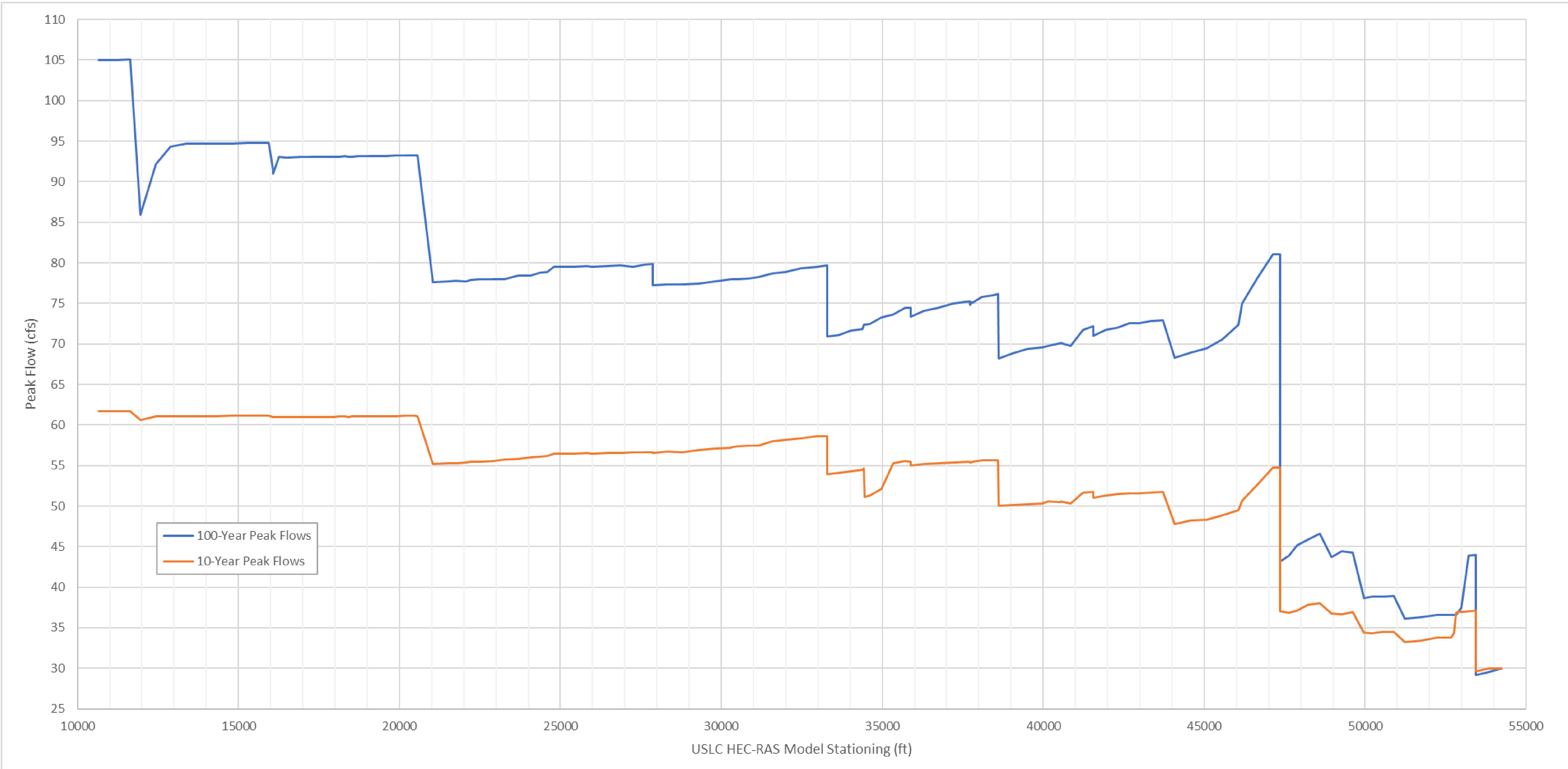

Figure V-3. 10-Year and 100-year Flow Rates for Utah and Salt Lake Canal (Diving Board to Bangerter Hwy).

approximately 140 cfs. The upstream boundary condition is a constant flow hydrograph of 30 cfs to simulate irrigation flows.

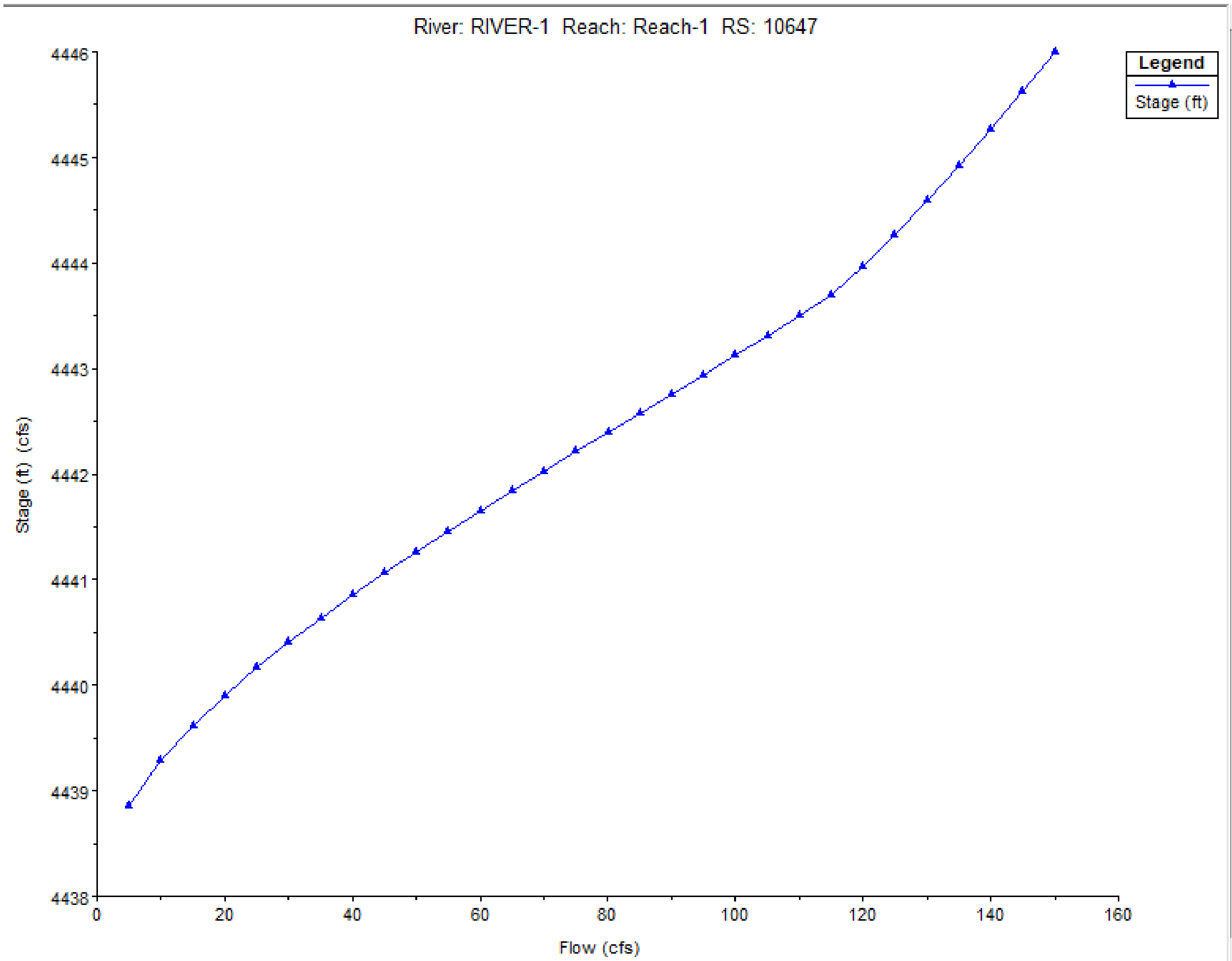

Figure V-4. Downstream Boundary Condition Rating Curve for Segment 1.

The second segment (which is also known as the USLC Extension) is from the downstream end of the diving board to the confluence with the C-7. The downstream boundary condition of the hydraulic model for this segment was assumed to be normal depth with a slope of 0.002 ft/ft. The upstream boundary condition was the outflow hydrograph from the upstream segment.

Table V-1 provides an estimated freeboard for all of Segment 1 by comparing the computed surface elevations of the hydraulic model (both the 10-year and 100-year) against the lowest bank elevation of the cross sections. The results are summarized by 5,000-foot segments of Segment 1.

TABLE V-1. USLC Freeboard Summary for Segment 1.

HEC-RAS Stationing		10-Year Freeboard (ft)		100-Year Freeboard (ft)	
Start Station	End Station	Min	Ave	Min	Ave
54233	50000	3.2	4.0	2.9	3.8
49999	45000	2.7	3.4	2.2	2.9
44999	40000	2.8	3.6	2.4	3.2
39999	35000	2.5	3.2	2.1	2.8
34999	30000	2.1	2.6	1.6	2.1
29999	25000	1.5	2.3	1.1	1.9
24999	20000	1.7	3.1	1.1	2.6
19999	15000	1.6	3.3	1.1	2.8
14999	10647	2.0	3.1	1.1	2.4
Overall		1.5	3.2	1.1	2.8

Segment 2 which is also known as the USLC Extension was modeled using 1D for the channel and 2D for the overbanks. The floodplain extents for the 10-year and 100-year runs are shown in Figure V-5 and Figure V-6 respectively.

DEFICIENCIES

Segment 1 was found to have sufficient capacity for the 10-year and 100-year runoff events.

Segment 2 (USLC Extension) included the following deficiencies.

- Existing 36-inch culvert at 9180 West shows flow going over the top of the road for the 100-year event.
- Minor flooding along the golf course was revealed for the 100-year event and 10-year events.

USLC EXTENSION

No deficiencies were found in the USLC. The USLC Extension is not a County flood control facility. The culvert deficiency could be addressed by adding another 3-foot diameter culvert next to the existing one or replacing the 36-inch culvert with a 48-inch culvert.

Since the USLC Extension is not a County flood control facility we are not providing recommended improvements, but rather showing the inundation extents which occur on the golf course. The flooding does not threaten existing homes. The extents of the flooding from the 10-year and 100-year events are shown in Figure V-5 and Figure V-6 respectively.

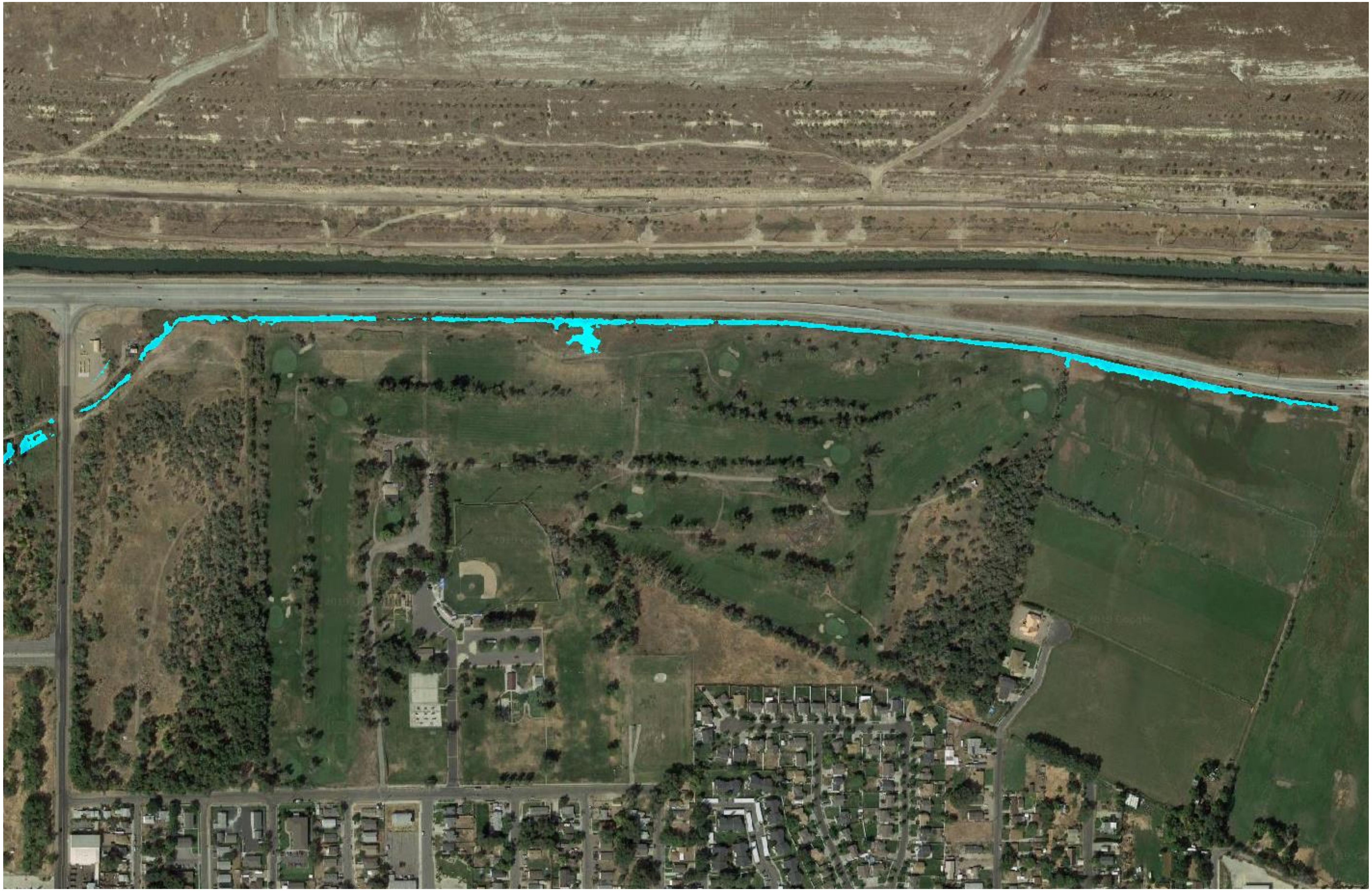

Figure V-5. 10-Year Flooding Extents along USLC Extension Near Golf Course.

Figure V-6. 100-Year Flooding Extents along USLC Extension Near Golf Course.

CHAPTER VI

CHAPTER VI – COON CANYON CREEK AND HARKER'S CANYON CREEK

EXISTING SYSTEM DESCRIPTION

The storm flows originating in Coon and Harker's Canyons are conveyed through the natural creek systems until the flow enters the urban area of Magna where it is conveyed through storm drainage pipelines to the C-7 Canal. The existing storm drainage system is shown on Figure VI-1.

PREDICTED 10-YR AND 100-YR FLOWS

Hansen, Allen and Luce performed a storm drain master plan in this area in 2008[8]. Additionally, the County retained HAL to design additional storm drainage capacity from 3500 South to 3100 South based on findings from the Master Plan in 2015. The 2008 master plan models were refined and the new infrastructure was designed and constructed to convey the 100-year detained mountain flows and the 10-year storm drainage flows in the urban portion of the tributary area. Before entering the storm drain system in Magna the runoff from both Coon Creek and Harker's Creek pass through the 4100 South Coon Harker Detention Basin (also known as the ATK Detention Basin) which has a detention volume of approximately 90 acre-feet. Annual maintenance of this facility is recommended which would include mowing of weeds and an inspection of the outlet works to verify proper function. This detention facility reduces 100-year runoff to approximately 5 cfs (providing critical flood control to Magna). This flow was assumed to be a constant source and a 10-year event was applied to the remaining urban area that was tributary to the network that discharges to the C-7 Ditch.

The previous modeling effort was used to generate the design flows for the outflow to the C-7 Ditch. The design peak outflow to the C-7 ditch from the Coon and Harker drainage area is approximately 190 cfs.

COON HARKER NORTH OF 3100 SOUTH

The master plan for Coon Canyon Creek and Harker's Canyon Creek north of 3100 south includes increasing the capacity of the storm drain in 80th West by adding a 36-inch diameter storm drain parallel to the existing storm drain to provide a total design capacity of 190 cfs. A cost estimate is provided in the Appendix.

COON HARKER DETENTION BASIN 4100 SOUTH (aka. ATK Detention Basin)

The 4100 South Coon Harker Detention Basin is critical to providing flood control for Magna. This facility should be maintained on a regular basis including periodic inspection of the outlet works and vegetation control. We recommend that inspection and maintenance operations be conducted at least twice a year: 1) after the snow melt runoff in the spring and 2) in the fall.

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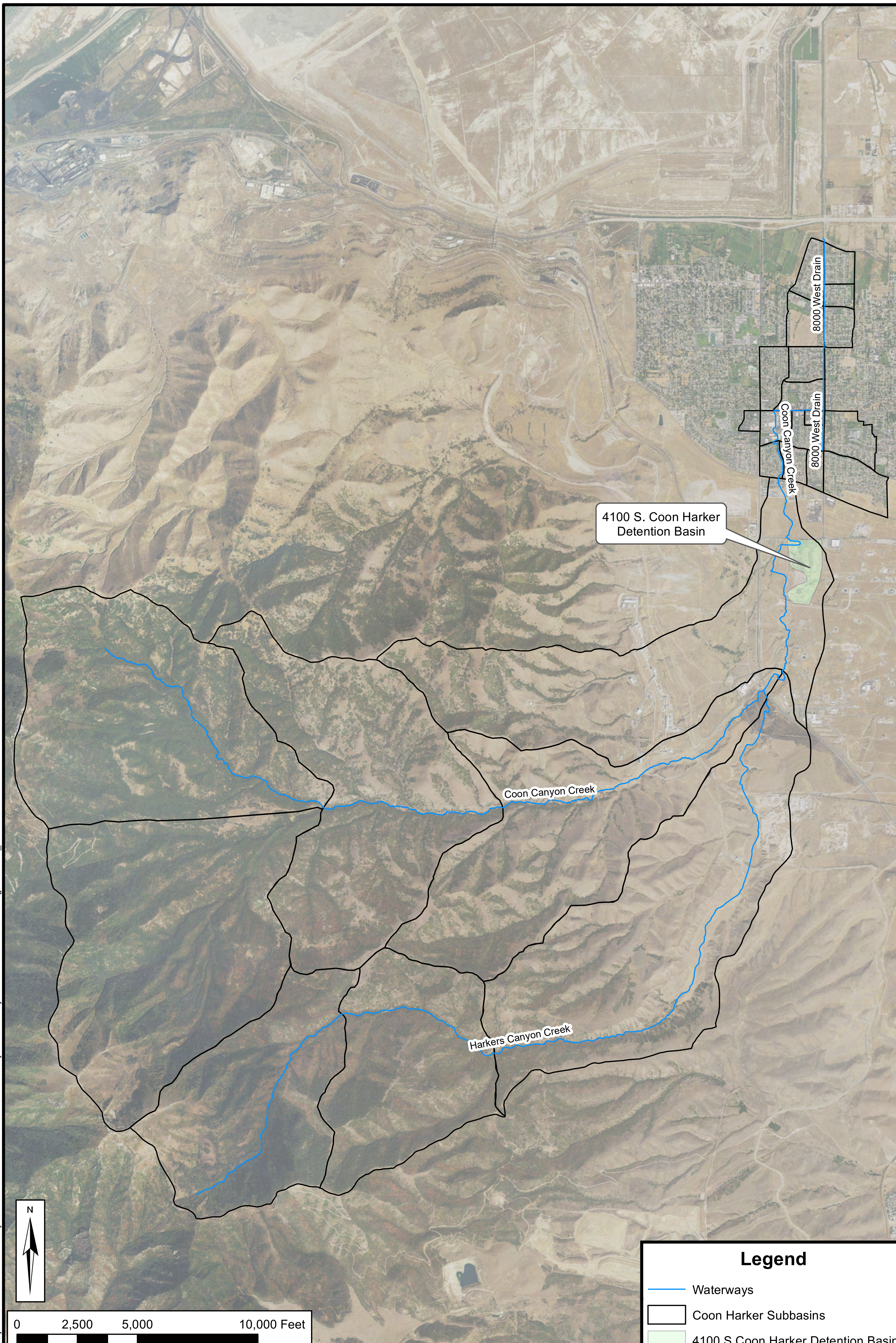
[8] Hansen, Allen, & Luce (2008). Coon/Harkers Creek Storm Drainage Master Plan, Salt Lake County, Utah.



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Coon and Harker Creek Alignments and Drainage Areas

FIGURE VI-1

COON CANYON CREEK AND HARKER'S CANYON CREEK MASTER PLAN SUMMARY

Coon Canyon and Harker Canyon subbasins as delineated in the 2008 master plan are shown on Figure VI-2. Future developments within the Coon/Harker's canyons are to detain storm runoff flows to pre-development conditions for the 2-year, 10-year, and 100-year runoff events. Coon/Harker's canyon subbasin characteristics for mountain areas are summarized on Table VI-1 along with the predicted 100-year 6-hour peak storm runoff flowrates.

