Lower St. Vrain Watershed
Phase 2 Hydrologic Evaluation

Post September 2013 Flood Event

Prepared for:

Colorado Department of Transportation
Region 4 Flood Recovery Office

Prepared by:

707 17th Street, Suite 2400
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With Support from:

July 2015

July 1, 2015

We hereby affirm that this report and hydrologic analysis for the Lower St. Vrain Watershed (Phase 2) was prepared by us, or under direct supervision, for the owners thereof, in accordance with the current provisions of the Colorado Floodplain and Stormwater Criteria Manual, and approved variances and exceptions thereto.

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EXECUTIVE SUMMARY

In late summer 2013, the Colorado Front Range experienced an extensive rainstorm event spanning approximately ten days from September 9th to September 18th. The event generated widespread flooding as the long-duration storm saturated soils and increased runoff potential. Flooding resulted in substantial erosion, bank widening, and realigning of stream channels; transport of mud, rock and debris; failures of dams; landslides; damage to roads, bridges, utilities, and other public infrastructures; and flood impacts to many residential and commercial structures. Ten fatalities were attributed to the floods.

During and immediately following the rainstorm event, the Colorado Department of Transportation (CDOT) engaged in a massive flood response effort to protect the traveling public, rebuild damaged roadways and bridges to get critical travel corridors open again, and engage in assessments and analyses to guide longer term rebuilding efforts. As part of this effort, CDOT partnered with the Colorado Water Conservation Board (CWCB) to initiate hydrologic analyses in several key river systems impacted by the floods. The work was contracted to three consultant teams led by the following firms.

Boulder Creek, Little Thompson River	CH2M HILL
Big Thompson River, St. Vrain Creek, Lefthand Creek	Jacobs
Coal Creek, South Platte River	URS

The purpose of the analyses is to ascertain the approximate magnitude of the September flood event in key locations throughout the watershed and to prepare estimates of peak discharge that can serve to guide the design of permanent roadway and other infrastructure improvements along the impacted streams. These estimates of peak discharges for various return periods will be shared with local floodplain administrators for their consideration in revising or updating any current regulatory discharges.

The primary tasks of the hydrologic analyses include:

1. Estimate peak discharges that were believed to have occurred during the flood event at key locations along the study streams. Summarize these discharges along with estimates provided by others in comparison to existing regulatory discharges. Document the approximate return period associated with the September flood event based on current regulatory discharges.
2. Prepare rainfall-runoff models of the study watersheds, input available rainfall data representing the September rainstorm, and calibrate results to provide correlation to estimated peak discharges.
3. Prepare updated flood frequency analyses using available gage data and incorporate the estimated peak discharges from the September event.
4. Use rainfall-runoff models to estimate predictive peak discharges for a number of return periods based on rainfall information published by the National Oceanic and Atmospheric Administration (NOAA) [NOAA Atlas 14, Volume 8, Updated 2013]. Compare results to updated flood frequency analyses and unit discharge information and calibrate as appropriate.

The hydrologic analyses were divided into two phases of work. Phase 1 focused on the mountainous areas in the upper portion of the watersheds, extending from the upper divides of the Big Thompson River, Little Thompson River, St. Vrain Creek, Lefthand Creek, Coal Creek, and Boulder Creek watersheds to the mouth of their respective canyons. The Phase 1 analyses have been documented in six reports with the following titles and dates.

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2. Little Thompson River Hydrologic Analysis Final Report, August 2014
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4. Hydrologic Evaluation of the Lefthand Creek Watershed, August 2014, revised December 2014
5. Coal Creek Hydrology Evaluation, August 2014
6. Boulder Creek Hydrologic Evaluation Final Report, August 2014

Copies of these Phase 1 reports can be downloaded from the CWCB website at the following link:

http://cwcb.state.co.us/water-management/flood/pages/2013floodresponse.aspx

Phase 2 of the hydrologic analyses focuses on the plains region of the Big Thompson River, Boulder Creek, Little Thompson River, and St. Vrain Creek from the downstream limit of the Phase 1 studies at the mouth of the canyons to the downstream confluences of the watersheds with their respective receiving streams. The hydrologic analyses were contracted to two consultant teams led by the following firms:

Boulder Creek, Little Thompson River	CH2M HILL
Big Thompson River, St. Vrain Creek	Jacobs

Phase 2 hydrologic analyses for each of the watersheds include flows from the original Phase 1 watersheds, as appropriate; the downstream reach of the Big Thompson River was modeled to include flows from the Little Thompson River. Likewise, the downstream reach of St. Vrain Creek includes flows from Lefthand Creek and Boulder Creek, with Boulder Creek in turn receiving flows from Coal Creek.

This report documents the Phase 2 hydrologic evaluation for the St. Vrain watershed from Lyons to the South Platte River. Figure 1 in Section 1.2 of the report provides an overview map of the study area.

Prior to September 2013, the last major flooding event on St. Vrain Creek was in 1978, with the flood of 1921 having the most damaging effect on Longmont. In 1981, the effective regulatory flow rates documented by the Federal Emergency Management Agency (FEMA) in the 2012 Flood Insurance Study (FIS) for Boulder County were developed by the U.S. Army Corps of Engineers. The effective peak discharges were developed using the EPA SWMM rainfall/runoff model and the Missouri River Division version of Harder's diffusion routing model.

In the current evaluation, a rainfall-runoff model was developed to transform ground-calibrated rainfall information for the September storm to stream discharge using the HEC-HMS hydrologic model (USACE, 2010). The hydrologic model was calibrated through adjustment of model input parameters that represent land cover, soil conditions and channel routing characteristics. A systematic approach was taken in the calibration process to ensure a consistent method was used throughout all of the watersheds studied. The goal was to obtain the best overall fit to the majority of the peak discharge estimates rather than try to match them all individually at the expense of calibration parameters being pushed beyond a reasonable range. The systematic approach prevents individual basins in the model from being biased toward unique occurrences such as levee breaches, split flows, or irrigation system impacts that may have been associated solely with this particular storm event. Table ES-1 provides a comparison of modeled peak discharges to peak discharges observed during the September 2013 Flood in the St. Vrain Phase 2 study area.

Table ES-1. Comparison of Modeled Discharges to Observed Discharges

Location	Observed 2013 Discharge (cfs)	Modeled 2013 Discharge (cfs)	Percent Difference
St. Vrain Creek above Airport Road	14,000	14,100	1%
St. Vrain Creek below Highway 287	14,500	15,000	3%
St. Vrain Creek below Lefthand Creek	18,500	18,700	1%
St. Vrain Creek at Interstate 25	23,500	23,300	- 1%
St. Vrain Creek at State Highway 66	23,000	23,900	4%
St. Vrain Creek at County Road 34	27,000	23,900	- 11%
Lefthand Creek at 63rd Street	7,000	7,210	3%
Lefthand Creek at Diagonal Highway	8,700	5,930	- 32%
Lefthand Creek at Hwy 287	5,000	5,090	2%
Lefthand Creek at St. Vrain Creek	4,800	4,800	0%

Loss parameters in the rainfall-runoff model were then individually adjusted using a runoff to rainfall ratio for each basin to provide an overall best fit with the estimated September peak discharges based on the peak 24 hours of the September rainfall rather than the entire multi-day storm. This was to prepare the model for developing predictive estimates of 10, 4, 2, 1, and 0.2 percent annual chance peak discharges (10-, 25-, 50-, 100-, and 500-year storm events) based on a 24-hour Soil Conservation Service (SCS) Type II storm distribution and the recently released 2014 National Oceanic and Atmospheric Administration (NOAA) Atlas 14 rainfall values. It should be noted that in general, the model focuses on peak discharge estimation along the main stem channels within relatively large watershed areas. Individual basins may produce greater discharges if divided into smaller areas or evaluated using shorter, more intense rainstorms. However, the larger basins and longer duration are appropriate for the major tributary peak discharges.

The predictive model peak discharges for the various return periods were compared to the results of an updated flood frequency analyses for St. Vrain Creek and Lefthand Creek, as well as to current regulatory discharges. This information is shown in Figure ES-1 and Table ES-2 for the 100-year event. Figure ES-1, including legend abbreviations, is discussed in more detail on page 33; however, several observations can be made:

1. Compared to the modeled discharges, more scatter is associated with the current regulatory discharges and flood frequency analysis results.
2. The regulatory discharges on St. Vrain Creek below the confluence with Boulder Creek (far right side) start to drop whereas the predictive model continues the linear trend.