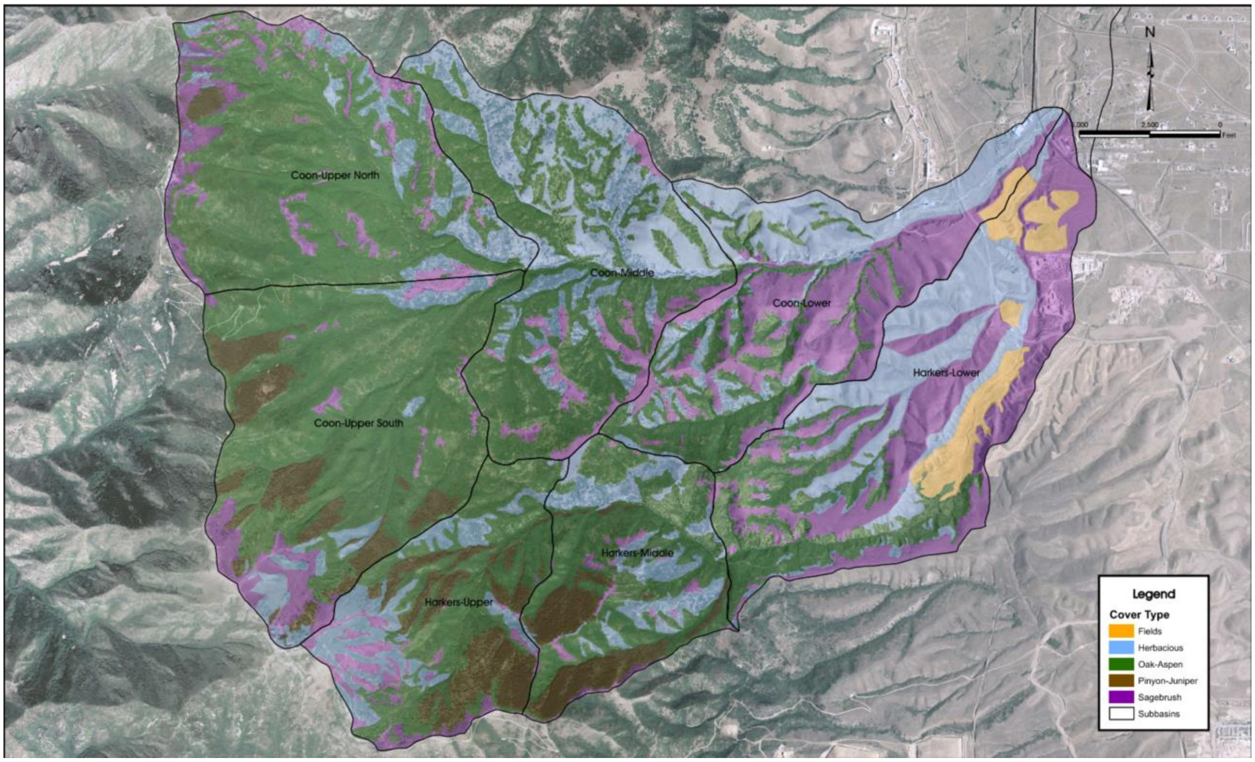


Figure VI-2. Coon Canyon Creek and Harker's Canyon Creek Subbasins

TABLE VI-1. Coon Canyon Creek and Harker's Canyon Creek Subbasin Characteristics for Mountain Areas – Existing Conditions

Subbasin ID	Area (acres)	Area Weighted CN	Lag Time (hr)	100-yr 6-hr Peak Storm Runoff Flow (cfs)
Coon-Lower	1,769	58.2	1.27	24
Coon-Middle	1,979	57.3	1.68	32
Coon-Upper North	2,093	45.5	1.82	0
Coon-Upper South	2,295	44.7	1.82	0
Harkers-Lower	2,243	67.2	1.30	137
Harkers-Middle	1,169	51.5	1.54	3
Harkers-Upper	1,288	45.2	1.60	0
Coon/Harkers Creek Confluence	383	63	0.48	184
Total:	13,219			

Snowmelt flowrates estimated using regional regression equations are summarized in Table VI-2.

TABLE VI-2. Estimated Snowmelt Flow Rates

Location	Predicted Snowmelt Flow Rates (cfs)		
	10 year	50 year	100 year
Coon/Harkers Creek Confluence	110	152	164

CHAPTER VII

CHAPTER VII – KERSEY CREEK

EXISTING SYSTEM DESCRIPTION

Kersey Creek begins at a culvert crossing of Highway 201 (approximately 7500 West). It flows generally to the north through a few other crossings until it discharges into the C-7 Ditch. The proposed alignment of Kersey Creek and its tributary area is shown in blue on Figure VII-1.

The existing Kersey Creek conveyance is poorly defined and has limited capacity. It was determined in consultation with the County that for the purposes of this study a master planning effort would be conducted to provide a design channel with sufficient capacity to convey future 100-year flows.

PREDICTED 10-YR AND 100-YR FLOWS

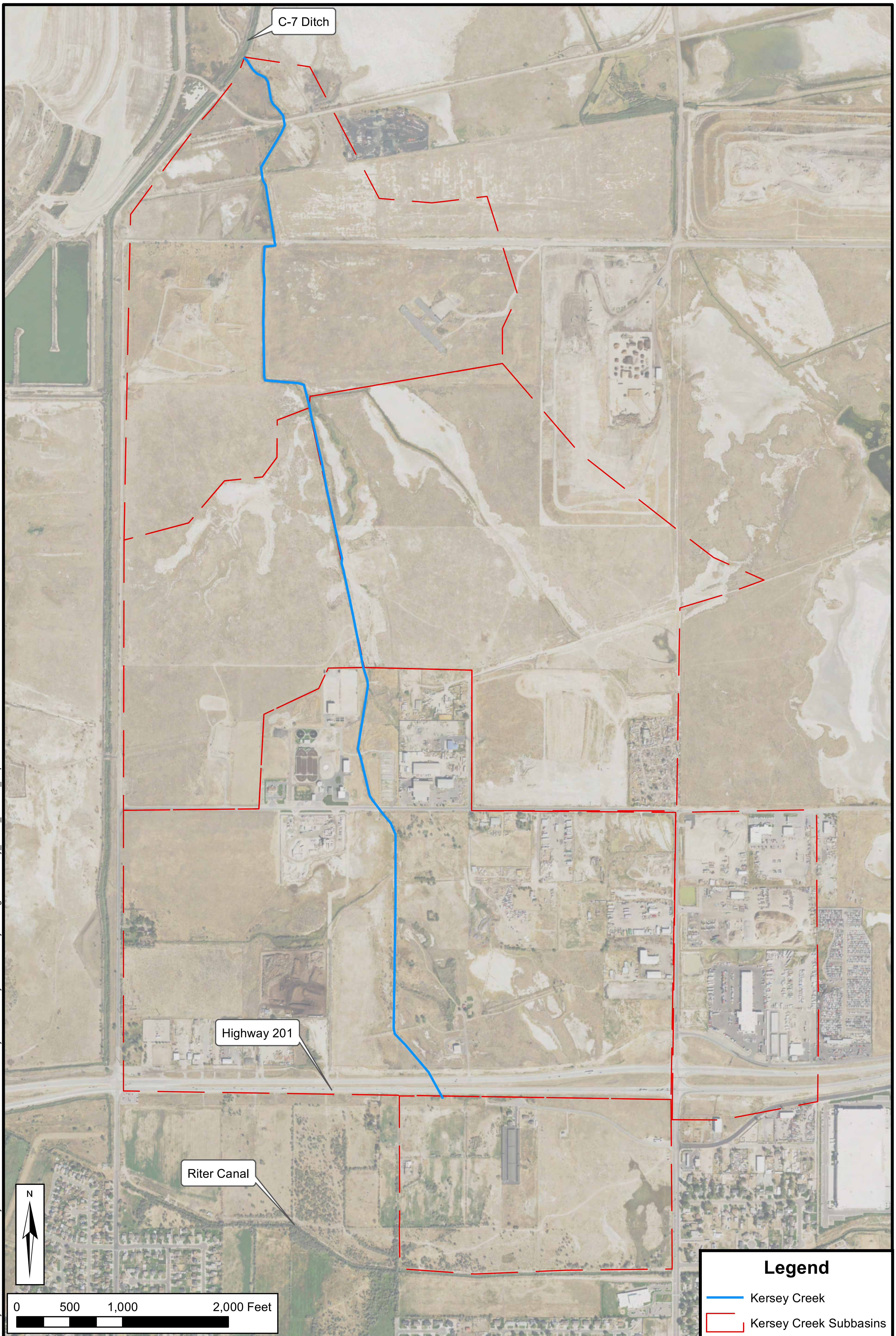
The area that is tributary to Kersey Creek was delineated to allow for the calculation of incremental hydrographs at discrete locations along the creek. These hydrographs were used to determine the required capacity for the Kersey Creek design channel. A map of the subbasins tributary to Kersey Creek are shown in Figure VII-1.

Future planning information was obtained from the County to assist in estimating future land cover types. It was estimated that these future developments would detain stormwater runoff to 0.2 cfs/acre in a 100-year storm runoff event. The flows were then added to the hydraulic model and routed all the way to the C-7 Ditch.

KERSEY CREEK MASTER PLAN

The preferred solution for Kersey Creek includes channel and culvert improvements. Figure VII-2 shows the proposed alignment and identifies segments of the reach that will have the same channel configuration.

The proposed channel is trapezoidal with dimensions and slope for each segment shown on Figure VII-2. Channel improvements will require both cut and fill to produce the required channel dimensions. An example of the design cross section at approximately Station 950 is shown in Figure VII-3. The profile of the design channel is shown in Figure VII-4. Figure VII-5 shows the minimum ground fill elevations around the banks of the design channel.



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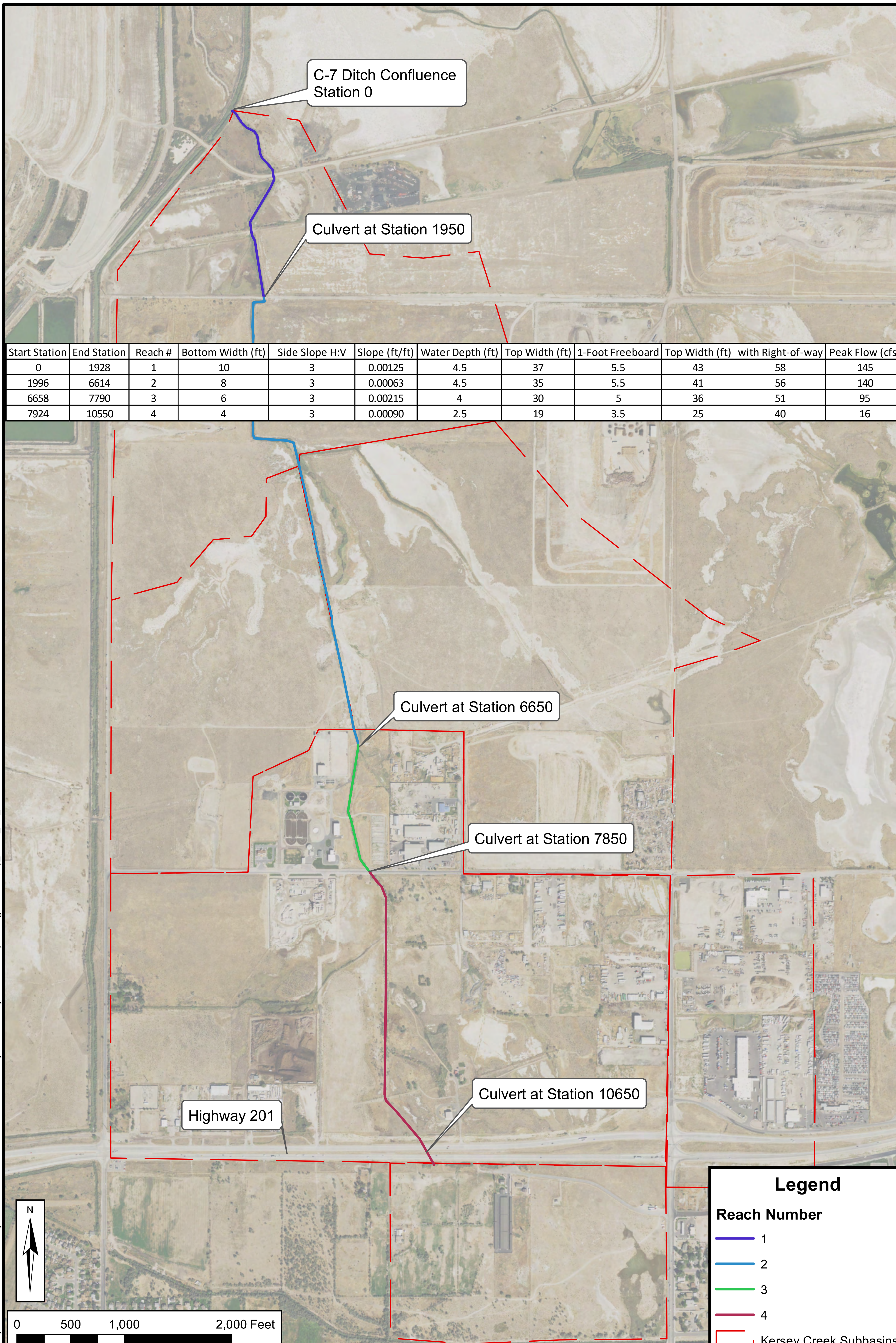
Flood Control

Kersey Creek
Alignment and Tributary Area

FIGURE
VII-1



Start Station	End Station	Reach #	Bottom Width (ft)	Side Slope H:V	Slope (ft/ft)	Water Depth (ft)	Top Width (ft)	1-Foot Freeboard	Top Width (ft)	with Right-of-way	Peak Flow (cfs)
0	1928	1	10	3	0.00125	4.5	37	5.5	43	58	145
1996	6614	2	8	3	0.00063	4.5	35	5.5	41	56	140
6658	7790	3	6	3	0.00215	4	30	5	36	51	95
7924	10550	4	4	3	0.00090	2.5	19	3.5	25	40	16

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Kersey Creek Master Plan Channel

FIGURE VII-2

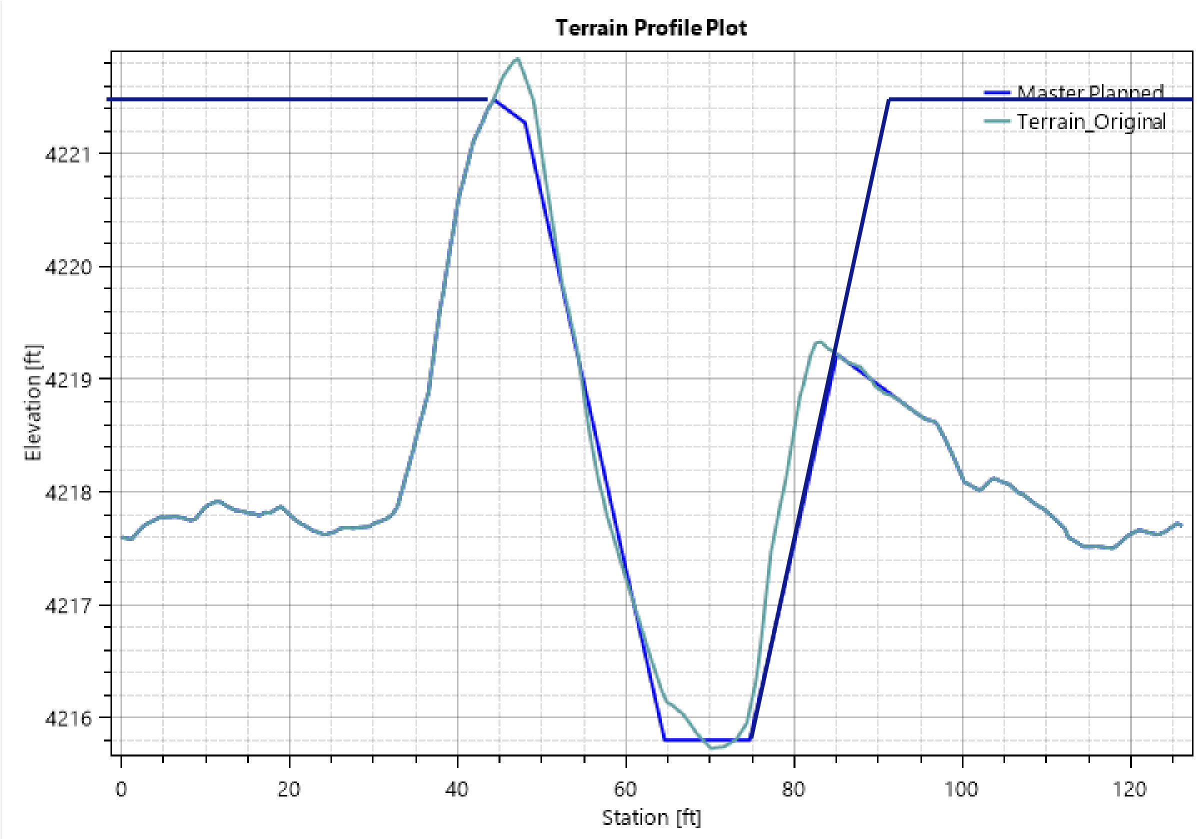


Figure VII-3. Kersey Creek Example Design Cross Section at Station 950.

The channel was designed to reflect similar vegetation growth as is seen in other nearby flood control channels. This allows for a channel that will convey future 100-year flows and not rely on a smooth channel assumption that overtime becomes challenging because of the difficulties associated with maintaining vegetation growth along the channel banks.

Proposed road crossings design flowrates and conceptual opening sizes are provided in Table VII-1.

TABLE VII-1. Kersey Creek Conceptual Crossing Opening Sizes.

CROSSING	STA (feet)	Crossing Length (ft)	Design Flow Rate (cfs)	Conceptual Culvert Size Span x Height (feet)
Highway 201	10,600	200	16	3 Dia. (Circular)
Unnamed	10,040	40	16	3 Dia. (Circular)
2100 S	7,800	110	19	3 Dia. (Circular)
Unnamed	7,310	20	95	6 x 4
Unnamed	6,630	20	95	6 x 4
1300 S	1,950	40	145	8 x 4
Unnamed	1,340	30	145	8 x 4
8000 W	740	60	145	8 x 4

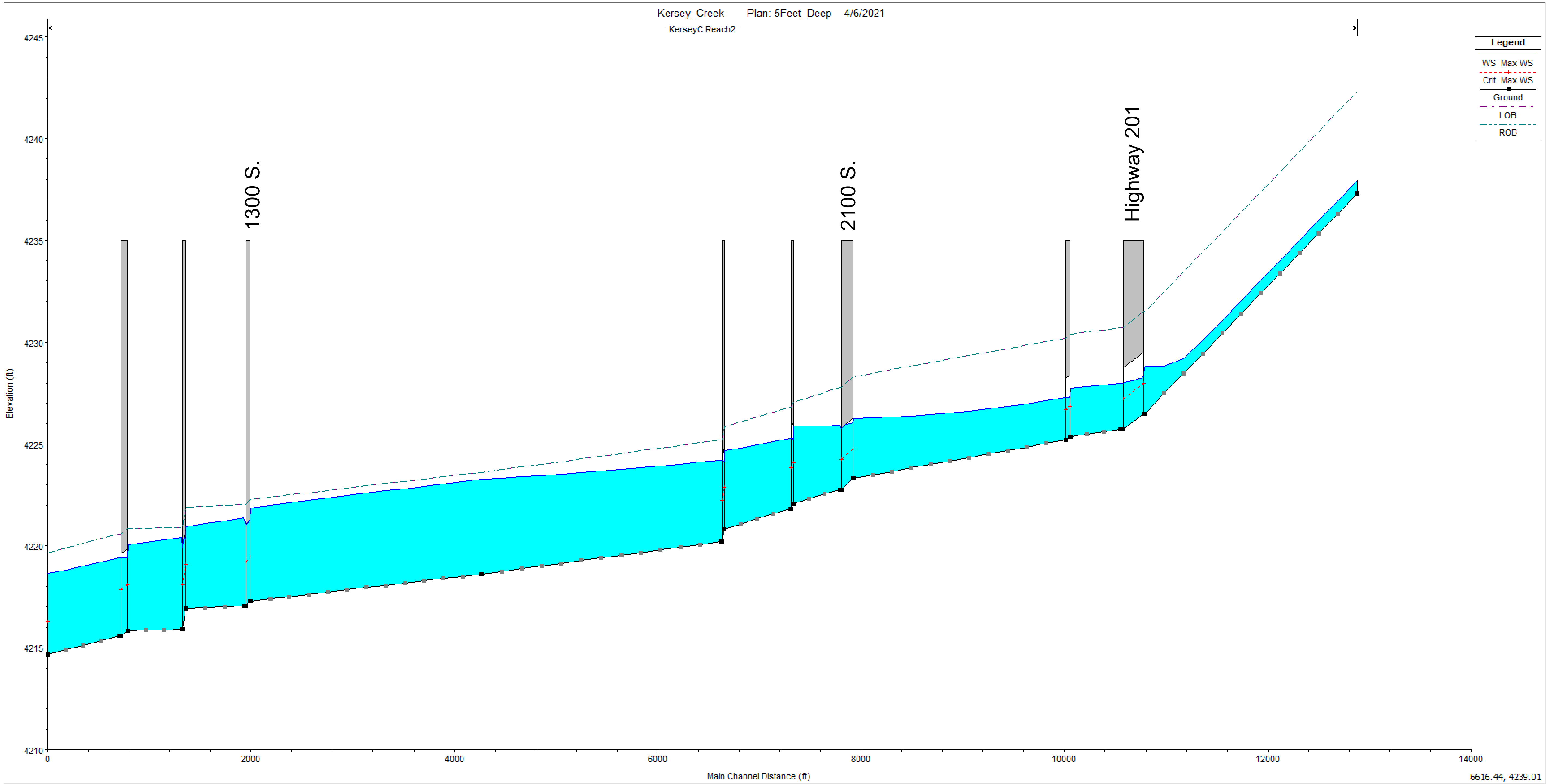


Figure VII-4. Kersey Creek Design Channel Profile.