Figure ES-1. Comparison of 100-year Discharges in the St. Vrain Watershed

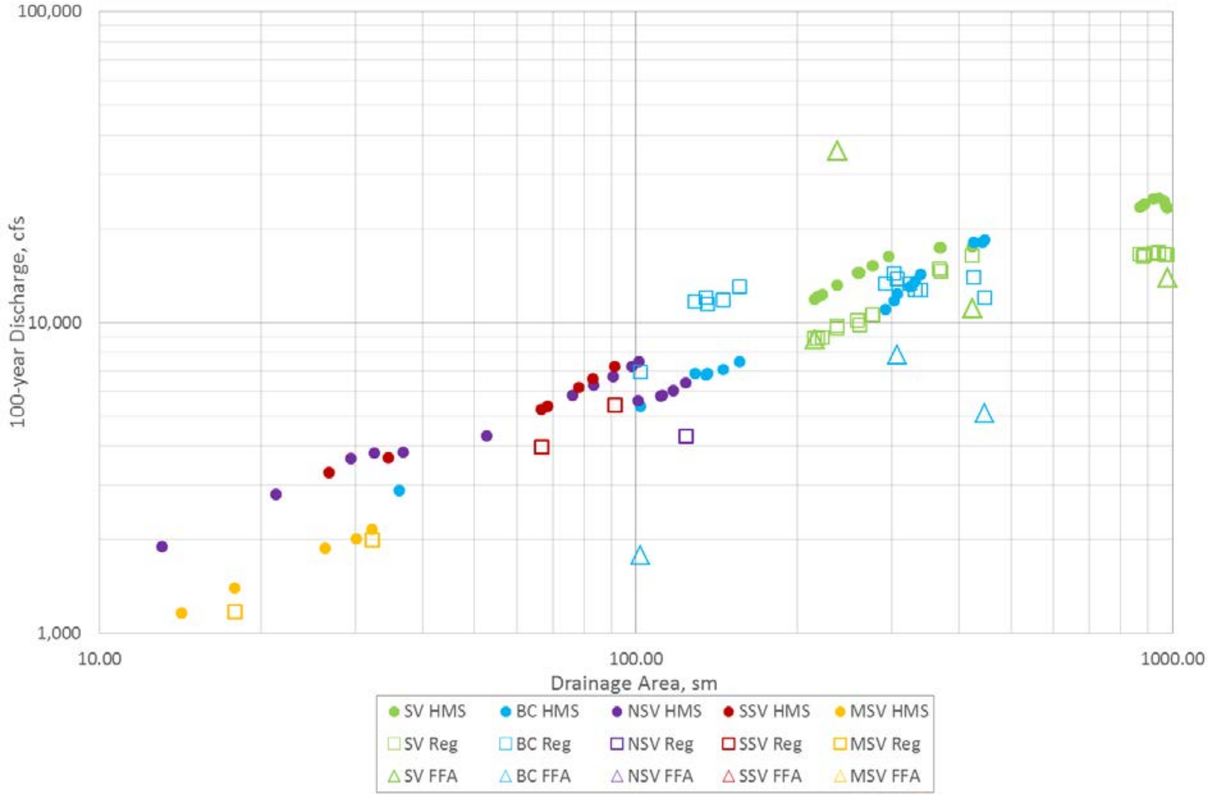


Table ES-2. 100-year Modeled Peak Flows Compared to Current Regulatory Discharges

Location	Current Regulatory Discharge (cfs)	Modeled Discharge (cfs)	Percent Difference
St. Vrain Creek at Hwy 36 below Lyons	8,880	12,100	+36%
St. Vrain Creek at Airport Road	9,580	13,200	+38%
St. Vrain Creek at Highway 287	10,580	15,200	+44%
St. Vrain Creek below Lefthand Creek	14,850	17,400	+17%
St. Vrain Creek above Boulder Creek	16,440	17,500	+6%
St. Vrain Creek at Interstate 25	16,510	24,100	+46%
St. Vrain Creek at State Highway 66	16,530	25,100	+52%
St. Vrain Creek at County Road 34	16,560	24,600	+49%
St. Vrain Creek at South Platte River	16,520	23,400	+42%
Dry Creek No. 1 at St. Vrain Creek	2,315	2,750	+19%
Spring Gulch (The Slough) at St. Vrain Creek	3,650	4,340	+19%
Lefthand Creek at Highway 36	6,700	5,820	- 13%
Lefthand Creek at 63rd Street	6,600	5,990	- 9%
Lefthand Creek at Diagonal Highway	6,330	6,040	- 5%
Lefthand Creek at St. Vrain Creek	4,610	5,740	+25%
Dry Creek No. 2 at St. Vrain Creek	2,600	4,920	+89%
Boulder Creek at St. Vrain Creek	12,000	18,500	+54%

The assumptions and limitations of various hydrologic methodologies used for development of the current regulatory discharges and for those used in this study were closely reviewed, compared, and contrasted. Based on this evaluation, the results of the current rainfall-runoff model using the 24-hour NOAA rainfall are viewed as suitable for use by CDOT in the design of permanent roadway improvements along St. Vrain Creek. In addition, the results of this modeling effort will be made available to local agencies for their consideration in revising discharges currently used for regulatory purposes. As described below, the rainfall/runoff model results better reflect the peak discharges in North St. Vrain Creek (Phase 1) and the overlap in hydrographs at the Boulder Creek confluence. Therefore it is recommended that the model results be considered for adoption as the updated regulatory peak discharges along St. Vrain Creek. It should be noted that this study was focused on peak discharge estimation in St. Vrain Creek and Lefthand Creek and was not developed with the intention of replacing regulatory values in the smaller tributaries. Additional analysis is recommended for smaller tributaries to evaluate shorter, more intense storms.

The 35 to 45 percent difference in 100-year peak discharges between Lyons and Longmont can be attributed to the fact that the current regulatory peak discharges were based on the assumption that Button Rock Dam would store runoff from North St. Vrain Creek and this tributary area was not included in the original model used to develop peak discharges. The 2013 flood is evidence that this assumption was not conservative enough and that significant peak discharges from the reservoir can occur causing flood damage downstream. In contrast, the predictive model developed as part of this study only accounts for attenuation of peak discharges as they pass through the Button Rock Dam Spillway, conservatively assuming the reservoir is full prior to the start of the storm.

The 40 to 50 percent difference in 100-year peak discharges downstream of the Boulder Creek confluence can be attributed to the fact that the current regulatory peak discharges were based on a 6-hour storm over the entire St. Vrain watershed (including Boulder Creek). The current regulatory peak discharges upstream and downstream of Boulder Creek are essentially identical which indicates that the Boulder Creek 6-hour hydrograph peak does not overlap at all with the St. Vrain Creek 6-hour hydrograph peak. This is largely because a shorter, more intense rainfall produces a tall, narrow discharge hydrograph which is less likely to overlap with other downstream discharge hydrographs in the model. In contrast, the predictive model developed as part of this study used a 24-hour storm over the entire St. Vrain watershed (including Boulder Creek). The longer duration storm produces peak discharge hydrographs with a much broader shape and more potential to overlap other hydrographs downstream. In the case of the Boulder Creek confluence, the predictive model resulted in a combined peak discharge that was approximately 65 percent of the direct sum of the two tributary peak discharges. This indicates that the two peak discharge hydrographs overlapped but that the instantaneous peak discharges were offset slightly. The overlap in hydrographs is further supported by the fact that both watersheds have relatively similar travel times at this location and that in the 2013 Flood the calibrated model matched the rising limb of the partial gage record at this location.

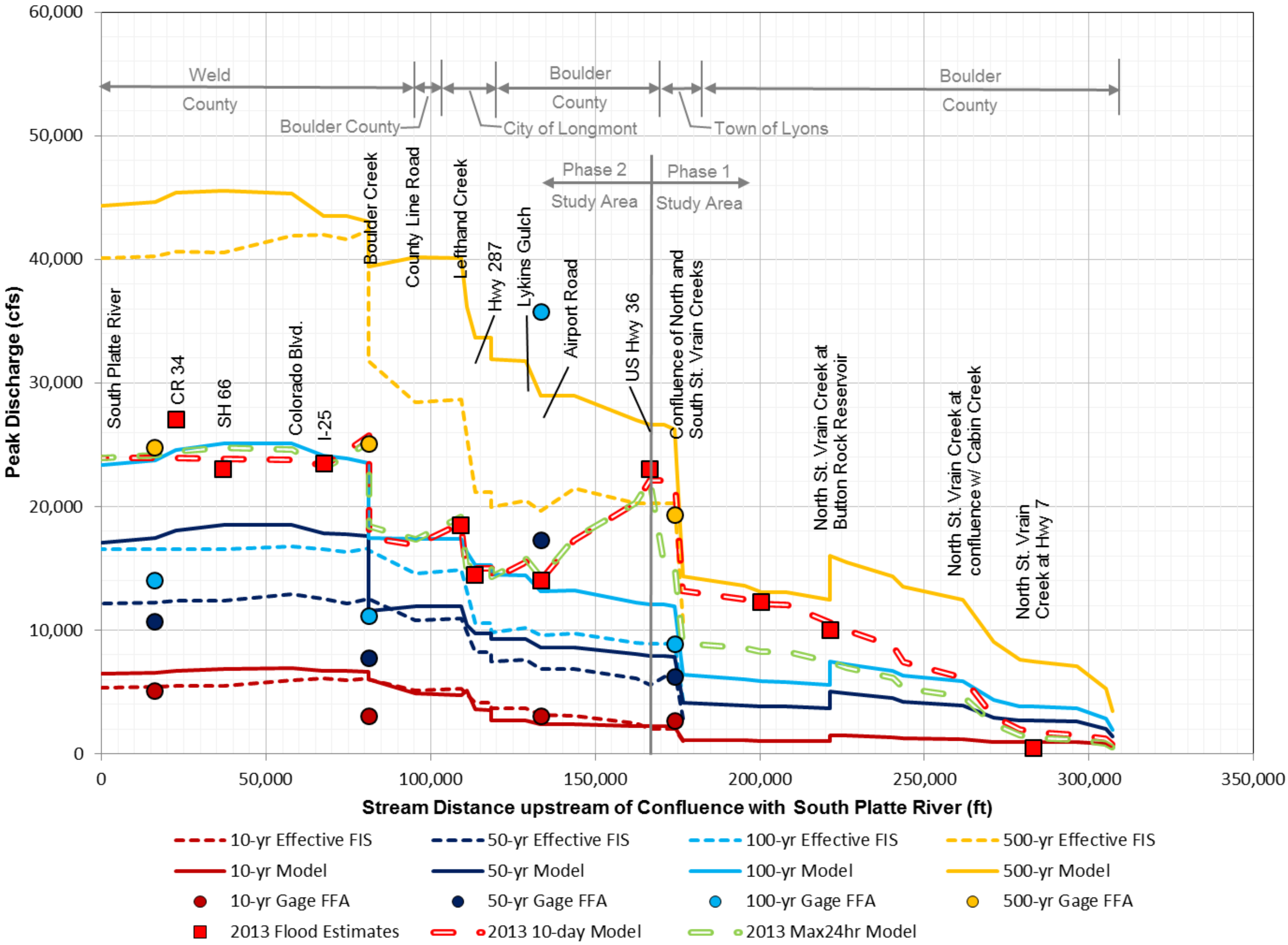
Based on the predictive model discharges for the return periods analyzed, as shown in Table ES-3 below, the peak discharge observed along Lower St. Vrain Creek (Phase 2) during the September 2013 flood event was approximately a 1 percent annual chance peak discharge (100-year storm) downstream of Lyons. Lower Lefthand Creek (Phase 2) experienced between a 0.2 percent annual chance peak discharge and a 2 percent annual chance peak discharge from upstream to downstream based on the predictive model.

Table ES-3. Estimate of September 2013 Peak Discharge Recurrence Interval based on Model Results

Location	Drainage Area (mi^2)	Measured Discharge (cfs)	Annual Chance Peak Discharge (cfs)					Estimated Recurrence Interval (yr)
			10%	4%	2%	1%	0.2%	
St. Vrain Creek at Hwy 36 Bridge (D-15-I)	218	23,000	2,200	4,860	7,950	12,100	26,600	100 to 500
St. Vrain Creek at Airport Road	237	14,000	2,360	5,280	8,570	13,200	29,000	~ 100
St. Vrain Creek at Highway 287	276	14,500	3,590	5,990	9,720	15,200	33,700	~ 100
St. Vrain Creek below Lefthand Creek	368	18,500	4,740	7,370	11,900	17,400	40,100	~ 100
St. Vrain Creek at Interstate 25	889	23,500	6,740	11,900	17,800	24,100	43,500	~ 100
St. Vrain Creek at State Highway 66	942	23,000	6,840	12,400	18,500	25,100	45,500	~ 100
St. Vrain Creek at County Road 34	965	27,000	6,710	12,100	18,100	24,600	45,400	~ 100
Lefthand Creek at 63rd Street	63	7,000	1,510	2,840	4,250	5,990	11,800	100 to 500
Lefthand Creek at Diagonal Highway	69	8,700	1,400	2,820	4,270	6,040	11,800	100 to 500
Lefthand Creek at Highway 287	72	5,000	1,380	2,640	4,060	5,810	11,600	50 to 100
Lefthand Creek at Ken Pratt Blvd.	72	4,800	1,370	2,580	3,990	5,740	11,400	50 to 100

Figure ES-2 provides a summary of the hydrologic evaluation in the form of peak discharge profiles for St. Vrain Creek from the headwaters to the confluence with the South Platte River (Phases 1 and 2). The figure includes 2013 peak discharge estimates, updated flood frequency analysis results, current regulatory peak discharges, and calibrated model peak discharges. A detailed discussion of the information presented on the figure is provided in Section 3.0 of the report. A larger version of Figure ES-2 is provided in Appendix D.6.

Figure ES-2. Peak Discharge Profiles for St. Vrain Creek and North St. Vrain Creek



1.0 BACKGROUND

1.1 Purpose and Objective

In late summer 2013, the Colorado Front Range experienced an extensive rainstorm event spanning approximately ten days from September 9th to September 18th. The event generated widespread flooding as the long-duration storm saturated soils and increased runoff potential. Flooding resulted in substantial erosion, bank widening, and realigning of stream channels; transport of mud, rock and debris; failures of dams; landslides; damage to roads, bridges, utilities, and other public infrastructures; and flood impacts to many residential and commercial structures. Ten fatalities were attributed to the floods.

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This report documents the Phase 2 hydrologic evaluation for the St. Vrain watershed from Lyons to the South Platte River.

1.2 Project Area Description

St. Vrain Creek is a perennial stream that drains the east slope of the Continental Divide by way of North, Middle and South St. Vrain Creeks in Boulder County. From the confluence of North and South St. Vrain Creeks at the Town of Lyons, St. Vrain Creek flows easterly through agricultural lands with numerous gravel pits adjacent to the channel before passing through the City of Longmont. Downstream of Longmont, St. Vrain Creek enters Weld County and the area becomes agricultural again with several more gravel pits adjacent to the channel from County Line Road to Colorado Blvd. At the confluence with Boulder Creek near Highway 119, St. Vrain Creek turns in a northeasterly direction and flows under I-25 to its confluence with the South Platte River. Elevations in the St. Vrain watershed range from 4,750 feet at the South Platte River to more than 14,000 feet at

Longs Peak. There are no significant flood storage reservoirs located in the Lower St. Vrain watershed (Phase 2 study area).

Figure 1 provides an overview map of the St. Vrain watershed and shows the boundary between the Phase 1 and Phase 2 study areas. St. Vrain Creek extending up North St. Vrain Creek is approximately 58 miles long and the watershed encompasses a total drainage area of approximately 978 square miles. Phase 2 of the St. Vrain watershed study extends from Highway 36 downstream of Lyons to the confluence with the South Platte River, a length of approximately 32 miles with slopes ranging between 0.1 percent and 0.9 percent. Of the total 978 square mile watershed, the Phase 2 St. Vrain study area only accounts for approximately 255 square miles. The remainder of the watershed is accounted for in the Phase 1 St. Vrain study area (218 square miles), Phase 1 Lefthand Creek study area (58 square miles) and the Boulder Creek watershed studied by CH2M Hill (447 square miles).

Several notable tributaries join St. Vrain Creek between Lyons and the South Platte River including from upstream to downstream: Lykins Gulch, Dry Creek No. 1, Spring Gulch (the Slough), Lefthand Creek, Dry Creek No. 2, Spring Gulch (Union Reservoir Ditch), Boulder Creek, and Godding Hollow (Tri-Area Drainageway).

Lykins Gulch originates near Red Hill, just west of Highway 36 in the foothills and flows east to its confluence with St. Vrain Creek at Golden Ponds Park and Nature Area. The majority of the 12 square mile watershed is pastureland with an unimproved channel. At the downstream end between Airport Road and St. Vrain Creek, the channel has recently been improved.

The Dry Creek No. 1 basin is located between the Lykins Gulch and Lefthand Creek drainage basins. Dry Creek No. 1 originates near the Table Mountain Antenna Field Sites just east of Highway 36 and flows east to its confluence with St. Vrain Creek. The drainage area of the Dry Creek No. 1 basin is approximately 14 square miles. Nearly all of the upper basin drainageways flow into irrigation reservoirs. The old outfall for Dry Creek No. 1 used to be near Highway 287. Now the channel is piped from Sunset Street along Price Road directly to St. Vrain Creek. The Dry Creek No. 1 floodplain is occupied by both commercial buildings and single-family dwellings from Sunset Street to the confluence with St. Vrain Creek. Flood protection measures along Dry Creek No. 1 include channel and culvert improvements from Sunset Street upstream to Grandview Meadows Drive. A small detention pond has been constructed upstream of Grandview Meadows Drive.