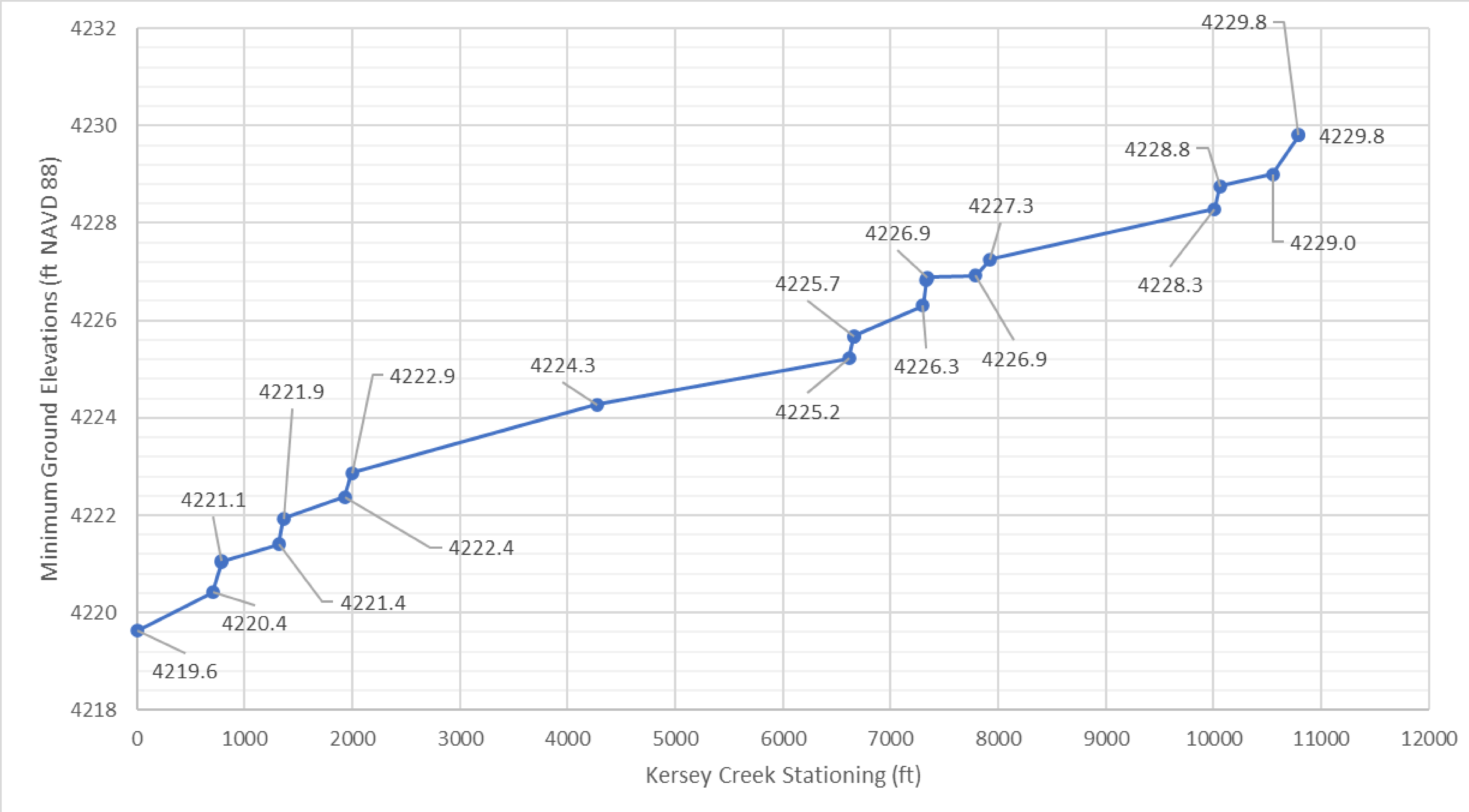


Figure VII-5. Kersey Creek Design Minimum Ground Fill Elevations.

CHAPTER VIII

CHAPTER VIII – RITER CANAL

EXISTING SYSTEM DESCRIPTION

A significant portion of the storm drainage from West Valley City discharges into the ponds that surround Stonebridge Golf Course. These ponds are the headwaters of the Riter Canal. There is a control structure just east of 5370 West that can be used to control releases from these ponds. The Riter Canal flows west approximately 3.5 miles from the control structure to the confluence with C-7 Ditch. Figure VIII-1 shows the alignment of the Riter Canal and its tributary area.

The Riter Canal includes a detention basin at approximately 6700 West on the left side (south side) of the channel. If flow exceeds the capacity of the gate opening, water is backed up into the detention pond. The location of this detention facility is shown in Figure VIII-1. Additionally, there is some significant natural floodplain storage at 6200 West which has also been included in our analysis.

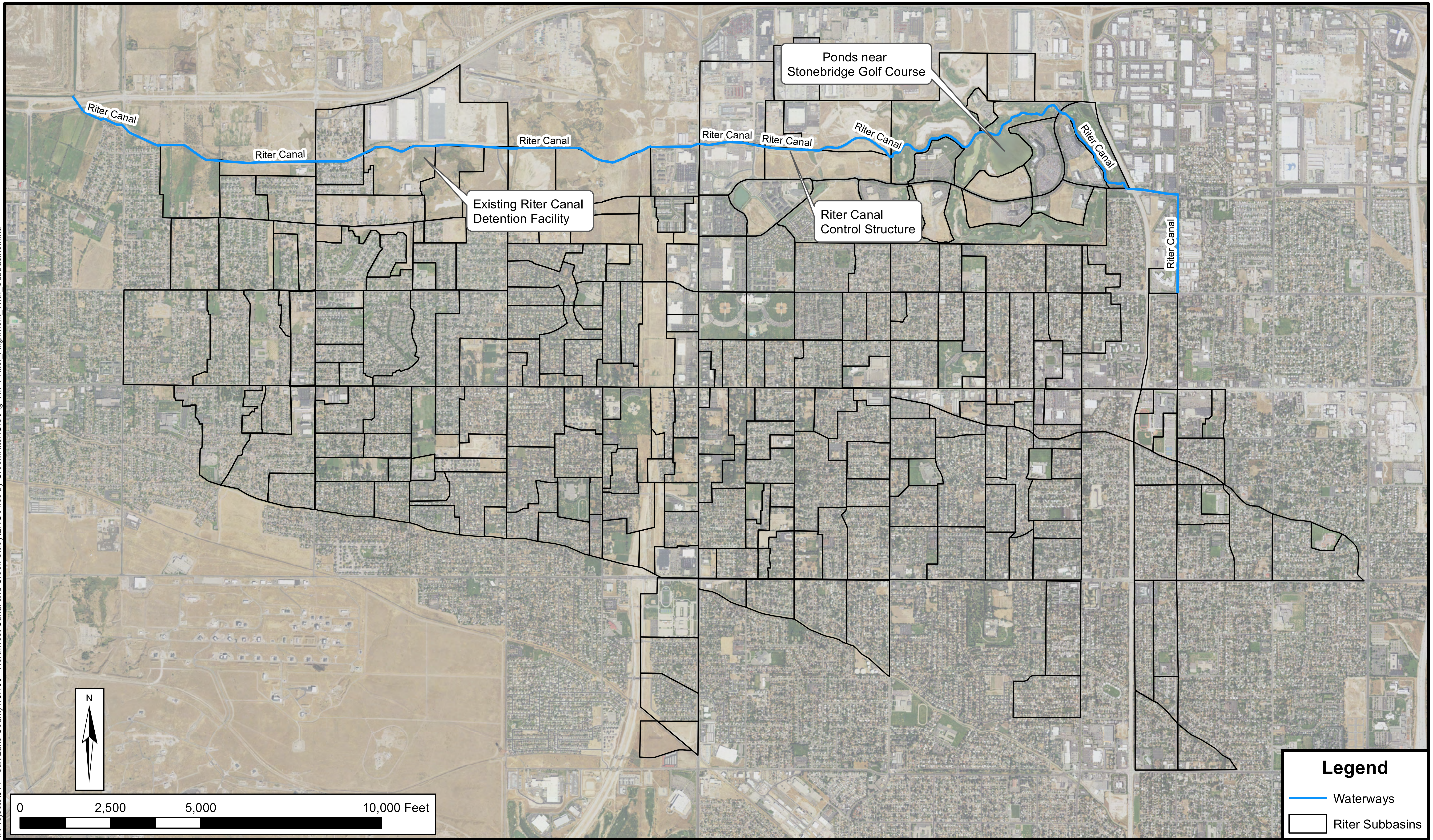
PREDICTED 10-YR AND 100-YR FLOWS

The area that is tributary to the Riter Canal was delineated by others. A combination of models provided by West Valley City and Magna were used to define the total drainage area and subbasin characteristics for the subbasins tributary to the Riter Canal. A map of the subbasins tributary to the Riter Canal is shown in Figure VIII-1.

Design storm considerations were given to the areas that are tributary to the Riter Canal. The approach for this study was to apply a Depth Area Reduction Factor (DARF) based on the area of a hypothetical storm shape covering the areas tributary to the Riter Canal simultaneously. Figure VIII-2 shows a hypothetical storm centered over the Riter Canal drainage area. The required hypothetical storm size to cover the drainage areas for the Riter Canal was approximately 30 sq. miles. Based on the SLC DARF curves the appropriate DARF factor for this area would be 0.78.

The flows for each subbasin were predicted utilizing the techniques described in Chapter II. The resulting hydrographs were inserted into the hydraulic model at the relative location they would expect to enter the canal. The flows were then routed through the canal to predict anticipated flowrates and stages in the canal. Figure VIII-3 and Figure VIII-4 show the extents of the hydraulic model and highlights cross sections that are included in Table VIII-1 with peak flows for the 10-year and 100-year events along the canal. It should be noted that the flow rates reported in Table VIII-1 do not include overbank flows, just flows conveyed in the main channel.

A key assumption in this analysis is that the ponds near Stonebridge Golf Course can restrict releases to 40 cfs. Based on this assumption and the results from the hydrologic modeling the ponds would be expected to rise to a water surface elevation of 4245.6 feet during a 100-year runoff event.



HANSEN ALLEN & LUCE INC ENGINEERS

Salt Lake County Flood Control

Riter Canal
Alignment and Subbasins

FIGURE
VIII-1

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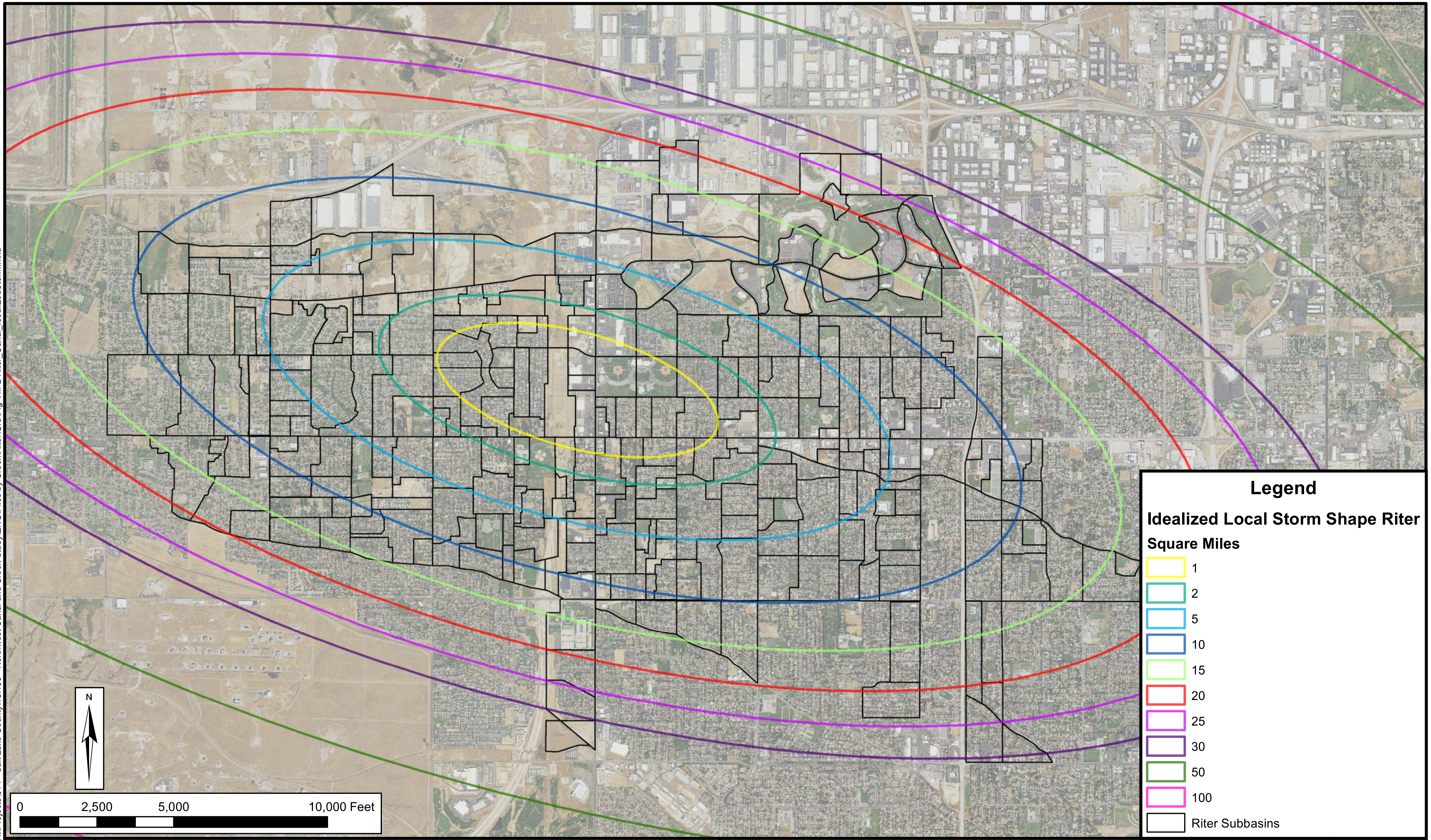

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Salt Lake County Flood Control

Riter Canal
Idealized Storm Area

FIGURE
VIII-2

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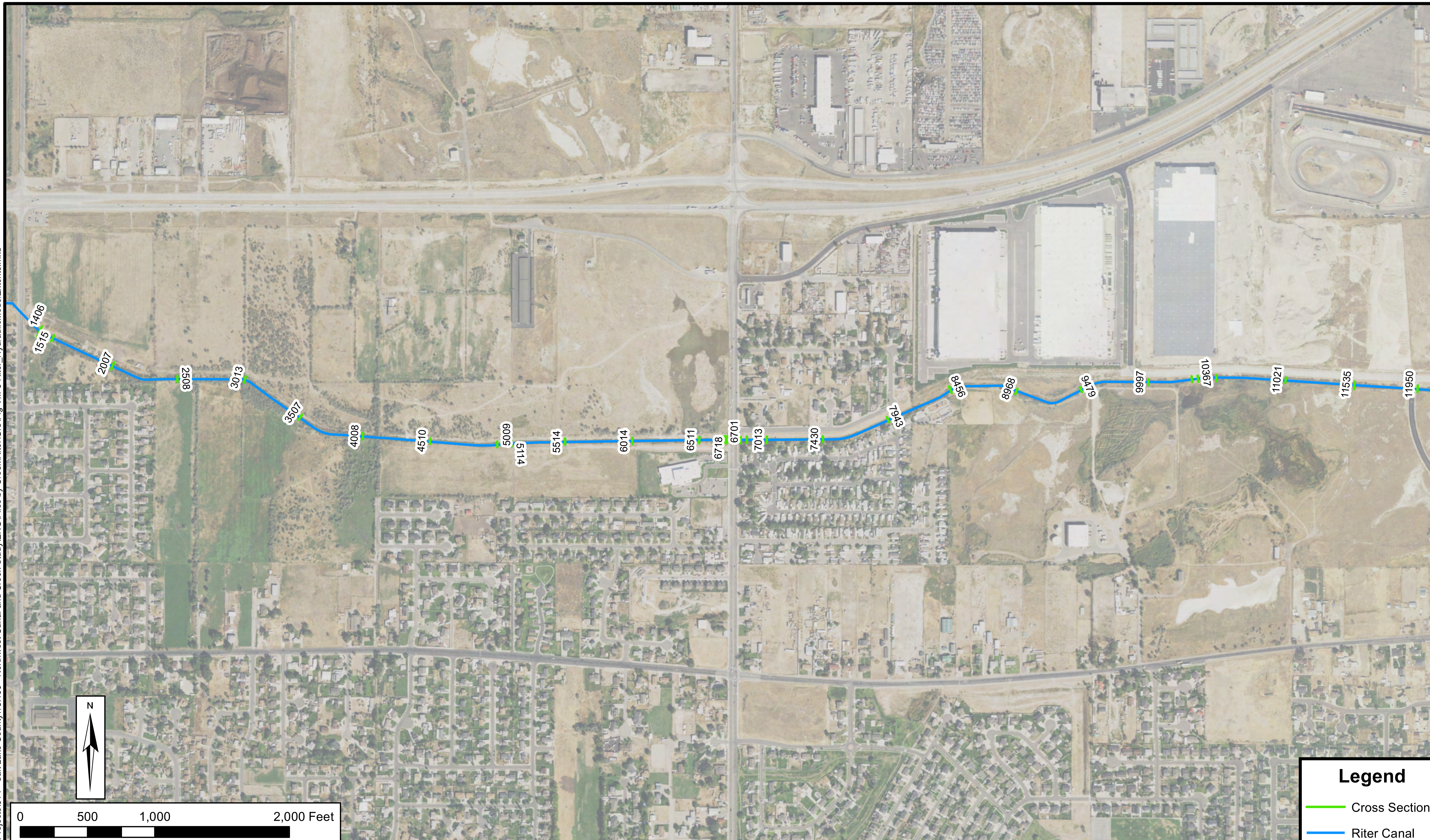

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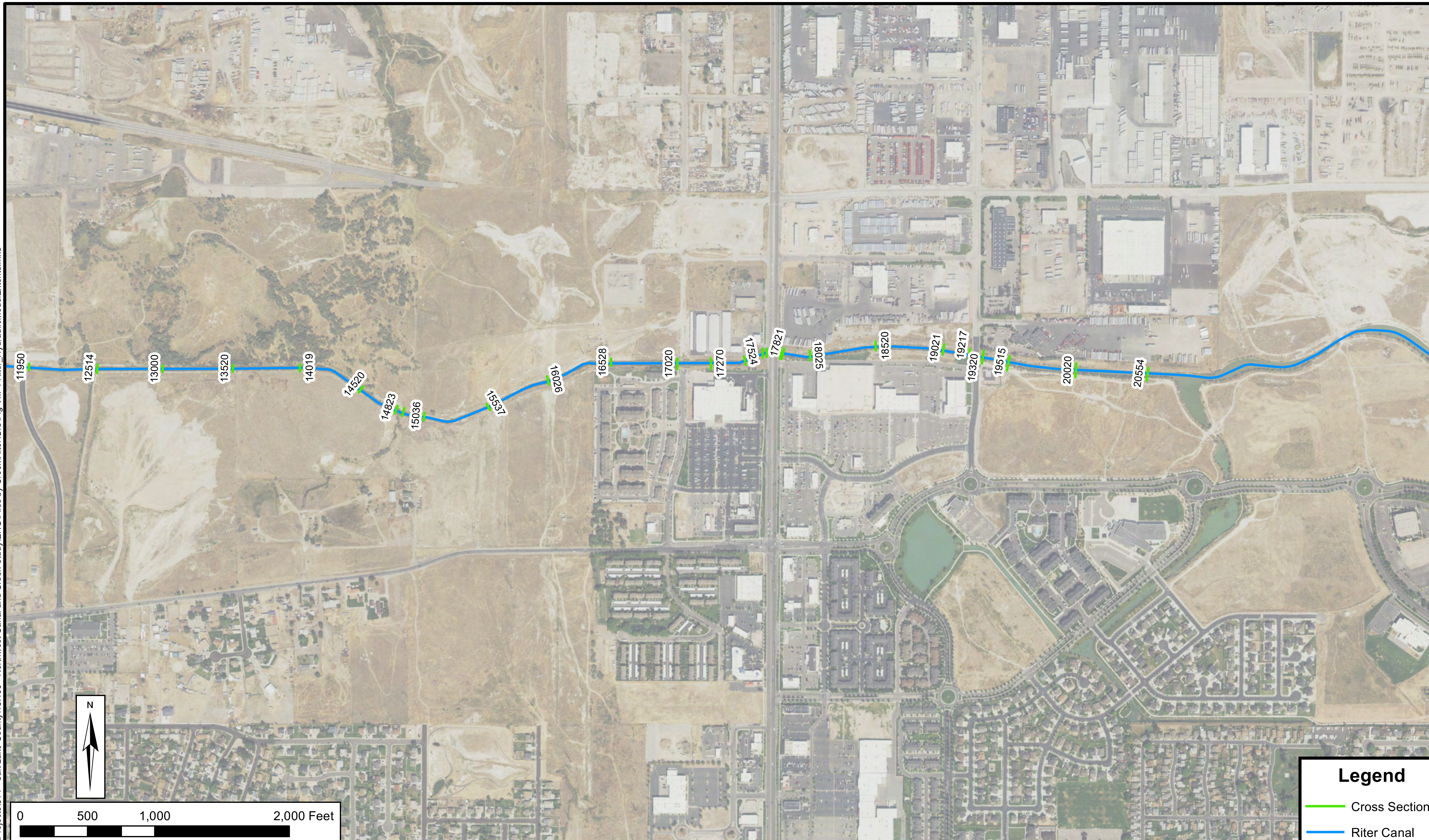
Salt Lake County Flood Control

Riter Canal
Hydraulic Model Extents (1 of 2)

FIGURE VIII-3



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Salt Lake County Flood Control

Riter Canal
Hydraulic Model Extents (2 of 2)

FIGURE VIII-4

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TABLE VIII-1. Riter Canal 10-year and 100-Year Existing Peak Flow Rates.