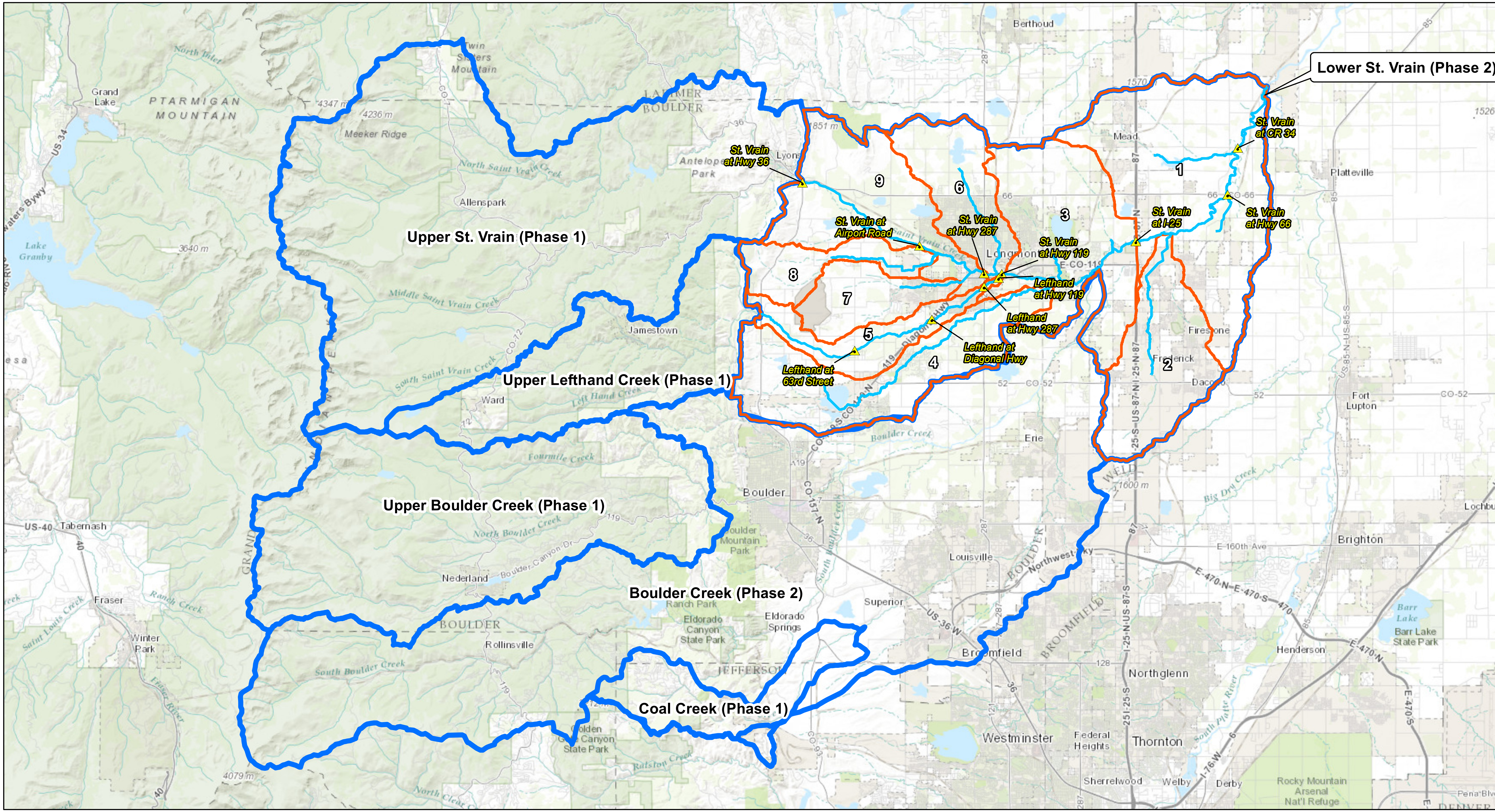
Spring Gulch (The Slough) has a drainage basin that extends north of Longmont to Terry Lake; however it only has a defined channel from 15th Avenue to the confluence with St. Vrain Creek. The Spring Gulch improved channel starts south of East 15th Avenue and continues as a concrete-lined channel through Spring Gulch Linear Park to Third Avenue, then down to St. Vrain Creek. Development in the Spring Gulch basin is pastureland north of Highway 66 and single-family dwellings south of Highway 66 to Third Avenue. Commercial and industrial buildings occupy the Spring Gulch floodplain from Third Avenue to the confluence with St. Vrain Creek. Various structural improvements such as grass- or concrete-lined channels, detention ponds, and outfall culverts have been implemented on Spring Gulch in the segment from Ninth Avenue to Third Avenue. However, these improvements are inadequate to contain even the 10-percent annual chance flood discharge. Loomiller Basin is a depression oriented in a northwest-southeast direction through central Longmont. Floodwater accumulates in the depression and subsequently drains into the Spring Gulch Basin near Third Avenue and Atwood Street. Loomiller Basin is totally developed with residential and commercial buildings. No flood protection measures have been taken in Loomiller Basin, except for using Loomiller Pond as a flood-storage facility. Various properties along the streets in Loomiller Basin have low retaining walls along the street frontage to prevent yard flooding.

The Lefthand Creek watershed extends approximately 30 miles eastward from its headwaters near the Continental Divide to its confluence with St. Vrain Creek in Longmont and encompasses a drainage area of approximately 72 square miles. Most of the watershed lies in the mountains and varies in elevations from 5,600 feet to 13,000 feet. The mountainous portion of the watershed (58 square miles) was studied in detail as part of the Phase 1 hydrologic analysis. The remainder of the watershed downstream of Highway 36 (14 square miles) lies in the high plains and has a channel length of approximately 11 miles resulting in a long, narrow shape. Lefthand Creek flows northeast through pastureland from Highway 36 to Pike Road. From Pike Road downstream to the confluence with St. Vrain Creek, the channel has been improved and flows through single-family and commercial developments.

Dry Creek No. 2 originates west of Boulder Reservoir and drains through Boulder Reservoir. Boulder Reservoir is not designed to provide flood control and overflow discharges continue northeast in Dry Creek No. 2 through the town of Niwot to St. Vrain Creek just downstream of County Line Road (Boulder-Weld county line). Dry Creek No. 2 downstream of Boulder Reservoir has a channel length of approximately 13 miles and flows through mostly pastureland with the exception of the short reach through Niwot. The total basin area is approximately 35 square miles.

Spring Gulch (Union Reservoir Ditch) originates near Walker Reservoir north of Longmont and flows southeast past Union Reservoir (Calkins Lake) to St. Vrain Creek just downstream of County Line Road and the Dry Creek No. 2 outfall. The basin has a drainage area of approximately 14 square miles. Union Reservoir is not designed to provide flood storage. The majority of the basin is pastureland but the western edge includes single-family houses in Longmont.

Boulder Creek encompasses approximately 447 square miles and joins St. Vrain Creek just upstream of Highway 119 near I-25. Boulder Creek has been studied separately by CH2M Hill and is described in more detail in a separate CDOT report. A brief memorandum documenting the Boulder Creek 10-day model calibration for the 2013 Flood is provided in Appendix E. It should be noted that memorandum in Appendix E is focused on the 10-day Flood whereas the actual Boulder Creek Hydrologic Evaluation report is focused on a 24-hour period and predictive storms.



Lower St. Vrain Study Area - Phase 2

Figure 1

CDOT Flood Recovery Hydrologic Evaluation

Service Layer Credits: Sources: Esri, HERE, DeLorme, TomTom, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community
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Miles

JACOBS

Legend

- Investigation Sites
- Tributaries
- St. Vrain Watershed
- Lower St. Vrain Major Basins

Major Basins

1. St. Vrain Creek (From I-25 to Confluence with South Platte River)
2. Godding Hollow (Headwaters to Confluence with St. Vrain Creek)
3. St. Vrain Creek (From Highway 287 to I-25)
4. Dry Creek No. 2 (Boulder Reservoir to Confluence with St. Vrain Creek)
5. Lower Lefthand Creek (Highway 36 to Confluence with St. Vrain Creek)
6. The Slough (Terry Lake to Confluence with St. Vrain Creek)
7. Dry Creek No. 1 (Headwaters to Confluence with St. Vrain Creek)
8. Lykins Gulch (Headwaters to Confluence with St. Vrain Creek)
9. St. Vrain Creek (From Highway 36 below Lyons to Highway 287)

The Godding Hollow drainage basin (Tri-Area Drainageway) originates near I-25 south of State Highway 52 and includes the City of Dacono and Towns of Frederick and Firestone. Godding Hollow parallels I-25 and drains north to its confluence with St. Vrain Creek near Colorado Blvd. The drainage basin encompasses an area of approximately 28 square miles. Land use in the basin is approximately 90 percent agricultural and 10 percent urban development. Drainage flows from agricultural fields south of Dacono run along the west edge of town before spilling into town along east-west streets. Drainage flows from the south and east of Frederick are intercepted by a levee and canal system and diverted to the west side of town, where the Tri-Area Drainageway parallels Colorado Blvd. to the north edge of town and continues south past the Town of Firestone before passing under Grant Avenue to reach Milavec Reservoir and eventually reaching Godding Hollow near the confluence with St. Vrain Creek.

1.3 Mapping

The United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's Geospatial Hydrologic Modeling Extension (HEC-GeoHMS), version 10.1 was used as the primary tool for delineating basins within the target watershed. HEC-GeoHMS is a public domain extension to Esri's ArcGIS Software and the Spatial Analyst extension. HEC-GeoHMS is a geospatial hydrology toolkit that allows the user to visualize spatial information, document watershed characteristics, perform spatial analysis, delineate basins and streams, construct inputs to hydrologic models, and print reports. This tool was decided upon for use because of its integration with the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) software and it was developed to use readily available digital geospatial information to construct hydrologic models more expediently than using manual methods.

HEC-GeoHMS was used to create background map files and basin model files. The basin model file contains hydrologic elements (basins) and their hydrologic connectivity (routing reaches). The basin area, length, length to centroid, and slope as well as the routing reach length and slope were determined using available geospatial data.

1.4 Data Collection

In order to facilitate the HEC-GeoHMS hydrologic modeling extension in Esri's ArcGIS software, several geospatial data sets were required. The HEC-GeoHMS extension uses a base digital surface elevation model to develop a series of raster data layers that are then used to delineate basin boundaries within the target watershed. A large amount of data is made available through the USDA/NRCS Geospatial Data Gateway (http://datagateway.nrcs.usda.gov/) and many of the necessary spatial data layers were downloaded from this website. Spatial data sets gathered from the USDA website included vector data files for 2013 Hydrologic Unit Code (HUC) boundaries, the 2012 National Hydrography Dataset (NHD), and the 2012 Gridded Soil Survey Geographic (gSSURGO) database. Raster data files were downloaded for Digital Line Graphs (DLG) and the 2001 National Land Cover Dataset. The base digital surface elevation model was created by the USGS as a 10 meter (1/3 arc second) Digital Elevation Model (DEM) shaded relief and Digital Raster Graphic (DRG) dataset. Raster and vector datasets for the study area were obtained through United States Geological Survey's (USGS's) National Map Seamless Server website, http://viewer.nationalmap.gov/viewer/. Roadway data sets developed by CDOT were also used. Digital aerial photography collected through the National Agriculture Imagery Program (NAIP) were downloaded and used for reference. The

National Flood Hazard layers for Boulder County and Weld County were obtained through FEMA to depict flood mapping. All of the datasets were used in the HEC-GeoHMS ArcGIS extension to define the parameters and variables required to accurately define and depict the sub-basin boundaries and routing reaches within the watershed.

1.5 Flood History

Unlike the September 2013 Flood, historical floods in Boulder County have mostly been due to snowmelt combined with heavy rainfall, although heavy rainfall, especially in the form of cloudbursts, have produced flooding in the past. Most of the major floods in Weld County have been caused by intensive rainstorms or cloudbursts. A brief summary of the St. Vrain Creek flood history obtained from the 2012 Boulder County Flood Insurance Study (FIS), the Preliminary 2013 Weld County FIS, and other supporting documents is provided here, for more detailed information please refer to the appropriate FIS.

As of 1981, the highest recorded peak flows at gages along St. Vrain Creek were 10,500 cfs at Lyons (June 22, 1941), 2,370 cfs below Longmont (May 17, 1978), and 11,300 cfs near the mouth (September 3, 1938). Notable floods along St. Vrain Creek occurred in 1844, June 1864, May 1876, May 1894, June 1914, July 1919, June 1921, August 1938, June 1941, June 1949, August 1951, May 1957, May 1958, May 1969, May 1973, and June 1976.

The historical flood having the most damaging effect in the Longmont area was the flood of June 2-7, 1921, which occurred when a long-duration rainstorm formed over the St. Vrain Creek basin with the heaviest rainfall accumulation downstream of the Lyons gaging station. Extensive damage was done to bridges, with severe erosion nearby to roads and along the channel banks. Public and private property damage amounted to $50,000.

Lefthand Creek experienced significant flooding in 1864, 1876, 1894, 1921, 1938, 1949, 1951, 1969, and 1973. On September 3, 1938 a peak discharge of 812 cfs was recorded at Highway 287. On June 4, 1949 a peak discharge of 1,140 cfs was recorded at the gage in the canyon upstream of Highway 36. Heavy rainfall with a long duration produced a large flood on May 7-8, 1969, with the primary damage being done to the South Pratt Parkway Bridge, which was ultimately destroyed by the floodwater. There is little known regarding floods of record other than what was stated concerning the gaged discharges. There are no existing stage data for the floods on Lefthand Creek later than May 1957. The largest flood on record was the one that occurred in June 1949.

The flood history for Boulder Creek is discussed in detail in the CDOT Report prepared by CH2M Hill.

The most significant floods of recent times along the Tri-Area Drainageway occurred in 1957, 1961, and 1975. On June 3, 1961, between 2 and 4 inches of rain fell in a 2-hour period and flows in the Tri-Area Drainageway swelled and flooded sections of Firestone, Evanston, Frederick, and Dacono. The rush of water caused the drainage ditch built along the south edge of Frederick to overflow, sending a wall of water through the town. This was similar to the flooding that occurred in 1957 and 1975.

2.0 HYDROLOGIC ANALYSIS

2.1 Previous Studies

The effective Boulder County Flood Insurance Study (FIS) was published by the Federal Emergency Management Agency (FEMA) on December 18, 2012. The Weld County FIS is still considered preliminary but was published by FEMA on May 31, 2013. Therefore, the information included in each of the FIS reports are up to date and there are no known relevant studies that occurred between the FIS effective dates and the September 2013 flood event. A summary of peak discharges from the FIS and supporting reports are shown in Table 1.

Previous studies pertaining to the Upper St. Vrain Creek and Upper Lefthand Creek watersheds are discussed in detail in the Phase 1 Hydrologic Evaluation Reports dated August 2014 and December 2014, respectively. This Phase 2 Report is dependent on the Phase 1 hydrology which serves as the upstream boundary conditions for the Phase 2 hydrology. The remainder of this section focuses on previous studies pertaining to the Phase 2 study area.

Frequency-discharge data for St. Vrain Creek in Longmont were originally based on information published in the June 1972 USACE Floodplain Information Reports Volume III for Lower St. Vrain. The 1-percent annual chance flood discharge on St. Vrain Creek was 10,200 cfs below Lyons, 11,200 cfs above the confluence with Lefthand Creek, and 13,200 cfs below the confluence with Lefthand Creek. The 0.2-percent annual chance flood discharges equaled the discharge for the standard project flood (generally 40% to 60% of PMF) as published in the 1972 Floodplain Information Reports. This relationship was based on a log-Pearson Type II analysis of peak runoff data recorded at gages on St. Vrain Creek near Lyons and Platteville in accordance with U.S. Water Resources Council Bulletin 15. The years of record varied from 79 years at the Lyons gage to 47 years at the Platteville gage.

The frequency-discharge data for St. Vrain Creek in Longmont was later updated by the USACE as documented in the April 1981 Floodplain Information, Flood Control, and Floodplain Management Plan prepared by Water Resources Consultants, Inc. The USACE used the EPA SWMM rainfall/runoff model and the Missouri River Division version of Harder’s diffusion routing model to generate updated peak discharge values from Lyons to the confluence with the South Platte River. Two different 6-hour storms were used in determining the peak discharges. The peak discharges above Boulder Creek were produced by centering a 6-hour storm over the St. Vrain basin above Boulder Creek, excluding the area above Button Rock Dam (109 square miles). From the mouth of Boulder Creek to the mouth of St. Vrain Creek, the peak discharges were developed by applying a 6-hour storm to the entire watershed.

The peak discharges for Dry Creek No.1 were taken from the April 1980 Floodplain Information and Flood Control Drainage Plan for Dry Creek No. 1 prepared by Water Resources Consultants, Inc. Discharge-frequency relationships in this study were developed using the EPA SWMM program and five irrigation reservoirs were modeled to account for their effect on peak flows. These reservoirs include Clover Basin Reservoir, Steele Lake No. 1, Swede Lake No. 1, Lagerman Reservoir, and Lefthand Reservoir. The reservoirs were assumed to be full to the level of the normal operation spillway.

Table 1. Select Peak Discharge Values from FIS Reports

Flooding Source and Location	Drainage Area (sq. mi.)	Peak Discharge (cfs)			
		10-yr	50-yr	100-yr	500-yr
St. Vrain Creek					
At Confluence with South Platte River	978	5,350	12,120	16,520	40,080
At Platteville Gage	976	5,410	12,200	16,540	40,230
Below Left Bank Tributaries	974	5,520	12,400	16,560	40,590
Below Right Bank Tributaries	935	5,920	12,900	16,760	41,900
At Colorado Boulevard	--	5,950	12,850	16,700	41,960
Below Idaho Creek (I-25)	903	6,070	12,500	16,510	41,960
Below Boulder Creek	879	6,110	12,500	16,630	42,400
Above Boulder Creek	386	6,010	12,500	16,440	31,790
at Boulder-Weld County Line (below Lefthand Creek)	351	5,250	10,950	14,850	28,670
Below Dry Creek No. 1 (Above Lefthand Creek)	265	4,110	8,240	10,580	21,200
Below Lykins Gulch	252	3,690	7,610	10,160	20,500
at 85th Street (Airport Road)	241	3,160	6,890	9,580	19,680
Just downstream of confluence of North St. Vrain Creek and South St. Vrain Creek	211	2,040	6,670	8,880	20,260
Dry Creek No. 1					
Just upstream of State Highway 119	--	340	845	1,170	2,127
Diversion across Nelson Road at Sunset St.	--	110	544	795	1,630
At Sunset Street	--	710	1,719	2,315	4,172
Downstream of Clover Basin Tributary	--	709	1,604	2,150	3,923
Dry Creek No. 1 (Old Channel)					
Upstream of confluence with St. Vrain Creek	--	320	627	802	1,199
Just downstream of State Highway 119	--	260	330	350	415
Spring Gulch (The Slough)					
At Confluence with St. Vrain Creek	--	1,950	3,150	3,650	4,200
Lefthand Creek					
At Confluence with St. Vrain Creek	72	520	2,480	4,610	10,320
At North 73rd Street	61.7	750	3,500	6,330	13,990
At North 55th Street	60.2	860	3,800	6,600	14,590
Above Foothills Highway (US Hwy 36)	56.4	1,035	4,145	6,700	14,990
Dry Creek No. 2					
At Highway 119 (Below Boulder Reservoir)	16.8	200	560	800	1,300
At North 107th Street (US Hwy 287)	30.2	900	1,900	2,600	4,295
At Confluence with St. Vrain Creek	34.0	900	1,900	2,600	4,240
Boulder Creek					
At Confluence with St. Vrain Creek	448	2,000	7,200	12,000	31,300
Tri-Area Drainageway					
At County Road 13	3.8	474	907	1,035	1,505

Discharge-frequency relationships for Spring Gulch were computed using the USACE HEC-1 Model. The effects of detention storage near State Highway 66 and at Long Peak Dam on Spring Gulch were studied and found to be insignificant for the magnitude of the floods considered. The portion of Spring Gulch Basin located north and east of Terry Lake was considered to be contained completely by Terry Lake.

Frequency-discharge data for Lefthand Creek in Longmont were originally based on information published in the January 1969 USACE Floodplain Information Reports Volume I for Lefthand Creek. The 1-percent annual chance flood discharge on Lefthand Creek was 4,250 cfs. The frequency-discharge data was updated in the December 1981 Floodplain Information Report for Lefthand Creek (Volume 1) prepared by Gingery Associates, Inc. The 1981 study used the EPA SWMM model with 6-hour rainfall depths to develop peak discharges. The 100-year flood discharges attenuated from 6,700 cfs at Highway 36 to 4,610 cfs at the confluence with St. Vrain Creek. The report states that the reduction of overbank storage by future development would tend to reduce the attenuation affect, thus causing an increase in peak discharges in the lower reaches of the study. The report also discusses the possibility for higher peak discharges downstream of Pike Road if the roadway embankment which temporarily impounds a considerable amount of flood water were to fail.

The peak discharges for Dry Creek No. 2 were taken from the June 1978 Flood Plain Information Report for Dry Creek prepared by the USACE. Discharge-frequency relationships in this study were developed using the EPA SWMM program and two irrigation reservoirs were modeled to account for their effect on peak flows. These reservoirs include Boulder Reservoir and West Gaynor Lake and were assumed to eliminate the peak discharges from the drainage area upstream of the reservoirs.