CROSS SECTION NUMBER	10-Year Peak Flow (cfs)	100-Year Peak Flow (cfs)
1406	98	158
4008	135	229
6819	82	124
9479	78	111
11853	205	427
12466	216	467
14019	122	183
14877	100	155
15537	202	400
17524	245	522
17700	105	181
19320	95	159
20554	40	40

EXISTING SYSTEM CAPACITY

The Riter Canal model hydraulic model was originally completed around 1990. It was updated in 2008 by Bowen Collins and Associates[9]. HAL georeferenced this model and converted it to a 1D/2D model. Additionally, HAL surveyed two locations to verify channel dimensions have not changed since the previous study was completed. The comparison of the survey data and corresponding sections showed the cross-sectional geometry is very close to what it was at the time the cross sections were developed and therefore the channel geometry in the model was not changed.

A 1D/2D model is when channel hydraulics are performed using standard 1D cross sections and any flows outside the overbanks are computed using a 2D grid. The connection between the channel and 2D overbanks is a lateral structure that is taken along the high ground of the channel. The Riter Canal cuts through a few natural drainages and therefore has a few low-lying elevations along its banks where water will back up into these natural low-lying areas. These areas provide excellent natural floodplain storage and attenuate peak flows. An example of this is approximately 2,700 feet downstream of 5600 West and is shown in Figure VIII-5. Our analysis assumes the highlighted area on Figure VIII-5 will remain available for floodplain storage into the future. The footprint of the low laying area providing the storage without modification is 11.5 acres (A cost estimate for acquiring this land is provided in the Appendix). That total area could be reduced with some additional modifications to the site (i.e. excavation). Dan Johnson with West Valley City indicated in one of our project meetings some of these areas have been filled in since the Lidar data was obtained in 2013. When new surface topography data is collected in this area new elevations can be extracted to the 2D grid and the lateral structures to update these models to account for the terrain changes.

[9] Bowen Collins and Associates (2008). Rital Canal Capacity Study. West Valley City, Utah.

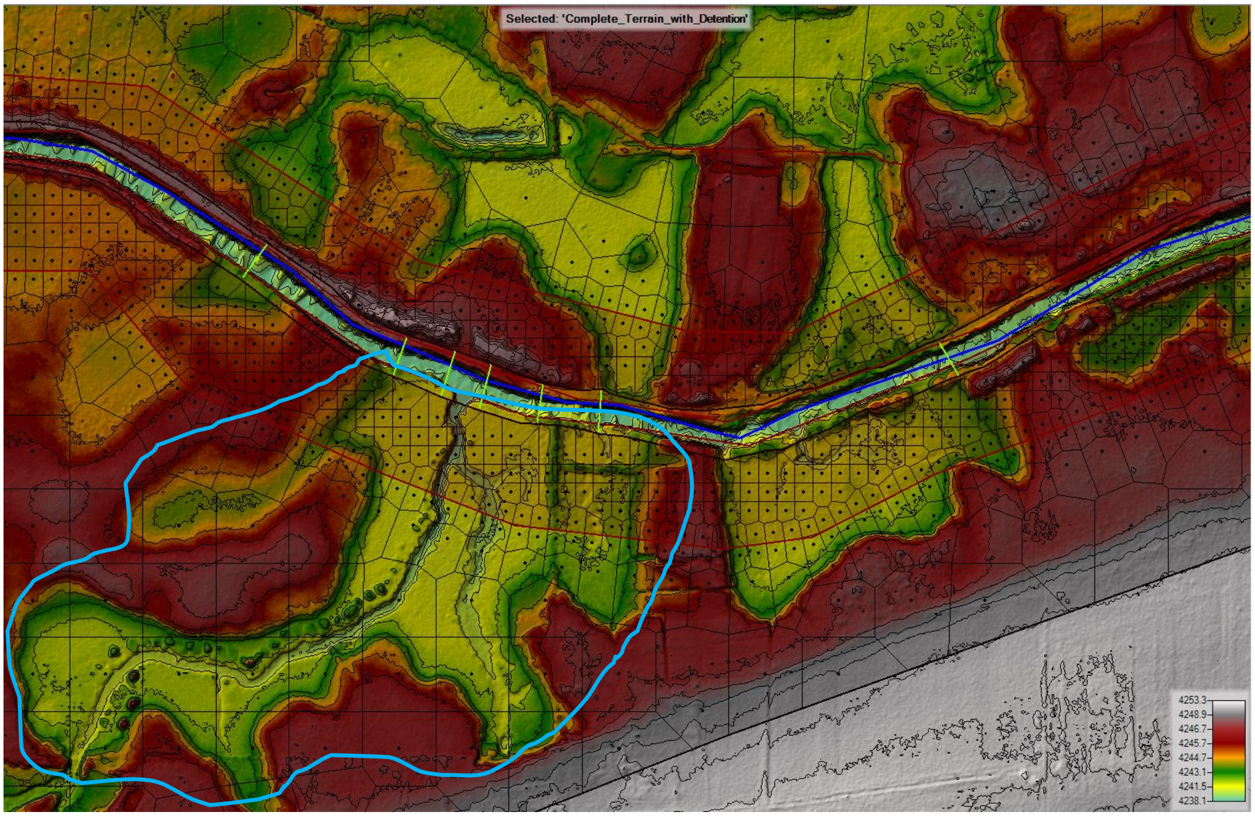


Figure VIII-5. Example of Natural Low-Lying Area Adjacent to the Riter Canal.

The plan and profile views (which includes inundation extents) for the 10-year and 100-year events are shown in Figure VIII-6 through Figure VIII-13.

DEFICIENCIES

The 10 and 100-year flows for existing conditions both show overtopping the banks of the Riter Canal in low bank areas. For the 10-year events most of the areas showing inundation are due to the low-lying areas along the channel that are undeveloped. There is flooding predicted at the businesses on the right (north) side of the channel at approximately 6750 West. A review of historical aerial imagery reveals this area was undeveloped at the time the Lidar data was collected. Recent aerial imagery shows that a road has been built along the right (north) side of the canal at this location. The construction of this road would have raised the bank elevations as compared to the 2013 Lidar dataset. The new elevation of the road likely eliminates flooding at this location during a 10-year event. This should be confirmed when an updated elevation surface is available.

The 100-year flows show some flooding of the neighborhood on the left side of the Rital Canal just east of 7200 West due to the undeveloped low lying ground to the east of the residential neighborhood as shown in Figure VIII-11. The same business area that showed some flooding for the 10-year event also shows flooding during the 100-year event. As noted previously development in this area has likely raised the banks as compared to the Lidar data used to define the height of the banks in the current model. It is recommended when updated terrain data is available that the bank elevations in the model be updated and re-run to determine if the flooding at this location is still of concern.

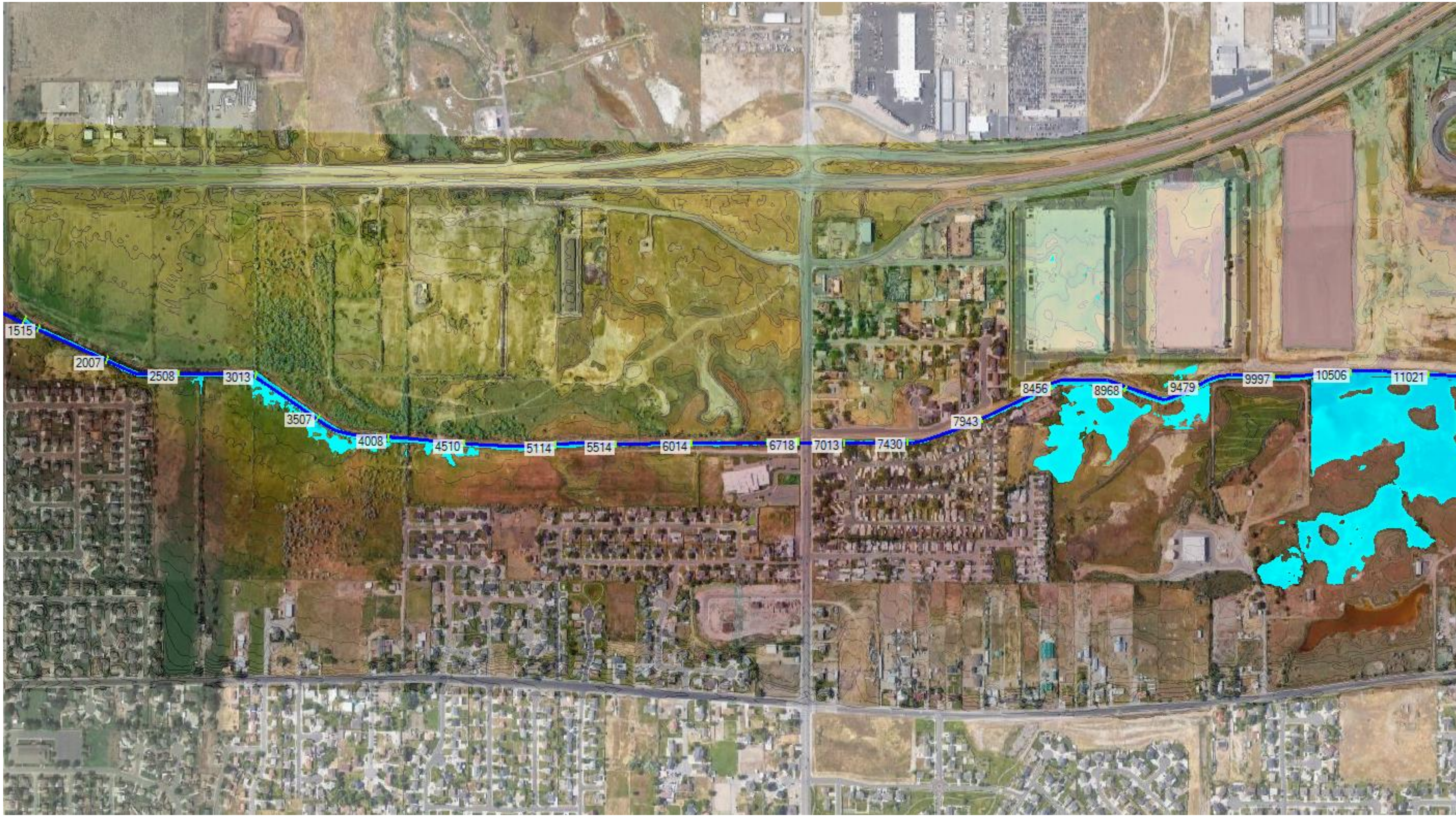


Figure VIII-6. Riter Canal Existing Conditions Plan View with Inundation Map for 10-Year Event (1 of 2).

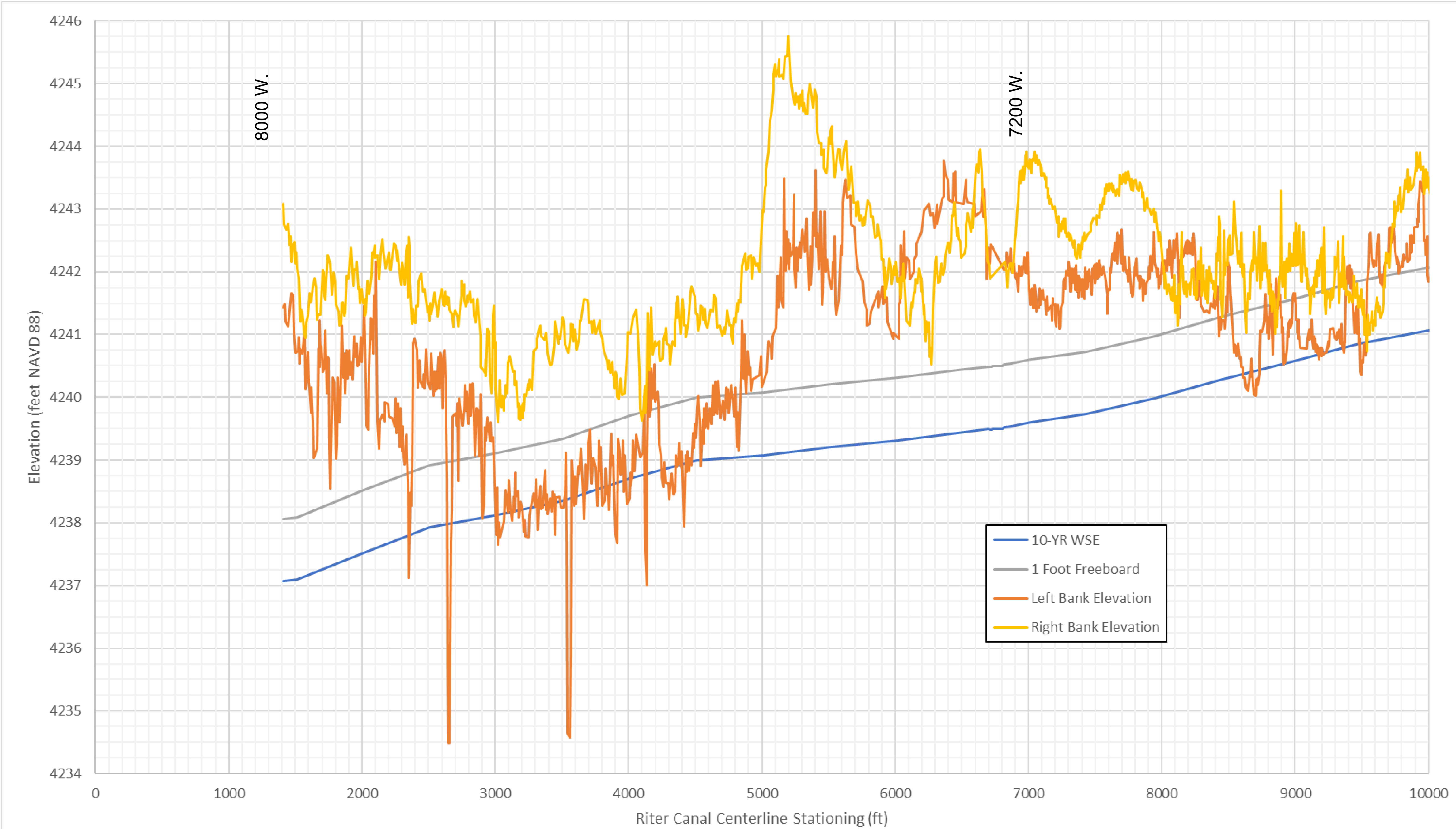


Figure VIII-7. Riter Canal Existing Conditions Profile with Bank Elevations for 10-Year Event (1 of 2).

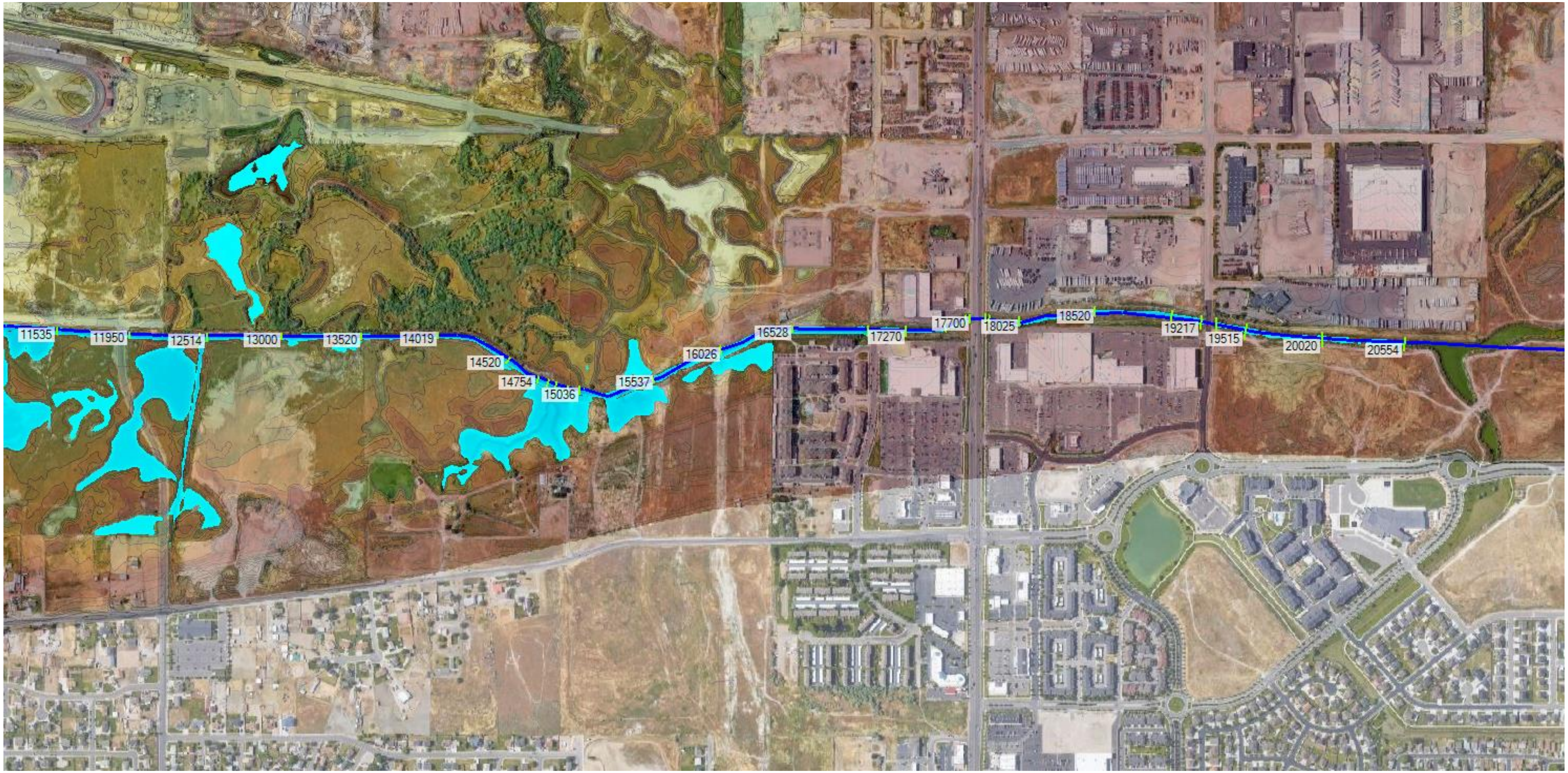


Figure VIII-8. Riter Canal Existing Conditions Plan View with Inundation Map for 10-Year Event (2 of 2).

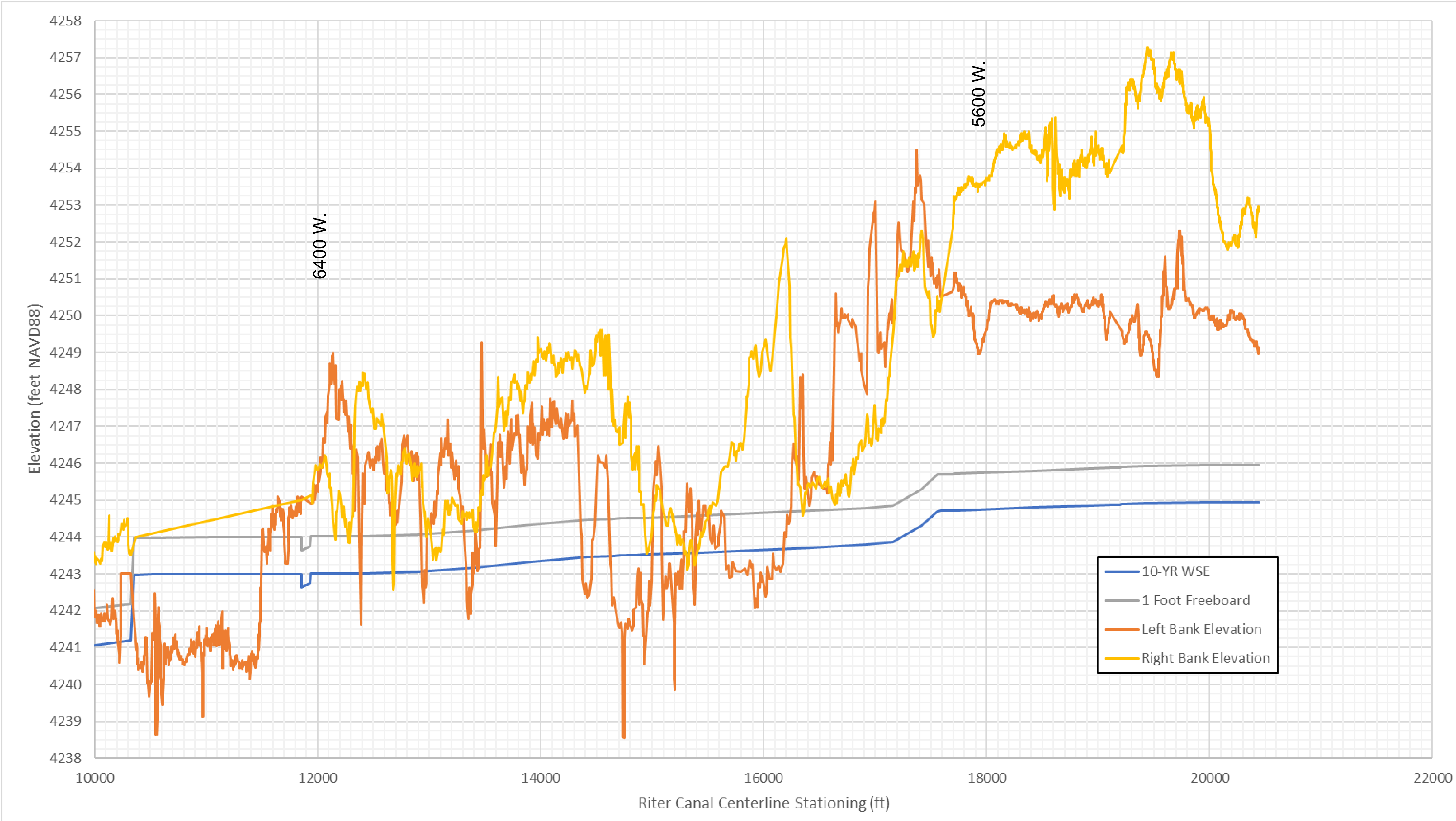


Figure VIII-9. Riter Canal Existing Conditions Profile with Bank Elevations for 10-Year Event (2 of 2).

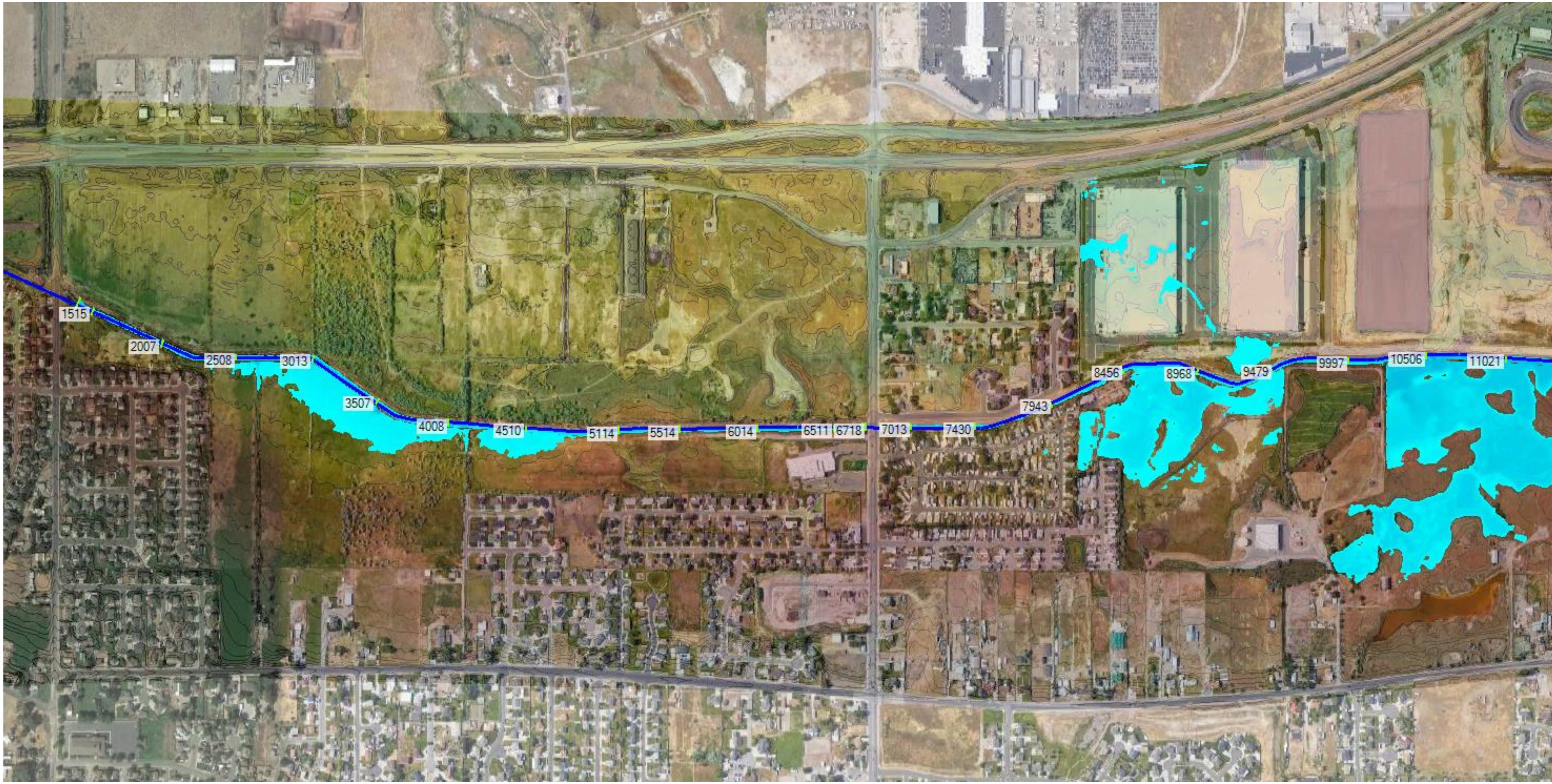


Figure VIII-10. Riter Canal Existing Conditions Plan View with Inundation Map for 100-Year Event (1 of 2).

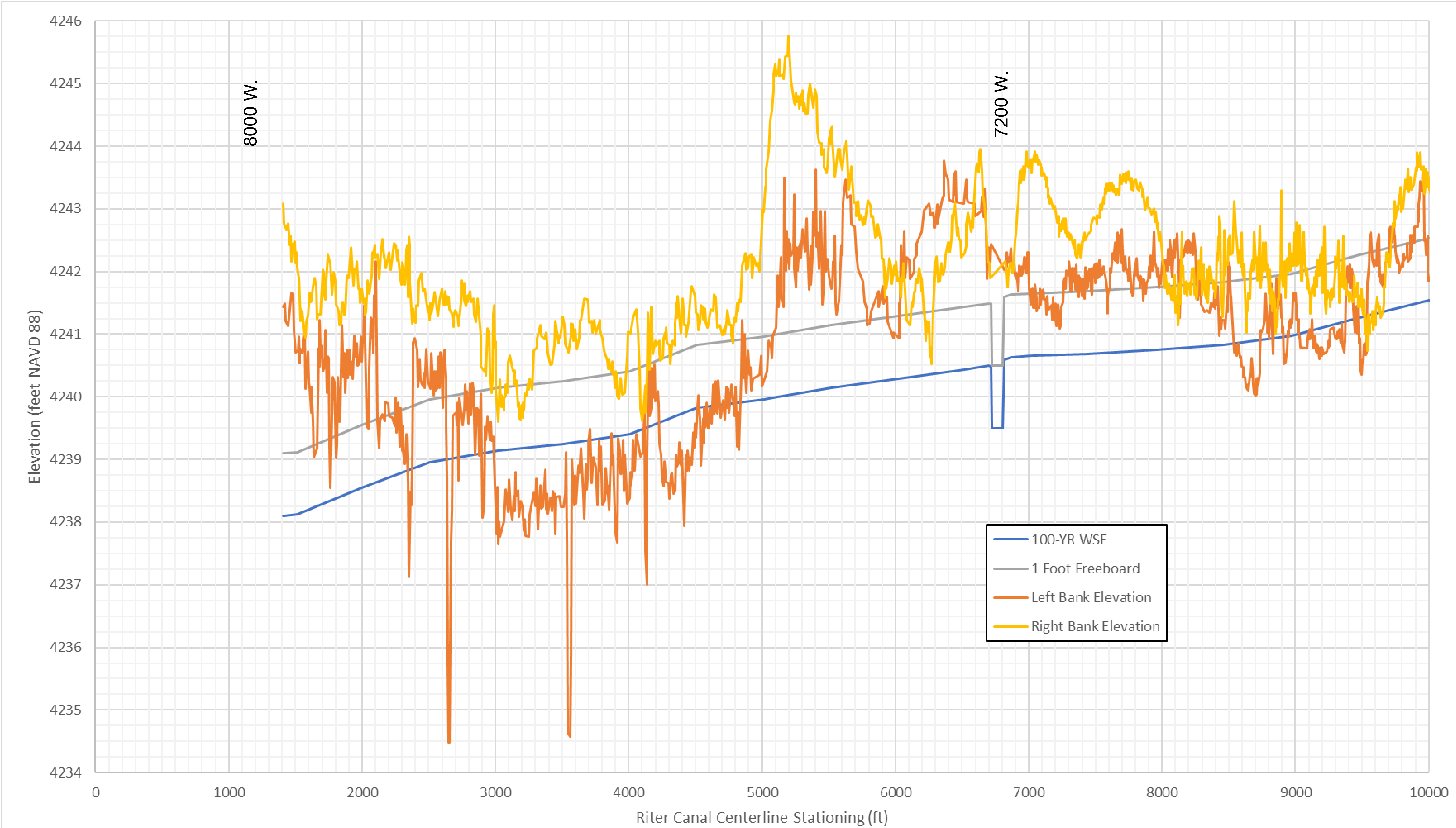


Figure VIII-11. Riter Canal Existing Conditions Profile with Bank Elevations for 100-Year Event (1 of 2).

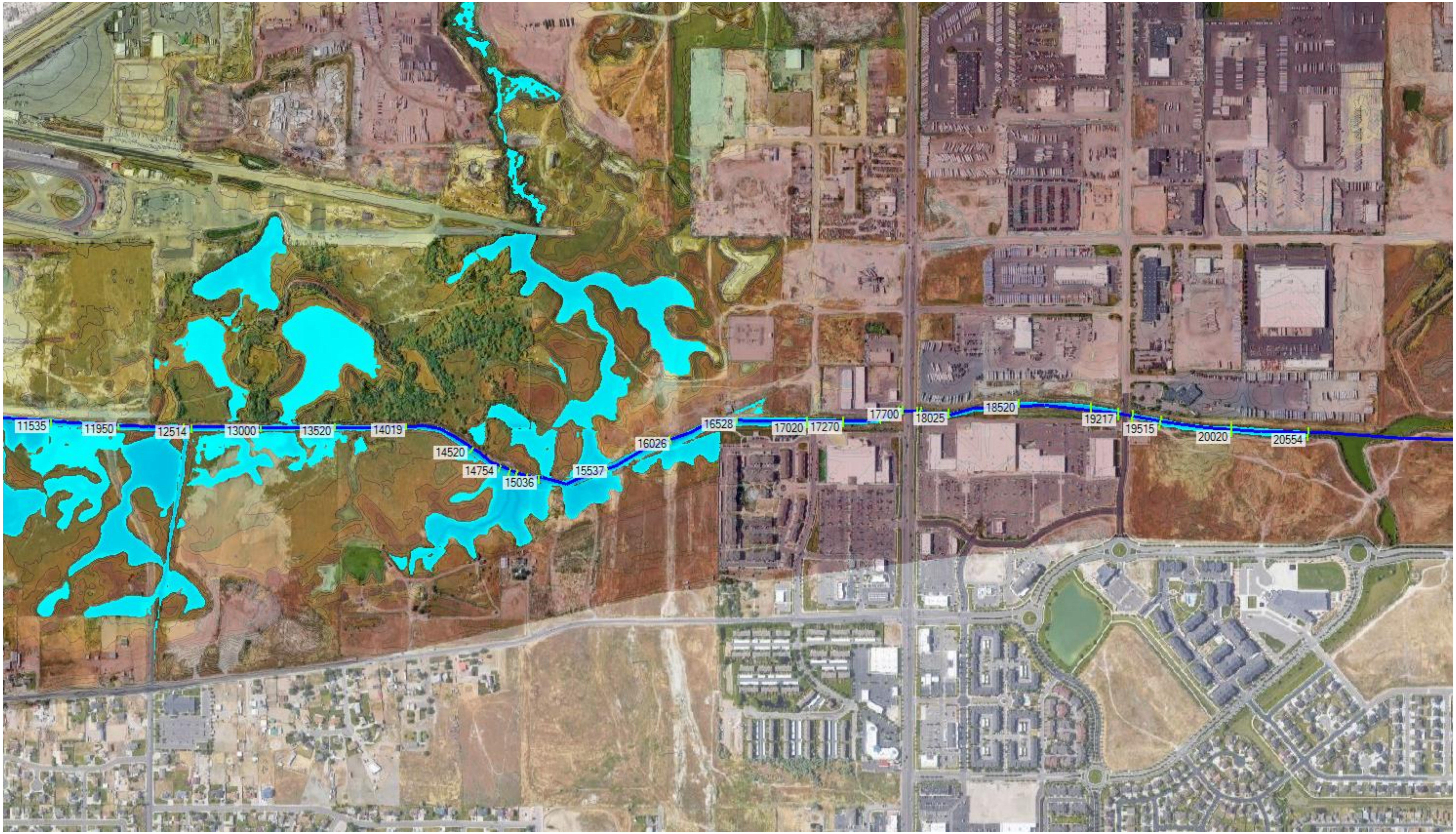


Figure VIII-12. Riter Canal Existing Conditions Plan View with Inundation Map for 100-Year Event (2 of 2).

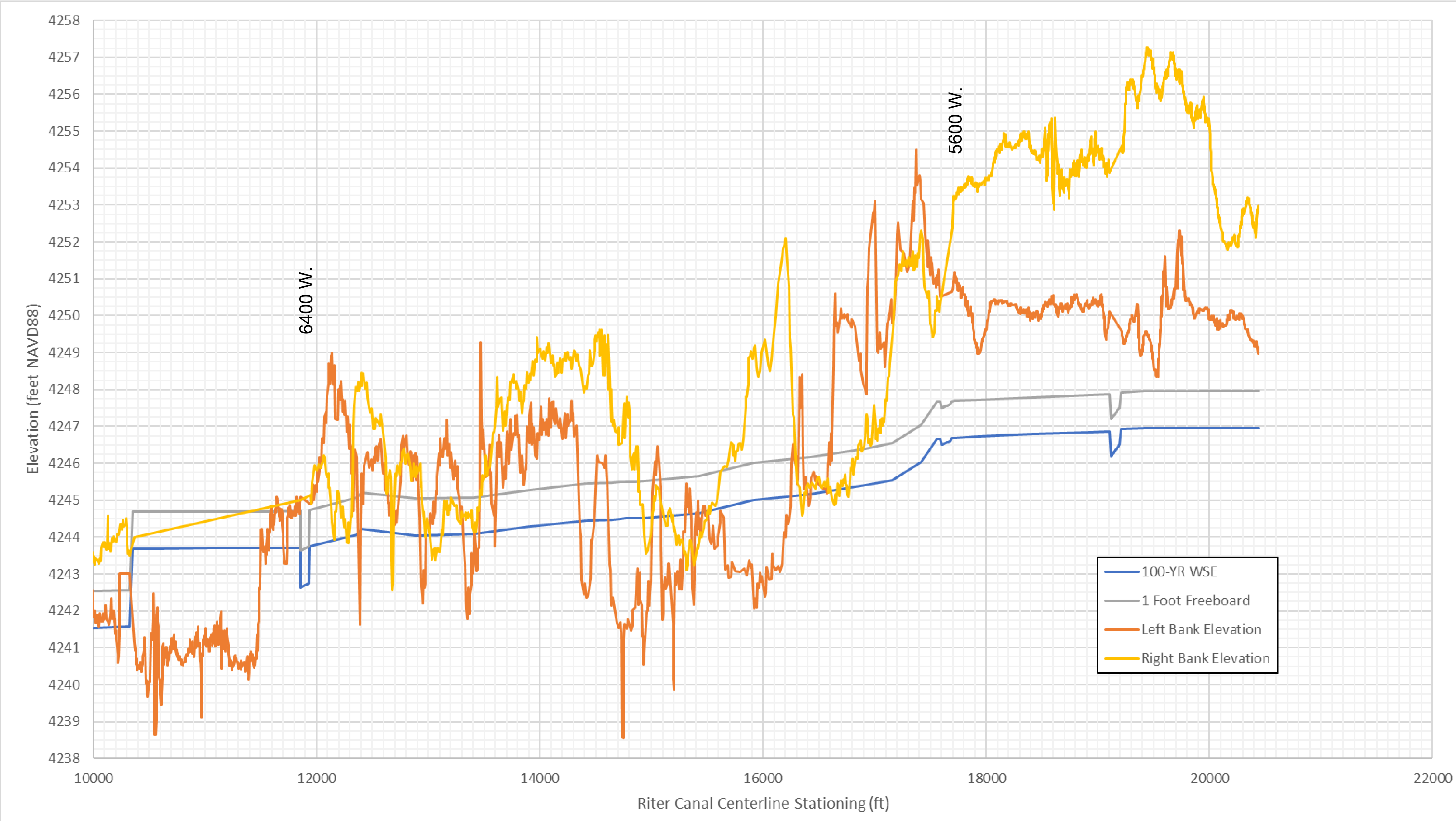


Figure VIII-13. Riter Canal Existing Conditions Profile with Bank Elevations for 100-Year Event (2 of 2).

ALTERNATIVES CONSIDERED

The model predicted instances of flooding along the Riter Canal are fairly minor and if development along the canal continues as it has in the past with the practice of using fill to bring the ground surface of the new developments above the Riter Canal 100-year water surface elevation; the issues identified by the current model may not present a flood risk anymore. The Riter Canal has development all around it and therefore the potential for expansion from its current footprint is limited. For this reason, alternatives that would involve increasing the size of the channel for additional capacity were not evaluated.

An alternative to reduce the computed water surface elevation (WSE) would be to control the channel and overbank vegetation to reduce the hydraulic roughness. Reducing the roughness will reduce the computed WSE for the same flows. The other component to the reduced roughness alternative would be establishing the required bank elevation to maintain one foot of freeboard for the 100-year event.