The hydrologic and hydraulic analyses for the City of Dacono, Town of Firestone and Town of Frederick were performed by Gingery Associates, Inc. in October 1977. Discharges were computed using SCS Technical Release 20.

2.2 September 2013 Peak Flow Estimates

CDOT and CWCB contracted with URS to obtain peak discharge estimates within the Phase 2 portion of the St. Vrain watershed following the September 2013 storm event. The technical memorandum summarizing the analysis is included in Appendix A. For the analysis, URS surveyed at least four cross-sections, collected bridge information for hydraulic modeling, and surveyed high-water markers at peak discharge estimate locations. The USACE's Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.1 model was used to construct a hydraulic model at each location. URS subsequently calibrated the model to high-water marks under subcritical and supercritical flow regimes. Generally, the subcritical flow regime was deemed more appropriate and used to develop peak discharge estimates at studied locations.

URS developed a total of ten peak discharge estimates for St. Vrain Creek and Lefthand Creek in the Phase 2 study area. The estimates on St. Vrain Creek were located at Airport Road, US Highway 287, Highway 119 (downstream of Lefthand Creek confluence), Interstate 25, State Highway 66, and County Road 34. The estimates on Lefthand Creek were located at North 63rd Street, Diagonal Highway, US Highway 287, and Highway 119. These sites were selected based on their location relative to: major tributary confluences, bridges that remained intact during the flood, and the quality of high water mark observations. Additional sites were evaluated, but were determined to be unsuitable for peak discharge estimation. The selected locations are shown on Figure 1 as Investigation Sites. The peak discharge estimates are presented on Table 4 later in the report.

These estimates were supplemented with recorded data from stream flow gages. Stream gages on St. Vrain Creek near Hover Road, Highway 119 below the Lefthand Creek confluence, and near the mouth recorded the stage and rising limb of the storm before they were washed out. This information was useful in calibrating the timing of the peak discharges in the model by comparing the rising limb of the hydrograph to the partial gage data. Although the peak discharges were not recorded, the timing of the maximum stage was used when available.

2.3 Updated Flood Frequency Analysis

Flood frequency analyses (FFA) were performed to supplement the hydrologic evaluation of St. Vrain Creek. The analyses followed the methods described in the document *"Guidelines for Determining Flood Flow Frequency"* published by the US Geological Survey on behalf of the Interagency Advisory Committee on Water Data, dated March 1982. This document is commonly known as *Bulletin 17B*.

Following the Bulletin 17B methods within the computer program HEC-SSP, Ayres Associates conducted the analyses using the annual peak flow records at the following four stream flow gages. Figure D.1 in Appendix D shows the location of these four gages.

St. Vrain Creek above Longmont near Hover Street

- CDWR Gage SVLONGCO (2002 – 2012)

St. Vrain Creek below Longmont near Highway 119 (Above Boulder Creek Confluence)

- USGS Gage 06725450 (1977 - 1982 & 1985 – 2012)
- CDWR Gage SVCBLOCO (operated by USGS)

St. Vrain Creek at Mouth near Platteville

- USGS Gage 06731000 (1905 - 1906 & 1927 – 1990)
- CDWR Gage SVCPLACO (1991 - 2012)

Lefthand Creek at Mouth near Longmont

- USGS Gage 06725000 (1927 - 1942 & 1954 – 1955)
- CDWR Gage LEFTLOCO (operated by USGS)

The St. Vrain Creek gage record above Longmont only has 11 annual peak flows. The 2013 peak flow of 14,000 cfs estimated by URS (discussed in Section 2.2) was added to the record. The 2013 flood peak is by far the largest in the record followed by a peak flow of only 1,090 cfs 2010. Based on the limited period or record and the wide confidence intervals from the FFA, this gage analysis is of no value for recuurence intervals greater than the 10-year event.

The St. Vrain Creek gage record below Longmont has 34 annual peak flows. The earliest is from 1977 and the latest is from 2012. Gaps in the record exist between 1982 and 1985. The 2013 flood was added to the data record with a peak flow of 18,500 cfs. The 2013 peak flow was estimated by URS as discussed in Section 2.2. The 2013 flood peak is the largest in the record, more than five times the next largest peak flow of 3,600 cfs in 1999.

Therefore, the 2013 peak flow was treated as an outlier in the FFA results (adjusted based on historical information).

The St. Vrain Creek gage record at the mouth has 87 annual peak flows. The earliest is from 1905 and the latest is from 2012. Gaps in the record exist between 1906 and 1927. The 2013 flood was added to the data record with a peak flow of 27,900 cfs. The 2013 peak flow was estimated by URS as discussed in Section 2.2. The 2013 flood peak is the largest in the record, more than 2.5 times the next largest peak flow of 11,300 cfs in 1938. Therefore, the 2013 peak flow was treated as an outlier in the FFA results (adjusted based on historical information).

The Lefthand Creek gage record at the mouth only has 18 annual peak flows. The earliest is from 1927 and the latest is from 1955. Gaps in the record exist between 1942 and 1954. The 2013 flood was added to the data record with a peak flow of 5,000 cfs. The 2013 peak flow was estimated by URS as discussed in Section 2.2. The 2013 flood peak is the largest in the record followed by a peak flow of 812 cfs in 1938. When the 2013 peak flow is treated as an outlier (peak adjusted based on historical information), the FFA results indicate very low peak flows for the different recurrence intervals. However, if the 2013 peak flow is not treated as an outlier, the FFA results are much more reasonable, especially when compared to current regulatory peak flows. Therefore, the 2013 peak flow was not treated as an outlier in this hydrologic analysis. Both sets of results are provided in Appendix B.

The hydrologic evaluation task force assembled by CDOT and CWCB for this effort conferred on the appropriate approach to take in the handling of stream flow gage data for flood frequency analysis. It was decided that to the extent practicable the methods recommended by Bulletin 17B should be followed. Stream gage analysis by Bulletin 17B methods requires as input the highest peak flow discharge for every year and the regional skew coefficient. The document recommends the use of a weighted skew coefficient that incorporates both the station skew and an appropriate general or regional skew. The regional skew coefficient has a strong influence on the resulting flood frequency relationship. It was agreed that the general skew coefficient map from Bulletin 17B would not be appropriate for this analysis because it is based on very old data. Therefore the approach initially taken in Phase 1 of this study was to develop a regression equation for the regional skew coefficient derived from an analysis of 24 gage stations along the northern Front Range. The peak discharge from the 2013 flood had only been determined for a fraction of the gage locations that were included in the regional skew analysis. In order to incorporate a large number of regionally appropriate gages into the analysis, it was decided to incorporate many gages for which the 2013 peak flood discharge had not yet been determined. For the sake of consistency, the 1976 flood and 2013 flood were omitted from all gages for the regression analysis in the St. Vrain watershed.

However, external review of the Phase 1 Draft Report led to comments that consideration should be given to revising the flood frequency analyses to simply use the station skew at each station rather than regionally weighting the skew coefficient. The comments arose from the observation that the analyses using the regional skew coefficients were yielding 100-year discharge values that were in some cases smaller than two or three of the flood peaks in the historical data. It was also observed that the difference between the station skew and regional skew coefficients exceeded 0.5 at some stations. Bulletin 17B warns that at such locations the regionally weighted skew approach can be inaccurate. Therefore, the flood frequency analyses presented in the Final Phase 1 Reports used the station skew only. The flood frequency analsyes in this Phase 2 report also used the station skew only.

The detailed input to, and output from HEC-SSP for both gages on St. Vrain Creek (Phase 2) using station skew only are included in Appendix B. The results are summarized in Table 2 below.

Table 2. Results of Flood Frequency Analysis for St. Vrain Creek

Exceedence Recurrence Interval (years)	St. Vrain Creek above Longmont (cfs)	St. Vrain Creek below Longmont (cfs)	St. Vrain Creek at Mouth (cfs)	Lefthand Creek at Mouth * (cfs)
2	419	935	1,664	156
5	1,343	1,938	3,414	474
10	2,993	3,053	5,086	916
50	17,250	7,716	10,649	3,332
100	35,722	11,146	13,990	5,486
200	73,229	15,908	18,060	8,834
500	186,890	25,097	24,789	16,145

* Lefthand Creek FFA results do not treat 2013 peak flow as an outlier.

Based on these FFA results, the 2013 Flood was between a 200-year and 500-year event at the St. Vrain Creek gage below Longmont (upstream of Boulder Creek confluence). These FFA results also indicate that the 2013 Flood was approximately a 500-year event at the St. Vrain Creek gage at the mouth. On Lefthand Creek at the gage near the mouth, the FFA results indicate the 2013 Flood was approximately a 100-year event. It should be noted that reliable flood-frequency relations are difficult to estimate when using short gage record lengths and when the contributing basins are heavily influenced by irrigation canals and reservoirs, particularly for semi-arid and arid basins in the western United States. The occurrence of high-outliers and low-outliers, mixed-population sources of flooding, non-stationarity (the effects of long-term variability on flood estimates), and other factors also contribute to uncertainty in flood-frequency estimates (Jarrett 2014).

2.4 Rainfall / Runoff Model for September 2013 Event

2.4.1 Overall Modeling Approach

A hydrologic analysis was performed on the St. Vrain watershed to evaluate and attempt to replicate the September 2013 Flood event along the Front Range. The September 2013 flood event was modeled using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) to calculate the peak runoff experienced during the flood. Calibrated HEC-HMS models were developed in Phase 1 of this hydrologic analysis for St. Vrain Creek upstream of Highway 36 at Lyons (St. Vrain Creek, North St. Vrain Creek, South St. Vrain Creek and Middle St. Vrain Creek) and for Lefthand Creek upstream of Highway 36 (Lefthand Creek, James Creek and Little James Creek). Similarly, a HEC-HMS model for the entire Boulder Creek watershed (Boulder Creek, South Boulder Creek, and Coal Creek) was developed by CH2M Hill. The model output, in the form of discharge hydrographs, from these three tributary models was then used as input to a separate model for the lower St. Vrain watershed (Lyons to the South Platte River).

Of the various hydrologic models accepted by FEMA, HEC-HMS version 3.5 was determined to be the best suited for modeling the rural mountainous watersheds included in the CDOT scope of work. During this study HEC-HMS version 4.0 was released, however version 3.5 was used to maintain consistency with the Phase 1 analysis. The primary reasons HEC-HMS was chosen are that it includes several different options to simulate the hydrologic response in a watershed including various infiltration loss methods (constant loss, exponential loss, CN method, Green-Ampt, Smith-Parlange, and soil moisture accounting), transform methods (kinematic wave and various unit hydrographs), and reach-routing methods (Modified Puls, Muskingum, Muskingum-Cunge, Lag, and Kinematic Wave). HEC-HMS also has a GIS interface (HEC-GeoHMS) which helped in obtaining the necessary model input parameters.

The Curve Number method was selected for infiltration losses due to its simplicity and the availability of soil and land cover data. However, as discussed in the Phase 1 reports, several other infiltration methods were evaluated to make sure the CN method was the most appropriate. For the transform method, the Snyder Unit Hydrograph was selected since it was developed in rural watersheds and is also the basis of the Colorado Unit Hydrograph Procedure (CUHP). The two required input parameters for the Snyder UH are lag time (Tlag) and peaking coefficient (Cp). These parameters were initially estimated from the subcatchment length, length to centroid, and slope as outlined in the CWCB Floodplain and Stormwater Criteria Manual. For channel routing the Muskingum-Cunge method with an 8-point cross-section was selected due to the irregular shape of the channel cross-sections and the recommendations provided in the CWCB Floodplain and Stormwater Criteria Manual.

After initial working models were developed in HEC-HMS using HEC-GeoHMS, as discussed in the following sections, the models were then calibrated to the peak discharge estimates derived from field investigations of high water marks. The first model developed was calibrated using 10-days of rainfall data from the 2013 Flood. As discussed in Section 2.5.3, a model was also developed and calibrated based on the maximum 24-hour period of rainfall to transition from the long duration 2013 Flood over to a standard 24-hour design storm. A third model was then developed to generate predictive peak discharges based on NOAA 24-hour rainfall depths. The following sections discuss the steps undertaken during the rainfall/runoff modeling process. Associated information is included in Appendix D, as described below.

2.4.2 Basin Delineation

The best available topographic data for watershed delineation were the 10-meter DEMs developed from USGS maps. HEC-GeoHMS uses DEMs to develop watershed boundaries and flow paths. Reaches were defined within the system based on a minimum tributary area of approximately two square miles. The upstream limits of the watershed are the Little Thompson River watershed to the north, the Phase 1 models for St. Vrain Creek and Lefthand Creek (extend to the Continental Divide), and the Boulder Creek watershed to the south. With the downstream limit of the study set at the confluence with the South Platte River, basins were delineated around all reaches and confluences. The overall watershed was divided into 38 basins ranging from 0.26 square miles to 18 square miles. Basins were manually subdivided where necessary in order to compare peak discharge estimates at investigation sites and stream gages with results from the hydrologic model. The ten peak discharge estimation locations are:

1. St. Vrain Creek at Airport Road (URS)
2. St. Vrain Creek at US Highway 287 (URS)
3. St. Vrain Creek at Highway 119 below Lefthand Creek (URS)
4. St. Vrain Creek at Interstate 25 (URS/CDOT)
5. St. Vrain Creek at State Highway 66 (URS)
6. St. Vrain Creek at County Road 34 (URS)
7. Lefthand Creek at North 63rd Street (URS)
8. Lefthand Creek at Diagonal Highway (URS)
9. Lefthand Creek at US Highway 287 (URS/Muller)
10. Lefthand Creek at Highway 119 near Mouth (URS)

2.4.3 Basin Characterization

The basin characteristics of the Lower St. Vrain watershed (Phase 2) consist mainly of agricultural and pasture lands with developed urban areas around Lyons, Longmont, Dacono, Frederick, Firestone, and Mead. The watershed topography generally slopes west to east with mild slopes. The individual basin slopes range from approximately 0.2 percent to 7.8 percent. Eight major tributary areas join St. Vrain Creek in the Phase 2 study area; Lykins Gulch, Dry Creek No. 1, Spring Gulch (The Slough), Lefthand Creek, Dry Creek No. 2, Spring Gulch (Union Reservoir Ditch), Boulder Creek, and Godding Hollow. The drainage area of the Phase 2 study area is approximately 255 square miles, whereas the total St. Vrain watershed encompasses approximately 978 square miles.

The CN values used for the hydrologic analysis were obtained from the TR-55 manual for various soil groups and land cover types. The curve numbers represent the four (4) hydrologic soil groups (A, B, C, and D) for various land cover types including, but not limited to:, mixed forest, shrub/scrub, herbaceous grasslands, pasture, rock outcroppings, developed land, and water bodies. A hydrologic condition of “good” was initially applied to all CN values. These individual soil group and land cover types were then compiled to create a CN lookup table. The soil type and land cover datasets were then merged in GIS using the union tool to create a single layer with polygons representing the intersections of the two datasets. The “Generate CN Grid” tool in HEC-GeoHMS then utilizes the CN lookup table and the merged soil type/land cover polygon layer to generate a “CN” field in the soil type/land cover attribute table. The basin delineation boundaries were then overlaid with the soil type/land cover polygon layer to calculate area-weighted CN values for each basin. The resulting area-weighted CN values ranged from approximately 40 to as high as 85. The CN method impervious percentage input value for each basin was set to zero because all impervious areas were accounted for in the area-weighted CN.

The Snyder Unit hydrograph transform method was utilized to determine the shape and timing of runoff hydrographs for each basin. The Snyder Unit hydrograph transform method requires two input parameters: peaking coefficient and standard lag time. A default peaking coefficient of 0.4 was initially selected for all basins as being representative of the watershed. The lag time was calculated using Equation CH9-510 and Table CH9-T505 in the CWCB Floodplain and Stormwater Criteria Manual. Default Kn values of 0.15 for evergreen forests and 0.10 for agriculture and heavy shrub/brush were used for the basin roughness factor. The remaining input parameters for the lag time equation include basin length (miles), length to basin centroid (miles), and average basin slope (feet per mile). These parameters were acquired using the HEC-GeoHMS program and the project DEM and DRG datasets. Appendix D.1 summarizes the final model input parameters.

2.4.4 Hydrograph Routing

The Muskingum-Cunge routing method was used to route the runoff hydrographs generated from each basin. The required input parameters for this method included: channel length (feet), channel slope (feet/feet), an 8-point cross-section to represent the channel width and side slopes, and Manning's n values for the channel and overbank areas. The length and slope of the channel reaches were acquired using the HEC-GeoHMS program and the 10-meter DEM and DRG datasets. Cross-section station-elevation data for the channel reaches was acquired from post-flood LIDAR mapping where available and supplemented with the 10-meter DEM and DRG datasets for overbank areas that exceeded the limits of the LIDAR mapping.

In many locations the cross-sections needed to be more than a mile wide in order to contain the 2013 flood extents. Cross-section data initially included several hundred points from the topographic data. These cross-sections points were then reduced to only eight points to appropriately reflect the channel and overbank areas. The Manning's n values were initially set to a default of 0.05 for the channels and 0.10 for the overbank areas. Appendix D.1 summarizes the final model input parameters.

2.4.5 2013 Rainfall Information

The rainfall data required for the meteorological component of the HEC-HMS model were obtained for the September, 2013 storm from Applied Weather Associates (AWA). The Storm Precipitation Analysis System (SPAS) was used to analyze and calibrate the rainfall. SPAS uses a combination of climatological basemaps and NEXRAD weather radar data that is calibrated and bias corrected to rain gage observations (considered ground truth) to spatially distribute the rainfall accumulation each hour over the entire domain of the storm. Therefore, SPAS through the use of climatological basemaps and weather radar data accounts for topography and locations of rain gages. For quality control, SPAS storm analyses have withheld some rain gages observations and run the rainfall analysis to see how well the magnitude and timing fit at the withheld rain gage locations. In almost all cases, the analyzed rainfall has been within five percent of the rain gage observations and usually within two percent.

In data sparse regions where there are a limited number of rain gages, there can be increased uncertainty in traditional rainfall analyses, especially in topographically significant regions. For the September 2013 storm, this was not the case. There was excellent weather radar coverage along with many rainfall observations with excellent overall spatial distributions at both low and high elevation locations. Another important point to note is that although convective rainfall estimated from

NEXRAD can be questionable in the Colorado Front Range foothills, there are many papers in the literature on the good to excellent reliability of NEXRAD for frontal/upslope storms such as the September 2013 storm. Further information on SPAS can be found at the Applied Weather Associates website: http://www.appliedweatherassociates.com/spas-storm-analyses.html.

Basin shape files were provided to AWA to overlay on top of the gridded data. NEXRAD radar imagery utilized a best fit curve to break down the hourly storm increments into five minute increments at a grid spacing of one kilometer. The gridded rainfall information was then converted to an average rainfall hyetograph for each basin and imported into HEC-HMS as time series precipitation gage data. The hyetographs include 10 days of 5-minute incremental rainfall depths at the centroid of each basin.