The historical Lee Creek drainage is bisected by the Riter Canal. HAL evaluated the possibility of sending water from the Riter Canal down the Lee Creek drainage if the Riter Canal was nearing its capacity limits. West Valley City indicated they were currently working with a developer at this location who would like to bring in fill and pipe a portion of Lee Creek. The downstream culvert limitation was also brought up as a concern for sending very much water this direction. West Valley City and the County decided that an overflow at this location was worthwhile, but its capacity should be limited to the downstream capacity of the culvert that goes under Highway 201 which is approximately 30 cfs.

Additional storage at the existing detention facility located on the left side of the Riter Canal at approximately 6700 West was evaluated as well. Our analysis indicated that the benefits of additional storage at this location are limited. According to model results, doubling the existing storage volume only reduces the computed WSE by approximately 0.5 feet at the detention facility. The reduction in WSE only propagates upstream for approximately 1,500 feet. Upstream of the bridge at 6400 West the computed WSE is essentially the same when comparing existing detention storage and doubling the existing storage volume scenarios.

PREFERRED SOLUTION

The preferred solution involves a combination of maintaining the channel to a level similar to what was found in the field during the study such that the channel roughness is 0.05 and raising the banks at key locations to achieve 1 foot of freeboard for the design flow events. If additional maintenance is performed (mowing the banks approximately twice per year) in the channel to reduce the channel roughness to 0.03 there would be a reduction of approximately 0.5 feet to required bank elevations at locations where they are deficient. Figure VIII-14 shows the computed 100-year peak flow rates assuming the preferred solution improvements have been implemented along the extents of the Riter Canal downstream of the control structure at the ponds near Stonebridge Golf Course.

One key assumption that these results are contingent on is leaving a portion of the historical Lee Creek Drainage open to provide some floodplain storage which allows for a substantial reduction to peak flows. The area we have assumed to be available for additional detention is highlighted on Figure VIII-5. This location provides approximately 18 acre-feet of storage for the design event and approximately 250 cfs of peak flow attenuation between model station 14754-15200.

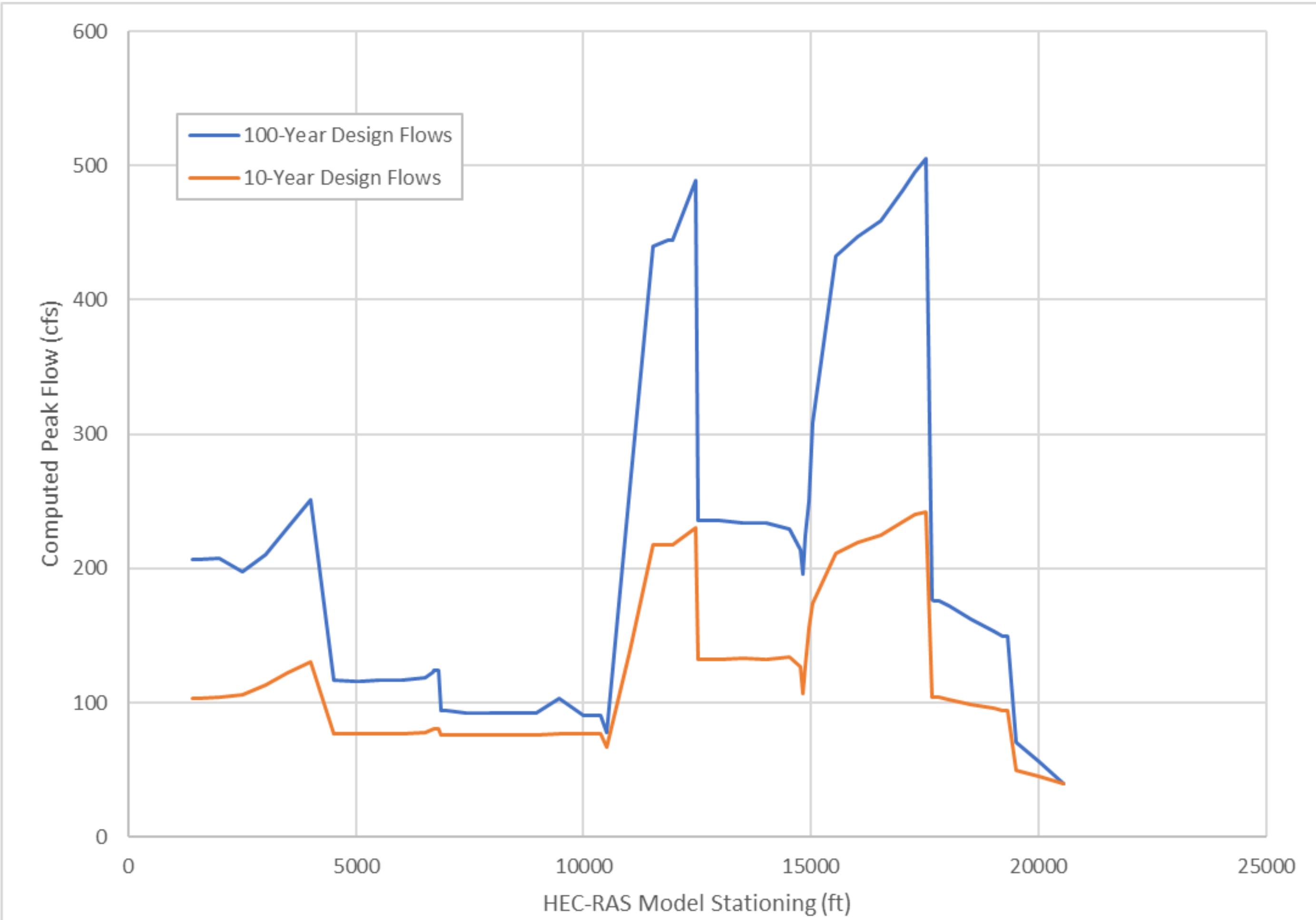


Figure VIII-14. Riter Canal Design Flow Rates.

The profile from the design 100-year event compared to existing bank elevations is shown in Figure VIII-15 and Figure VIII-16.

We recommend that fill be brought in for developments along the Riter Canal such that the ground provides at least 1-foot of freeboard, and first floor elevations of structures be two feet minimum above the 100-year WSEL. In developed areas the Lidar data generally shows an existing 8-foot bank top width. We recommend maintaining at least the existing 8-foot channel bank top width in existing developed areas. If an area is not yet developed, we recommend requiring an access road on at least one of the banks that is at least 15 feet wide.

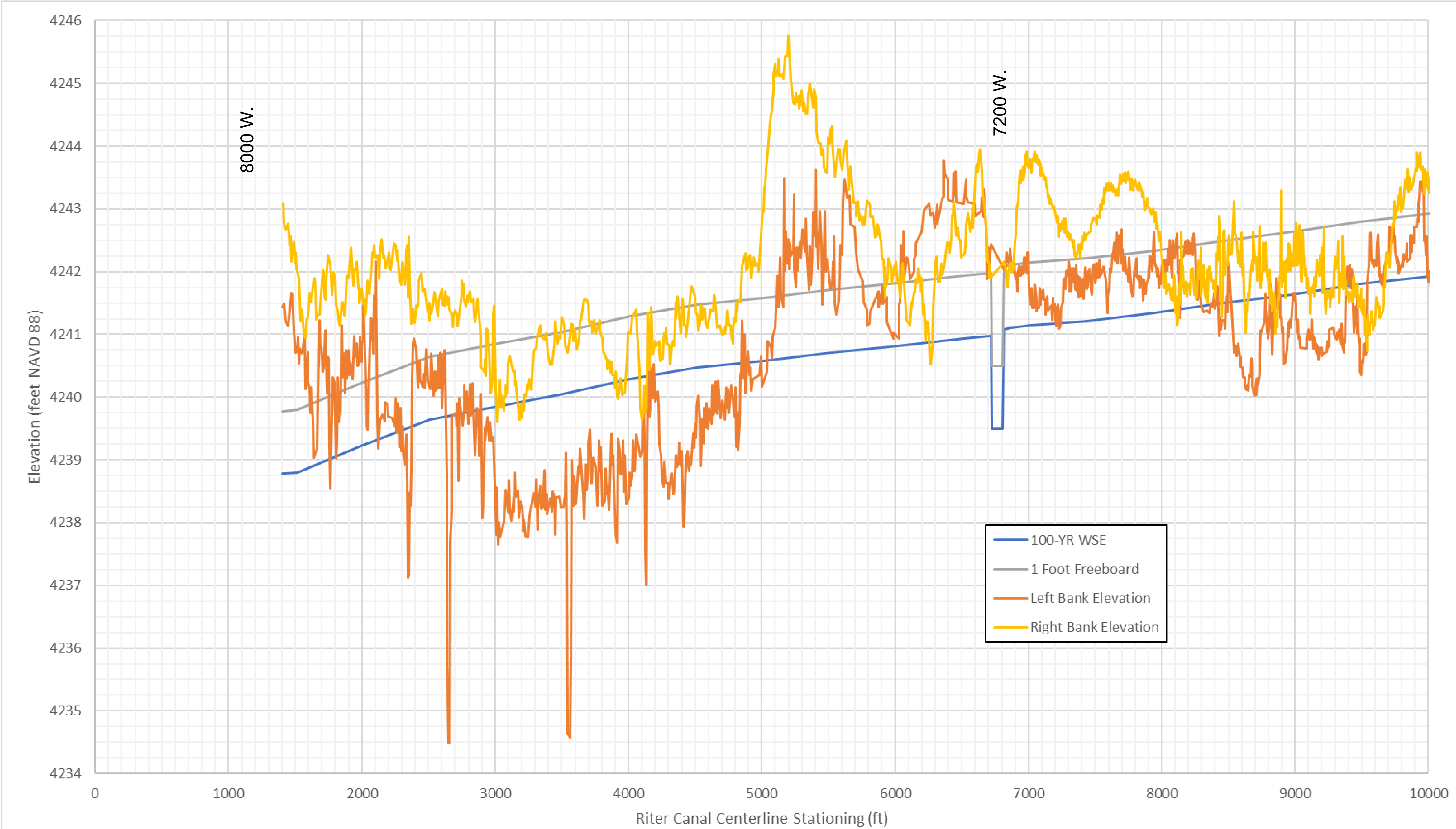


Figure VIII-15. Riter Canal Bank Elevation Profile for 100-Year Design Flows (1 of 2).

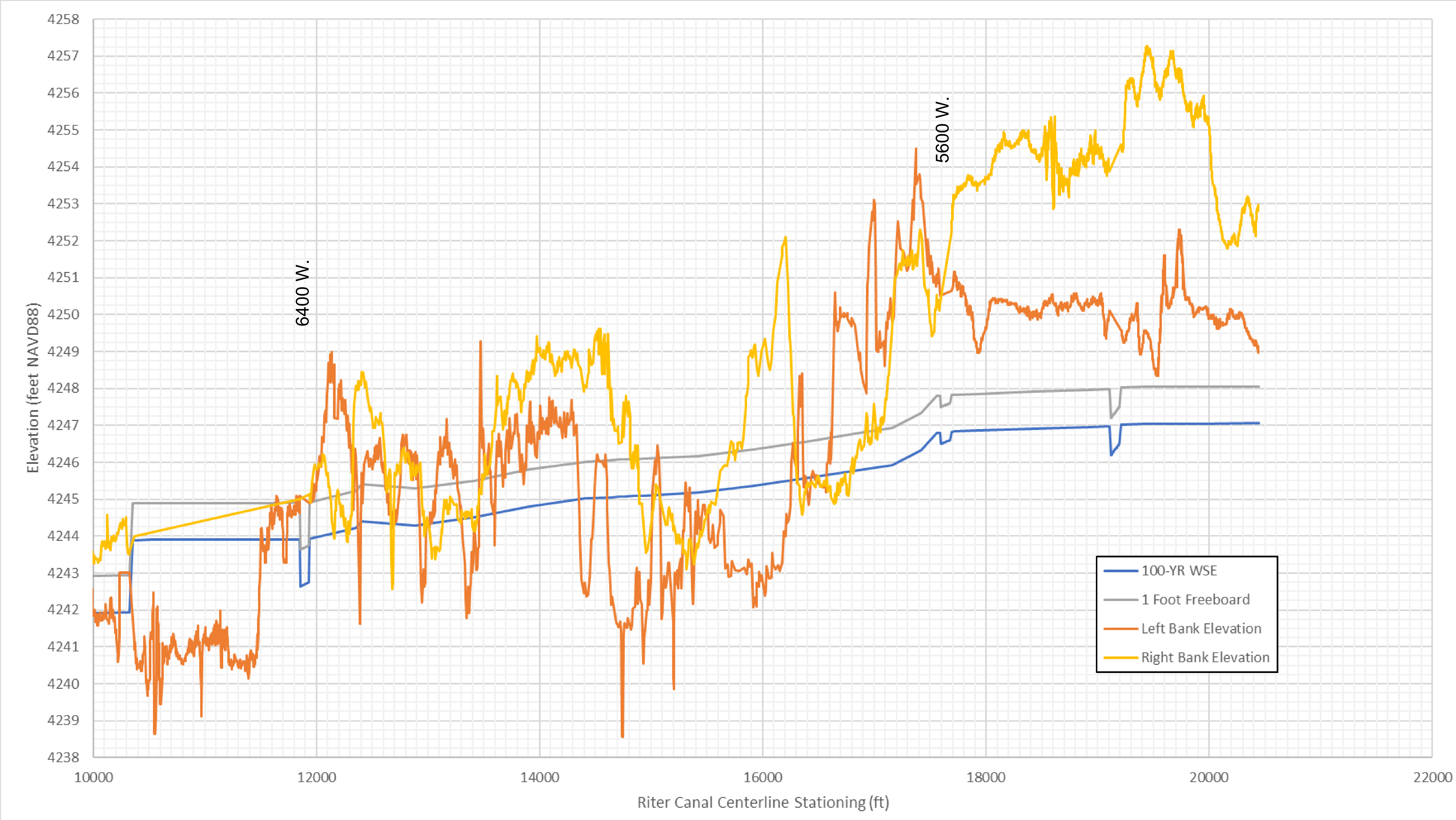


Figure VIII-16. Riter Canal Bank Elevation Profile for 100-Year Design Flows (2 of 2).

CHAPTER IX

CHAPTER IX – GOGGIN DRAIN

EXISTING SYSTEM DESCRIPTION

Goggin Drain begins on the west side of the Salt Lake International Airport at approximately 5100 West and 1000 North. The Goggin Drain receives flood waters from the Jordan River via the Surplus Canal. The flood flows in the Jordan River are managed in accordance with the Utah Lake Compromise Agreement. It flows generally to the west and passes under the North Point Canal on its way to eventually discharge into the Great Salt Lake. The extent of the Goggin Drain and the areas directly tributary to it are shown in Figure IX-1.

Beginning about 8800 West, there is erosion along both banks of the Goggin Drain that continues downstream towards the Great Salt Lake for approximately 4500 linear feet. There is an existing levee located near the south bank of the Goggin Drain on Rio Tinto (Kennecott) property that is at risk of being compromised because of this erosion. This levee protects ponds to the south. At the time of this study, it is unknown how fast the erosion is progressing.

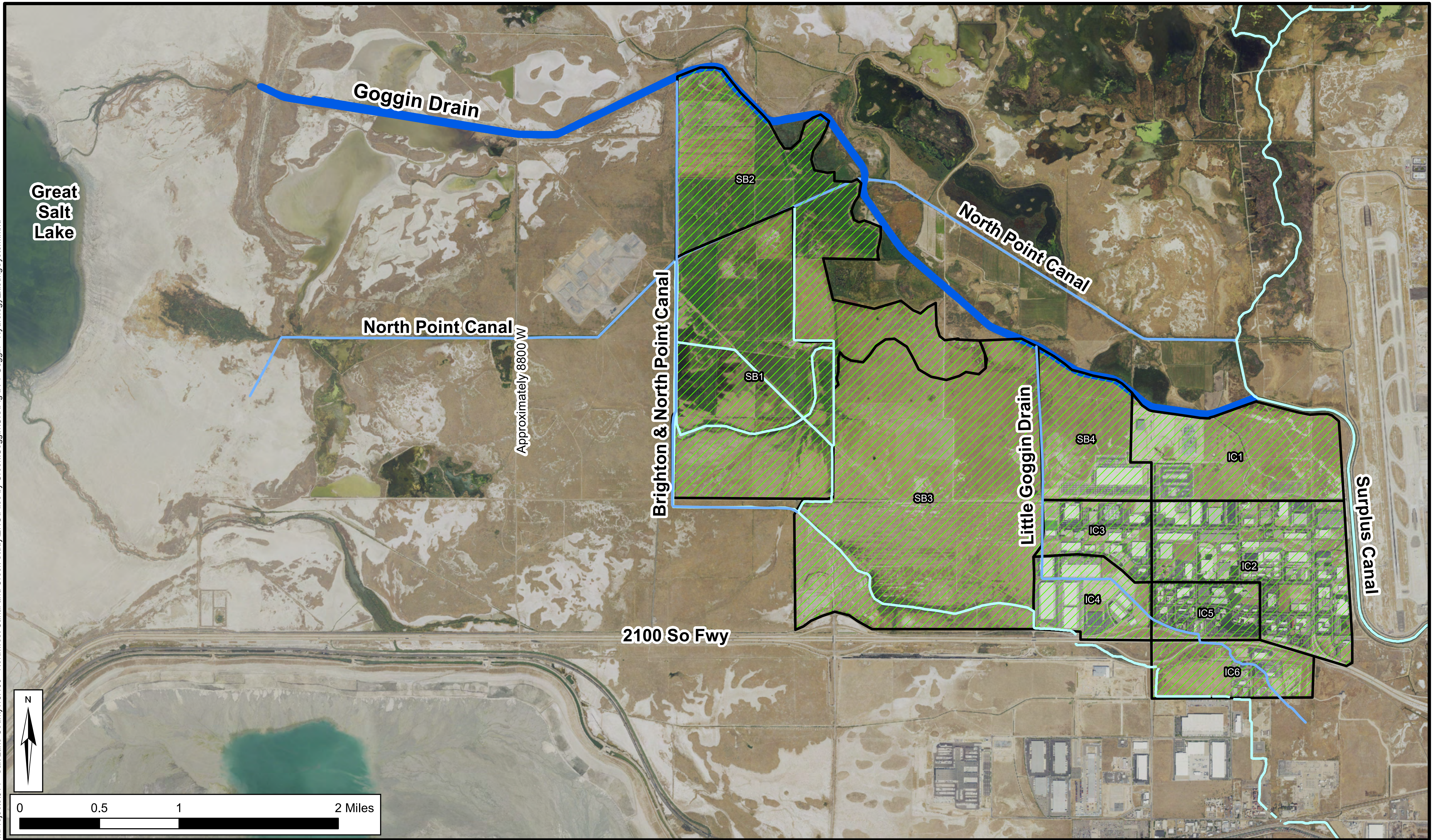
PREDICTED 10-YR AND 100-YR FLOWS

The basins west of the International Center draining to the Goggin Drain were delineated in a previous study completed by HAL in 2011. The land use was updated using the Northwest Quadrant Master Plan, which was adopted in 2016. Specifically, the Future Land Use Map was used to predict what development will look like in this area at buildout. Measurements of land use by basin (per the Future Land Use Map) were tabulated. Percent impervious for eco- and light industrial were assumed to be 85% directly connected. Subbasins representing the International Center (blue shaded on Figure IX-2) were added to the model. The HEC-HMS model schematic for the area tributary to the Goggin Drain is shown in Figure IX-2.

Based on these parameters, the predicted peak storm runoff flowrates for the 10-year and 100-year storm events are summarized in Table IX-1 for areas west of the International Center.

Table IX-1. Hydrology Results based on Land Use from 2016 NWQ Master Plan.

Basin	Area (ac)	Land Use (percent)			Percent Imperv.	Q_{10} cfs	Q_{100} cfs
		Natural	Eco-Industrial	Light Industrial			
SB1	959	0	0	100	85	1,000	1,816
SB2	839	45	12	43	46.5	488	989
SB3	1,520	11	3	86	75.4	1,420	2,608
SB4	317	61	13	27	34.2	137	301
IC1	1,021	15	8	77	72.3	912	1,692
IC2	300	0	0	100	85	105	105
IC3	524	0	0	100	85	314	569
Outfall	4,117					1,217	2,166
					Peak Flow Runoff	1,810	3,260



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HANSEN ALLEN & LUCE INC ENGINEERS	SALT LAKE COUNTY	HYDROLOGY BASINS AND OTHER CONVEYANCE FEATURES NEAR THE GOGGIN DRAIN	FIGURE IX-1

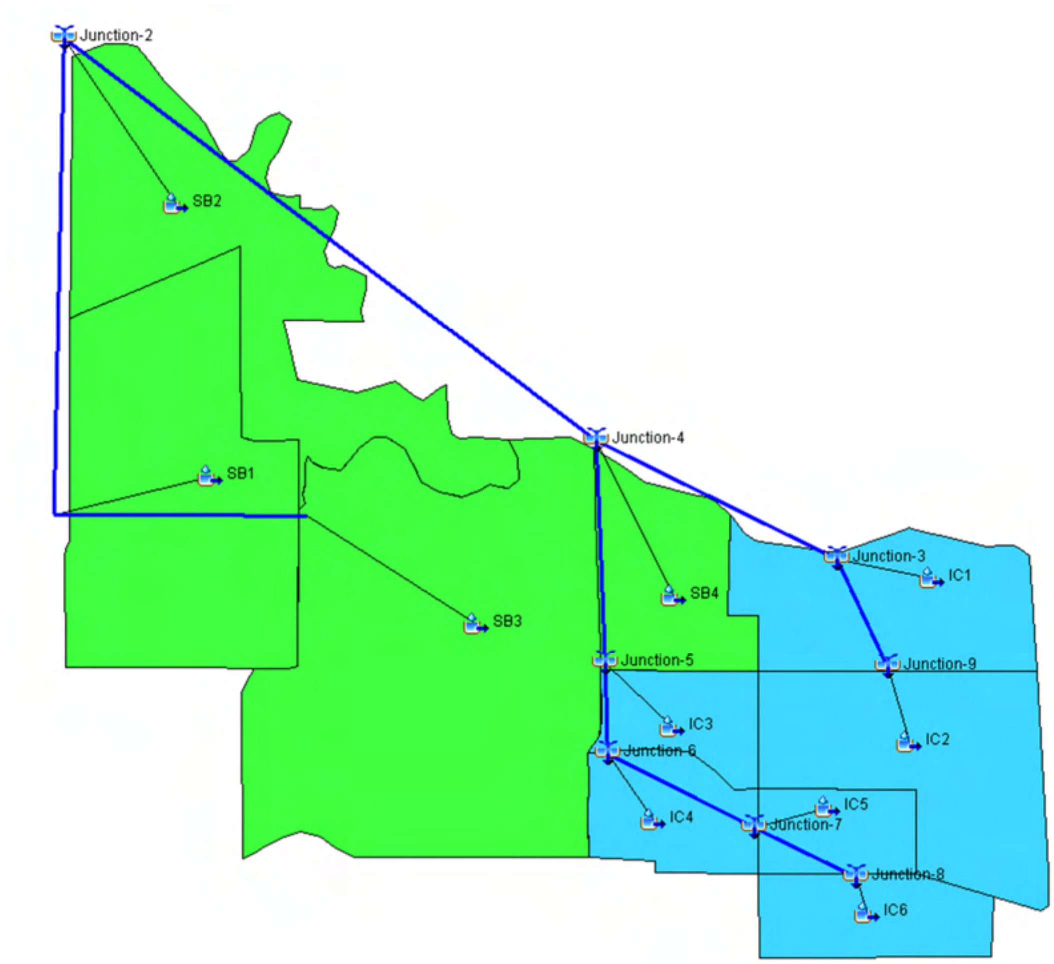


Figure IX-2. Goggin Drain HEC-HMS Model Schematic.

In addition to the local runoff reported above, the estimated maximum historic peak flow coming from the Surplus Canal is 2,500 cfs. Two detention scenarios were tested in the hydraulic model to assess the potential impact of development in this area.

EXISTING SYSTEM CAPACITY

The existing system capacity was tested using the HEC-RAS model developed in the 2011 HAL study. The HEC-RAS model was run with three different flows. Each flow represented a different scenario; scenarios included the base case of the 100-year snowmelt with 100-year storm runoff from existing development (2,500 cfs plus 480 cfs = 2,980 cfs) and a second and third case assuming the undeveloped tributary area is detained to a release rate of 0.1 or 0.2 cfs per acre. The downstream boundary condition was set to an elevation of 4214.0 feet above sea level (NAVD 88) based on the approximate elevation of the FEMA BFE at the Great Salt Lake. The flow change used for the local drainage west of the International Center was at section 11910, at the outfall of the Brighton & North Point Canal/Jordan Meander outfall. A summary of the hydraulic modeling results are shown in Table IX-2.

Table IX-2. Hydraulic Modeling Scenario Comparisons.

Q	Scenario	WSE (NAVD 88)	Δ WSE	Δ Floodplain Acres
2,980	Base Case	4215.24	0.00	0
3,600	0.2 cfs/acre	4215.56	0.32	29

Notes:
Elevation Data Source - 2011 Survey and 2013-2014 LiDAR
WSE Measurement Location - Cross Section 11910
GSL Elevation - 4214 (approximate FEMA BFE at GSL)

PREDICTED FLOODPLAIN MAPPING

The model results were overlaid on the publicly available Wasatch Front LiDAR 2013-2014 surface to compare the results. The resulting predicted floodplain for the base case (2,980 cfs) and the development with 0.2 cfs/acre detention case (3,600 cfs) are shown on Figure IX-3. The 0.2 cfs/acre detention scenario results in additional flooding of about 29 acres.

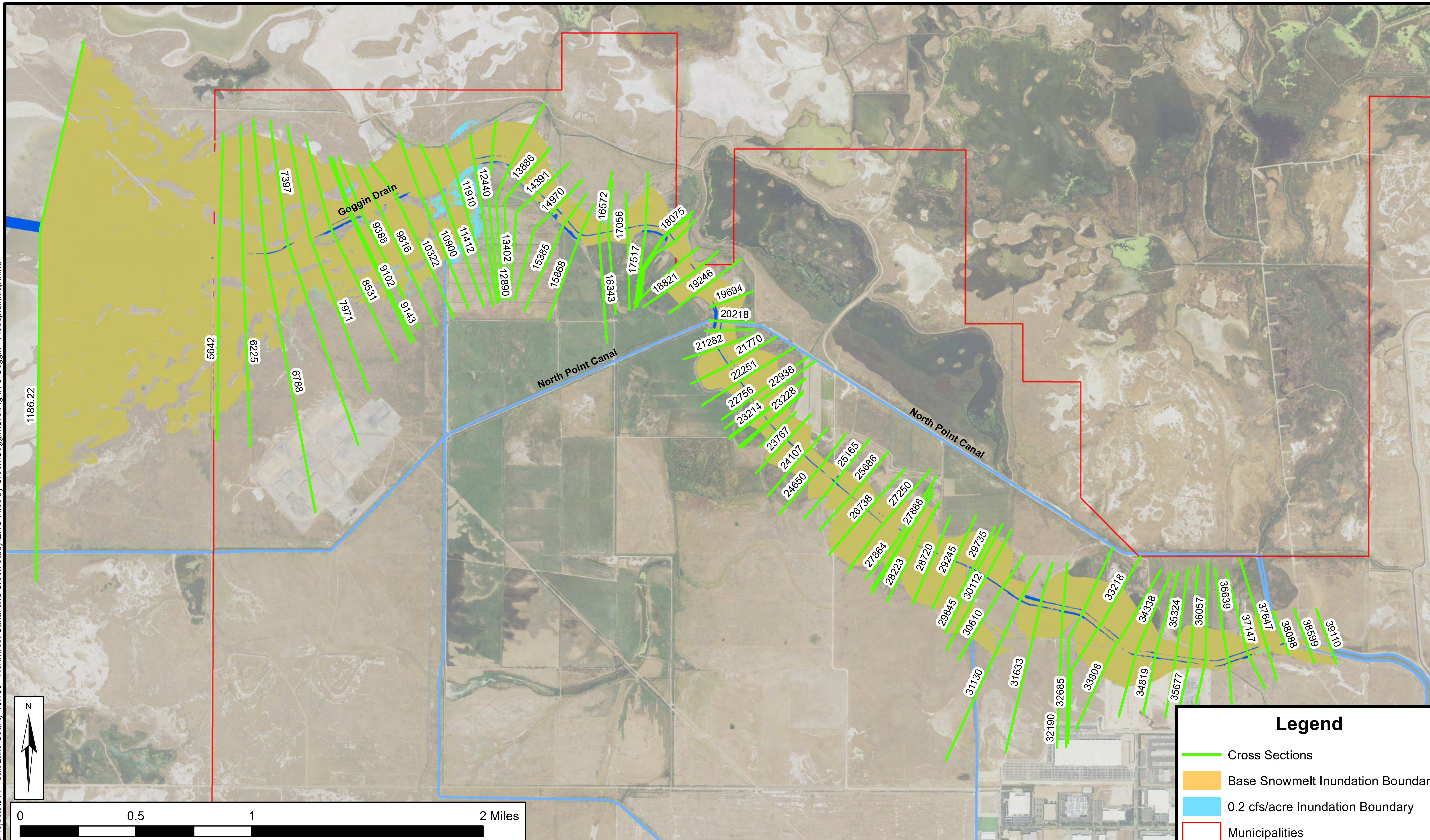
ALTERNATIVES CONSIDERED

The alternatives considered for the Goggin Drain include:

- Improvement of the Goggin Drain downstream of the Jordan Meander Confluence outfall so that it can carry increased development flows without impacting the 100-year floodplain.
- Requirement of future development detaining to the 0.1 cfs/acre standard.
- Requirement of future development detaining to the 0.2 cfs/acre standard.

RECOMMENDED SOLUTION

The preferred solution requires development to detain their stormwater runoff to 0.2 cfs/acre in a 100-year event consistent with current Salt Lake City detention requirements. Implementing the 0.2 cfs/acre requirement minimizes the impacts from development, but still allows it to occur and make beneficial use of the surrounding land. Additionally, we recommend that the first-floor elevation of buildings be built with two feet minimum freeboard above the computed 100-year water surface elevation.



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SALT LAKE COUNTY FLOOD CONTROL

GOGGIN DRAIN INUNDATION COMPARISON

FIGURE IX-3

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CHAPTER X

CHAPTER X - KEARNS-CHESTERFIELD DRAIN

EXISTING SYSTEM DESCRIPTION

The Salt Lake County conveyance Kearns-Chesterfield Drain begins at an overflow structure on the Utah and Salt Lake Canal just downstream of 4700 South. It extends to the east for approximately 1800 feet and then bends to the North and parallels Bangerter Highway until approximately 2900 South where it goes under Bangerter Highway. Up to this point the Kearns-Chesterfield Drain is a closed conduit drainage system. The details for the piped section were extracted from the UDOT Bangerter Highway plans which included the construction of this closed piped section. After going under Bangerter Highway the Kearns-Chesterfield Drain generally drains to the east. It passes through Decker Lake and eventually discharges to the Jordan River. The conveyance after crossing Bangerter Highway includes sections of open channel flow and large box culverts under road crossings. The alignment of the Kearns-Chesterfield Drain and its tributary area are shown on Figure X-1.

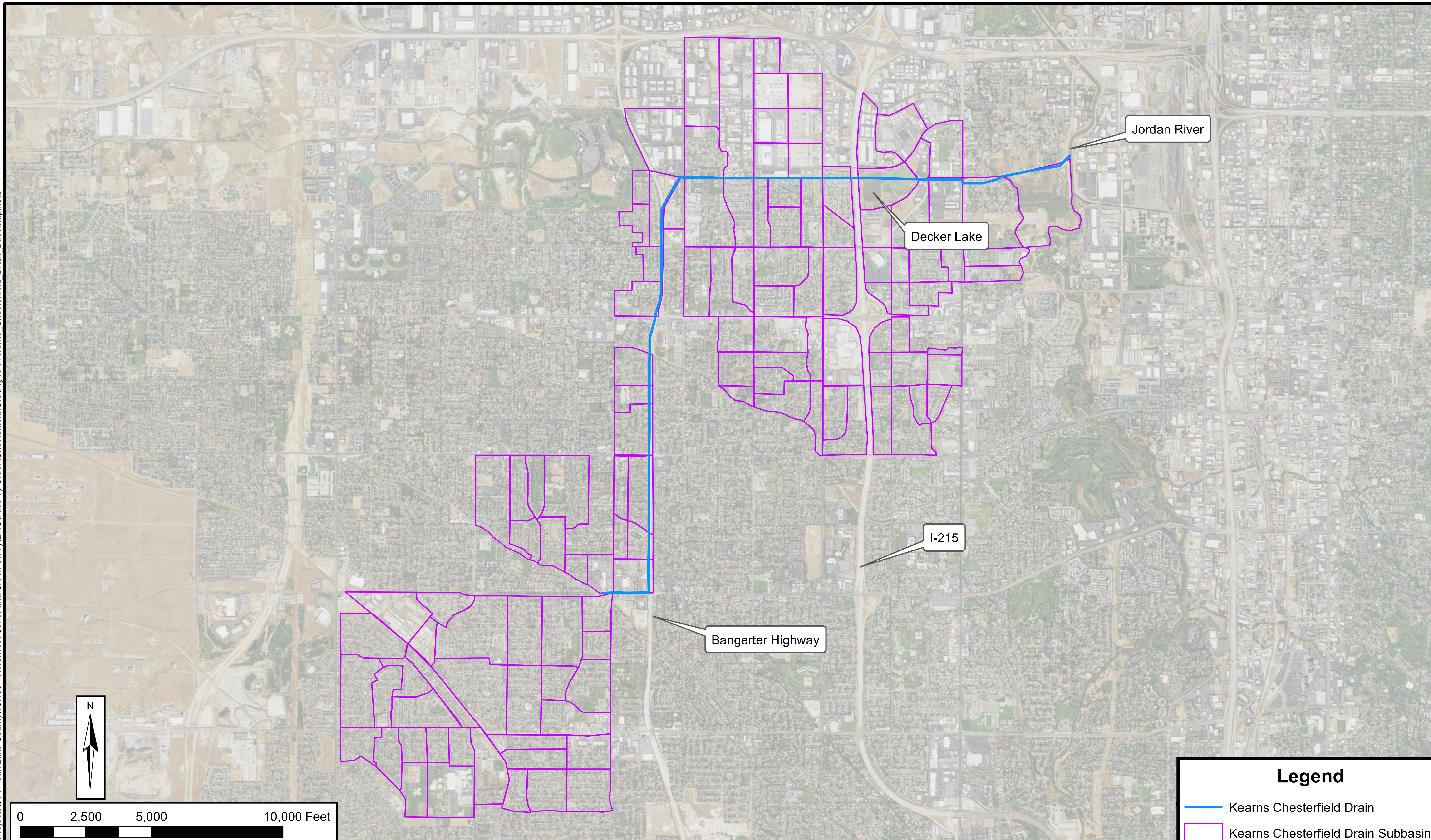
PREDICTED 10-YR FLOWS

The area that is tributary to the Kearns-Chesterfield Drain was delineated to determine incremental hydrographs at discrete locations along the drainage facility. A map of the subbasins tributary to the Kearns-Chesterfield Drain are shown in Figure X-1.

Design storm considerations were given to the elongated and discontinuous nature of the areas that are tributary to the Kearns-Chesterfield Drain. The approach for this study was to apply a Depth Area Reduction Factor (DARF) based on the area of a hypothetical storm shape covering the areas tributary to the Kearns-Chesterfield Drain simultaneously. Figure X-2 shows a hypothetical storm centered over the Kearns-Chesterfield Drain drainage area. The required hypothetical storm size to cover the drainage areas of the Kearns-Chesterfield Drain was approximately 25 square miles. Based on the SLC DARF curves the appropriate DARF factor for this area would be 0.80.

The flows for each subbasin were predicted utilizing the techniques described in Chapter II. The resulting hydrographs were inserted into the hydraulic models at the relative location they would expect to enter the drainage facility. The Kearns-Chesterfield Drain was modeled using two different software because of the distinct differences between the closed conduit portion and the open channel segments. The closed conduit section was modeled in SSA (or SWMM) and the open channel section was modeled in HEC-RAS. Figure X-3 shows the extents of both models. Figure X-3 also displays the 10-year peak flow rates as calculated by the models. The HEC-RAS model takes the computed outflow hydrograph from the SSA model and uses it as the upstream boundary condition.

The profile plot of the SWMM model is shown in Figure X-4. The computed flows were conveyed by the existing infrastructure without any computed HGLs exceeding the rim elevations. There is a maximum surcharge on the top of the pipes of approximately 0.6 feet at node 12. The computed HGL is anticipated to be more than 2 feet below the rim elevation at this location and is therefore considered to have sufficient capacity to convey the 10-year flows. The maximum computed depths for the piped section of the Kearns-Chesterfield Drain are shown in Table X-1.



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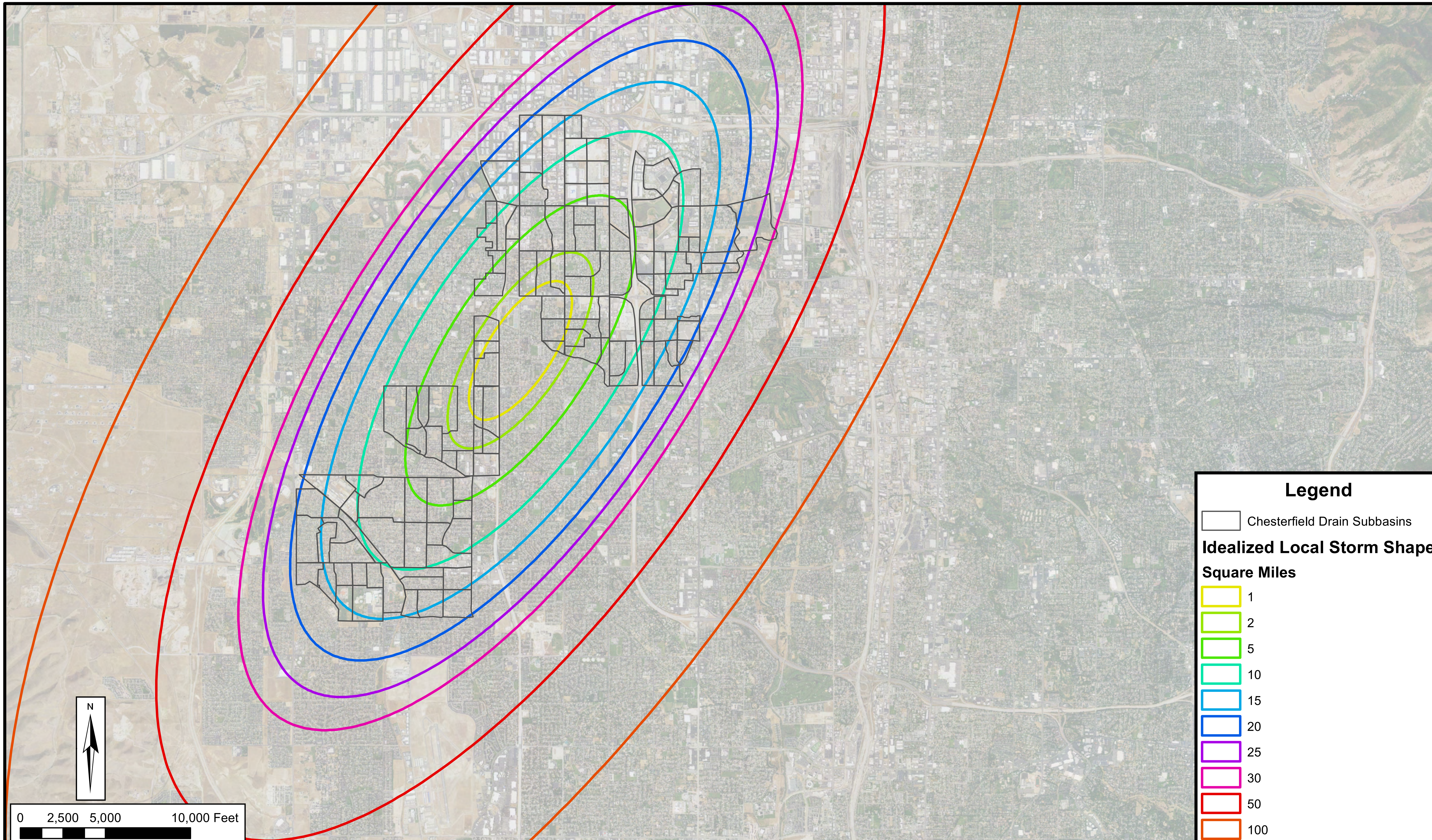
Salt Lake County Flood Control

Kearns Chesterfield Drain
Alignment and Tributary Area

FIGURE
X-1



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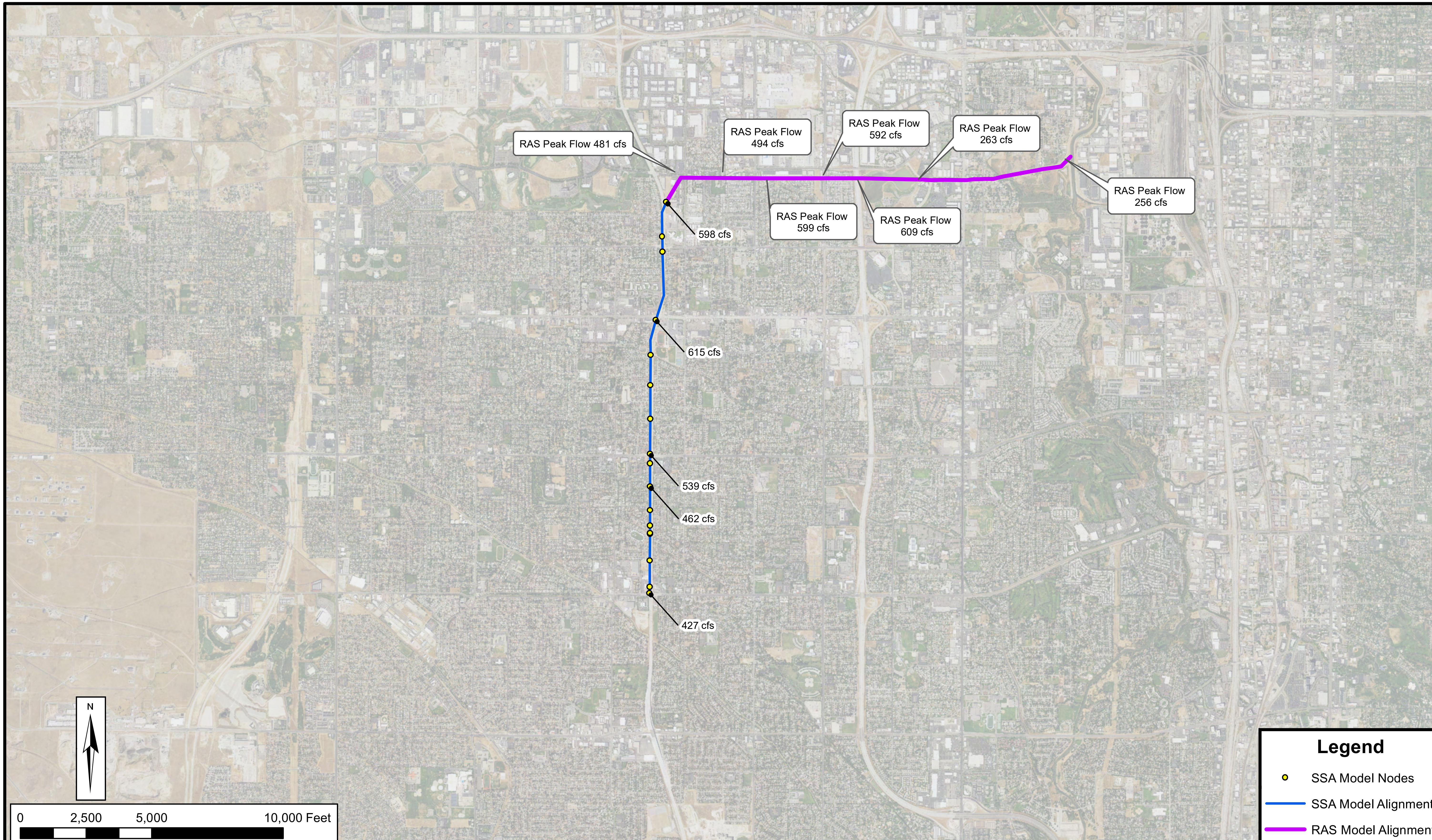
Salt Lake County Flood Control

Kearns Chesterfield Drain Idealized Storm Cell

FIGURE X-2



HANSEN ALLEN & LUCE INC ENGINEERS	**Salt Lake County Flood Control**	**Kearns Chesterfield Drain** **10-Year Modeling Results and Extents**	**FIGURE X-3**

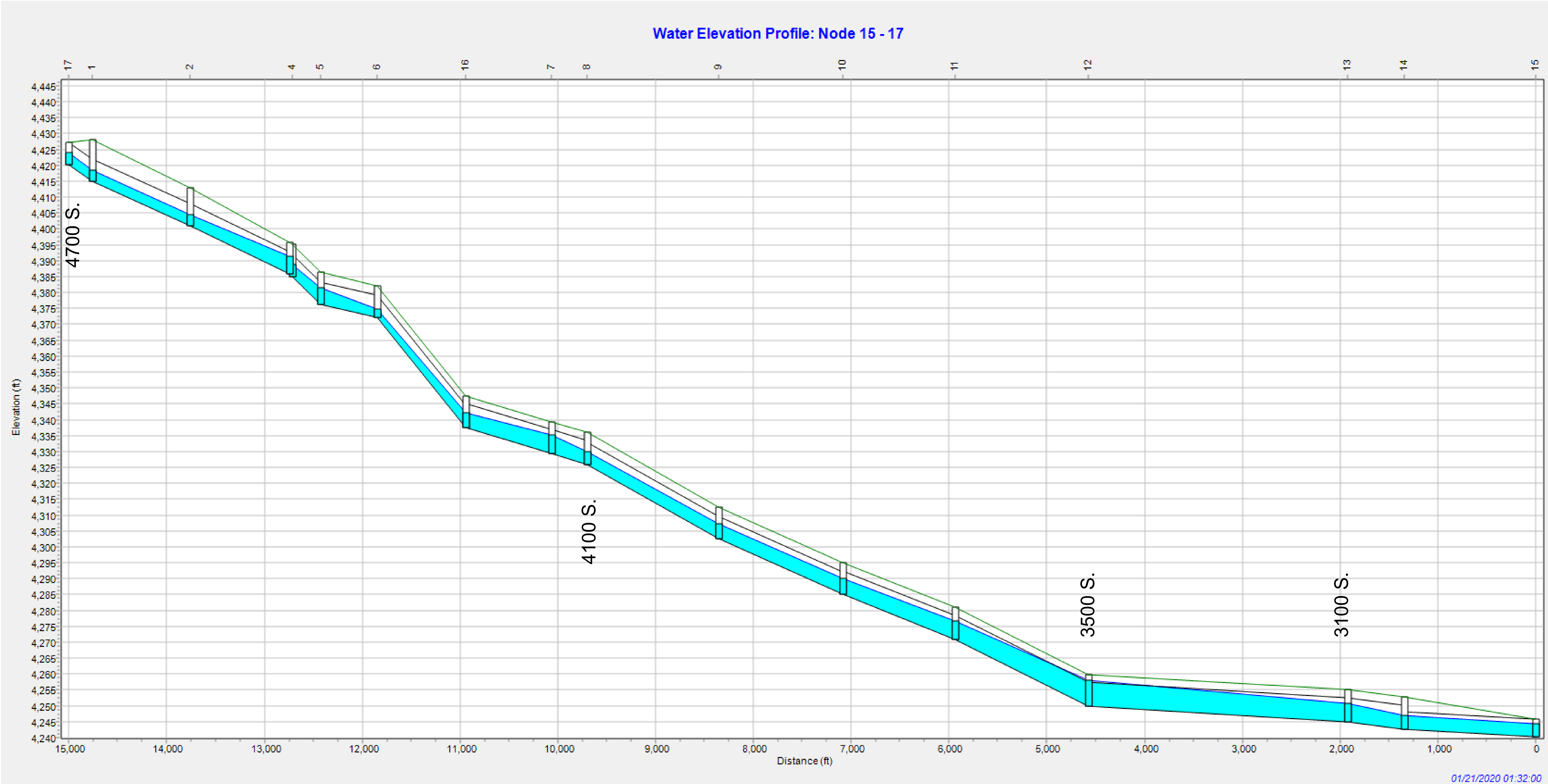


Figure X-4. Piped Section of Kearns Chesterfield Drain Profile.

TABLE X-1. Kearns-Chesterfield Drain Piped Section Computed Maximum Depths.

Node	Max Computed Depth (ft)	Pipe Size In (ft)	Pipe Size Out (ft)
17	3.8	N/A	7 ft Dia.
1	3.7	7 ft Dia.	7 ft Dia.
2	3.5	7 ft Dia.	7 ft Dia.
3	5.4	7 ft Dia.	6 x 6 Box
4	3.6	6 x 6 Box	7 ft Dia.
5	5.3	7 ft Dia.	7 ft Dia.
6	2.6	7 ft Dia.	7 ft Dia.
16	4.6	7 ft Dia.	7.5 ft Dia.
7	5.8	7.5 ft Dia.	7.5 ft Dia.
8	4.1	7.5 ft Dia.	7 ft Dia.
9	4.7	7 ft Dia.	7 ft Dia.
10	5.2	7 ft Dia.	7.5 ft Dia.
11	5.7	7.5 ft Dia.	(1) 7.5 x 6 Box
12	8.1	(1) 7.5 x 6 Box	(2) 7.5 x 6 Box
13	5.8	(2) 7.5 x 6 Box	(2) 7.5 x 6 Box
14	4.4	(2) 7.5 x 6 Box	(2) 5.5 x 10 Box
15 (Outlet)	3.9	(2) 5.5 x 10 Box	Open Channel

The HEC-RAS model as noted previously takes the output from the SSA model as the inflow hydrograph at the upstream end of the model. The model passes through Decker Lake which is modeled as a storage area with the Storage-Elevation curve being extracted from the 2013-2014 Wasatch Front Lidar dataset. As shown on Figure X-3 Decker Lake is anticipated to attenuate peak flows from about 610 cfs down to approximately 260 cfs. The downstream boundary condition was the anticipated 100-year water surface elevation from the Jordan River. The profile for the reach upstream of Decker Lake and the reach downstream of Decker Lake are shown in Figure X-5 and Figure X-6 respectively.The computed WSE for the segment upstream of Decker Lake reaches the top of most of the long culverts with freeboard on the banks generally greater than 1 foot.

EXISTING SYSTEM CAPACITY

The Kearns-Chesterfield Drain was found to have adequate capacity for the calculated 10-year event.

DEFICIENCIES

No deficiencies were identified for the Kearns-Chesterfield Drain based on the modeling assumptions presented. However, we recommend that an inspection of the pipe be completed due to its age.

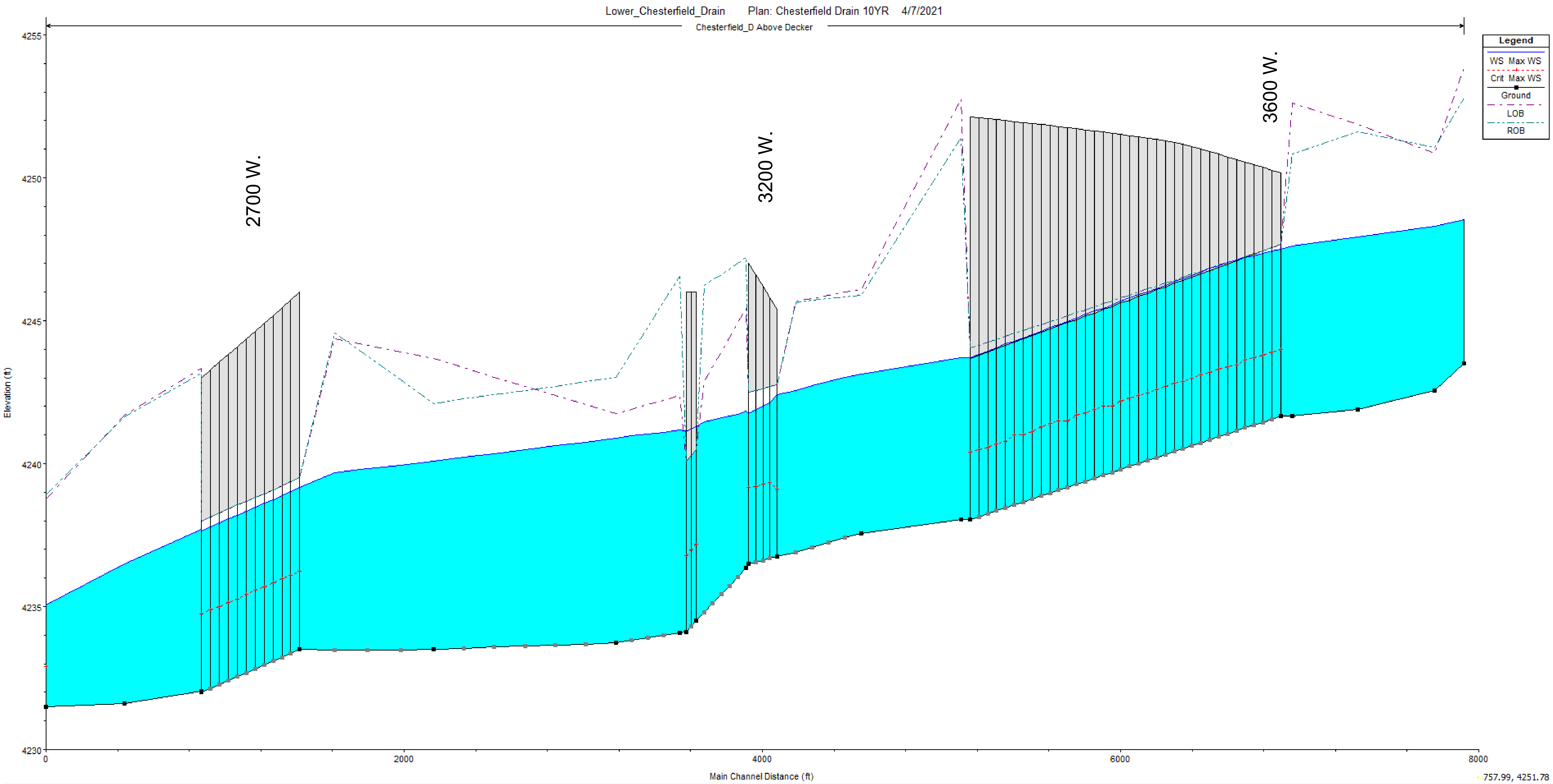


Figure X-5. Kearns Chesterfield Drain Profile from Piped Section Outfall to Decker Lake.

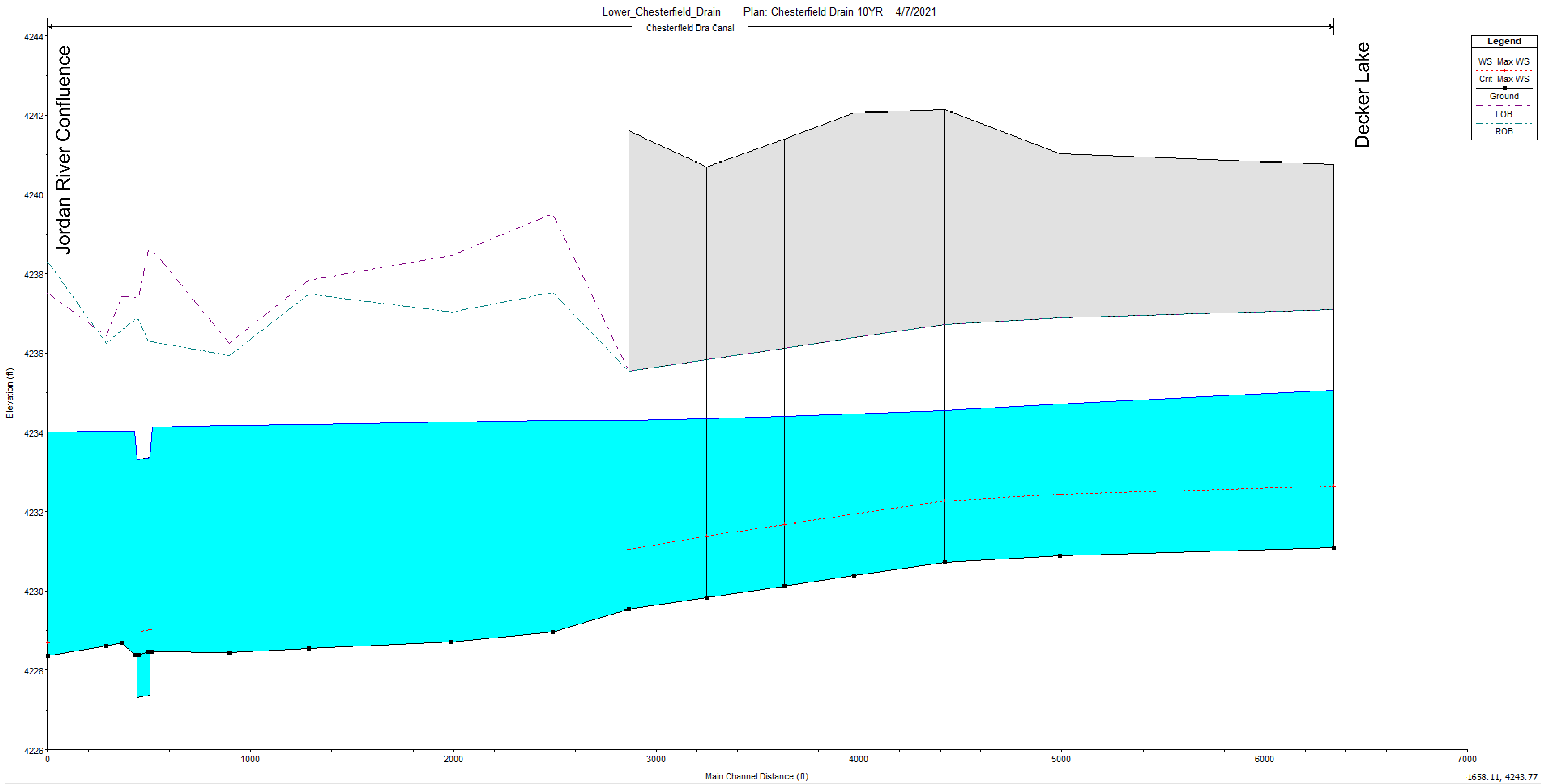


Figure X-6. Kearns Chesterfield Drain Profile from Decker Lake to Jordan River.

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APPENDIX

LEE DRAIN IMPROVEMENTS SUMMARY

May 2021

CROSSING	STA	Length (ft)	Span x Height	COST EST.
Gladiola Street	18,600	117	5 x 4	$132,000
Brighton & North Point Canal	16,879	58	5 x 4	$71,000
Access Road	16,700	58	5 x 4	$71,000
Rail Road	13,200	120	8 x 5	$170,000
Bangerter HWY	12,900	200	8 x 5	$272,000
Gramercy Road	12,200	200	10 x 5	$358,000
Rail Road	8,400	97	12 x 5	$210,000
4800 West	7,000	170	12 x 5	$356,000
5070 West	5,300	170	12 x 6	$365,000
5500 West	2,500	170	16 x 6	$551,000
5600 West	1,800	170	16 x 6	$551,000
		SUBTOTAL		$3,107,000
		Contingency & Eng		$932,000
		Total Road Crossings		**$4,039,000**

CHANNEL IMPROVEMENTS

Item	Quantity	Units	Unit Cost	Total
Earthwork	34,020	CY	$10.70	$364,000
Revegetation & Erosion Protection	49,300	SY	$9.00	$444,000
		SUBTOTAL		$808,000
		Contingency & Eng		$242,000
		Total Channel Work		**$1,050,000**

TOTAL PROJECT ESTIMATE	$5,089,000

LEE CREEK RECOMMENDED LAND ACQUISITION SUMMARY

December 2021

As described in the body of the report there is detention storage that is critical to the Lee Creek flood control facility on the upstream side of Highway 201 (see Figure IV-5 in the main body of the report). Much of the land is already owned by UDOT, however we estimate another 2.5 acres of land should be acquired in order to maintain the storage volume required for the anticipated future runoff.

LAND COST ESTIMATE

Facility	Quantity	Units	Unit Cost	Total
Lee Creek	2.5	acres	$370,000	$925,000

RITER CANAL IMPROVEMENTS AND LAND ACQUISITION SUMMARY

December 2021

As described in the body of the report there is some natural detention storage that is critical to the Riter Canal flood control facility about 2,700 feet downstream of 5600 West (see Figure VIII-5 in the main body of the report). All of this land is currently under private ownership. The existing footprint of the natural detention is approximately 11.5 acres. The overall footprint of a future detention facility could be decreased to reduce costs with some additional modifications to the site (i.e. excavation).

LAND COST ESTIMATE

Facility	Quantity	Units	Unit Cost	Total
Riter Canal	11.5	acres	$370,000	$4,255,000

Additionally, an overflow structure is planned on the Riter Canal that would flow into Lee Creek. A cost estimate for the overflow structure is provided below.

OVERFLOW STRUCTURE COST ESTIMATE

Facility	Quantity	Units	Unit Cost	Total
Riter Canal Overflow Structure	1	LS	$72,000	$72,000

COON AND HARKER CREEK IMPROVEMENTS SUMMARY
July 2021

The following is the cost estimate for increasing the storm drain capacity in 8000 West from 3100 South to the C-7 Ditch.

STORM DRAIN LOCATION	LENGTH (feet)	Unit Cost (36")	Total
3100 South to 2700 South	1393	$404	$562,772
2700 South to C-7 Ditch (about 2500 South)	2838	$404	$1,146,552
SUBTOTAL	4231		$1,709,324
Contingency and Engineering			$512,797
TOTAL ESTIMATED PROJECT COST			$2,222,121