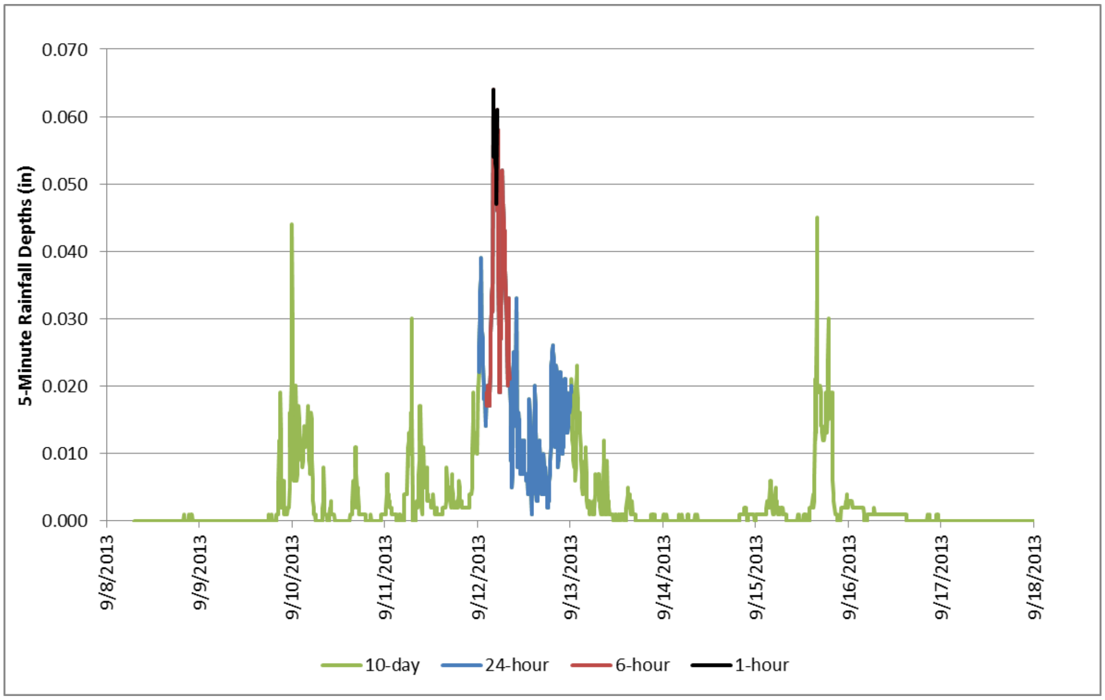
The average 10-day cumulative rainfall depth for all of the basins in Phase 2 was 7.59 inches, ranging from as low as 5.57 inches up to 10.37 inches for the individual basins. However, almost half of this rainfall fell within a 24-hour period starting around 1 P.M. (MST) on Wednesday, September 11, 2013. The average maximum 24-hour rainfall depth for all of the basins was 3.38 inches, ranging from 1.96 inches up to 5.51 inches for the individual basins. The average maximum 24-hour rainfall depth of 3.38 inches roughly corresponds to between a NOAA 10-year and 25-year rainfall depth. Table 3 shows the September 2013 rainfall depths for various durations in five representative basins from the study area. It also shows the associated NOAA Atlas 14 recurrence interval for each depth-duration pair.

Figure 2 shows a hyetograph for a basin in the headwaters area of Dry Creek No. 2. The incremental depths are based on a 5-minute time step. As shown in Table 3, Dry Creek No. 2 experienced some of the highest rainfall totals and intensities in the Phase 2 study area. The time of occurrence for maximum rainfall depth for various durations is shown on Figure 2 in different colors. It should be noted that the 10-day rainfall total is approximately a 1000-year event, the maximum 24-hour rainfall total is between a 100-year and 200-year event, the maximum 6-hour rainfall total is between a 10-year and 25-year event, and the maximum 1-hour rainfall total is less than a 2-year event. This is a good indicator that although the total rainfall depth is an extremely rare event, the rainfall intensities for shorter durations were not that extreme.

Table 3. Representative Rainfall Depths from September 2013 Flood and Associated NOAA Atlas 14 Recurrence Intervals

Location	Dry Creek No. 2 Headwaters (SV97C)		St. Vrain Creek below Lyons (SV77A)		St. Vrain Creek below Longmont (SV95)		Godding Hollow Headwaters (SV81B)		St. Vrain Creek at Mouth (SV 68)	
Duration	Rainfall (in)	NOAA RI (yr)	Rainfall (in)	NOAA RI (yr)	Rainfall (in)	NOAA RI (yr)	Rainfall (in)	NOAA RI (yr)	Rainfall (in)	NOAA RI (yr)
10-day	10.37	1000	8.34	200 to 500	5.60	25 to 50	9.03	> 1000	7.25	200
24-hour	5.44	100 to 200	4.12	25 to 50	2.26	2 to 5	4.91	100 to 200	2.91	10 to 25
6-hour	2.53	10 to 25	1.90	5 to 10	1.20	2	2.68	25	2.49	10 to 25
1-hour	0.66	1 to 2	0.53	< 1	0.46	< 1	0.97	2 to 5	1.42	10 to 25

Figure 2. September 2013 Rainfall Hyetograph for Dry Creek No. 2



The HEC-HMS model Control Specifications were set to coincide with the rainfall period start and end times. The background map for the model used the GIS basin delineations shapefile to provide spatial reference for the model components.

2.4.6 Model Calibration and Validation

The first step in the model calibration process was calibrating the rainfall data from the 2013 storm to ground measurements, as discussed in the previous section. Once all required model input parameters were obtained and the rainfall data from the 2013 flood were incorporated, initial runs of the model were made to identify any potential errors in the setup. After the base model was up and running correctly with the default input parameters, the next step was to incorporate inflow hydrographs from the Phase 1 Upper St. Vrain Creek model, Phase 1 Upper Lefthand Creek model and the Phase 2 Boulder Creek model. These inflow hydrographs are provided in Appendix D.3. Once all of the required inputs were added, the model was calibrated to match the estimated peak discharges and available gage data for the 2013 flood event.

Many of the model input parameters are physically based such as lengths and slopes of basins and channels. However, there are several input parameters that are empirical and can be used as calibration parameters. Five calibration parameters were evaluated to try and match the estimated peak discharge points from the 2013 flood event including: Curve Number (CN), Peaking Coefficient (Cp), Basin Roughness (Kn), Channel Roughness (Manning's n), and Channel Loss (Loss). Some parameters had more pronounced effects on the model results than others as described below.

Changing the CN value impacts the initial abstraction and the decaying infiltration rate which has the combined effect of reducing the total runoff volume over the 10-day period. More specifically, changing the CN value has noticeable effects on runoff volume during the first few days of the storm when the initial abstraction is being utilized, but then high peak discharges are still observed when the most intense part of the hyetograph occurs later.

Changing Cp and the Kn value in the lag time equation had some effect on localized basin peak discharges, but these effects did not translate downstream very far in the routing network. Changing the steepness of the hydrograph or the timing of the peak had little influence downstream because of the nature of this long duration storm event with recurring periods of high rainfall. The individual basin runoff hydrographs typically had at least two peaks close together which regardless of small shifts in timing would still overlap with the peaks from adjacent basins as they are routed downstream.

Attempts to calibrate the model using the channel roughness alone did not produce noticeable impacts. Dramatic adjustments to the Manning's n value up or down had some minor effect on the timing of peaks but had no effect on the magnitude of the peak. The 8-point cross-sections in several reaches of St. Vrain Creek were over a mile wide and considerable floodplain attenuation was expected, yet adjustment of the overbank Manning's n values were unable to produce any noticeable reduction in the peak. After some additional research, it was concluded that the Muskingum-Cunge method, as well as several of the other HEC-HMS routing options, do not provide peak flow reduction through attenuation in the overbanks, but instead emphasize the timing of the hydrograph translation. Therefore, this factor limited the effect of the roughness coefficient as a calibration parameter for reducing peaks. However, the parameter was effective in adjusting travel times to avoid coincidental peaks. Further review of literature, specifically reports by Jarrett (1985) and Barnes (1967) regarding the appropriate Manning's n values for Colorado streams was conducted and it was determined that values ranging from 0.05 to 0.15 were appropriate for the channels in the Phase 2 study area. The final calibrated Manning's n values are provided in Appendix D.1

The actual flood attenuation during the 2013 Flood was caused by a number of real world factors including:

- Irrigation head gates that diverted water from St. Vrain Creek, possibly to storage reservoirs or adjacent watersheds. Even if this water eventually returned to St. Vrain Creek it was most likely delayed relative to the peak observed in St. Vrain Creek.

- Bridge crossings that acted as constrictions limiting the peak discharge downstream by backing up water into the floodplains. This impounded water either spills downstream along an overland flow path or is stored until the peak flow starts to recede and the water can pass through the constriction. Regardless, this constriction results in a shaving off of the hydrograph peak. This type of impact is evident in FIS profiles which show backwater conditions upstream of bridge crossings. This type of impact is evident in FIS profiles which show backwater conditions upstream of bridge crossings.

- Gravel pits located within the floodplain that have available storage capacity and potentially result in split flows by diverting water along historic channel alignments. One example of this form of attenuation is the split flow that occurred near Hygiene and diverted flow to the north of the St. Vrain Creek channel where it then flowed east following the railroad tracks toward Longmont. This split flow is evident in the observed flood extent mapping produced for the City of Longmont. This diverted flow had a much lower velocity following the higher ground to the north of the channel than the peak moving down the main channel.

Although the actual flood attenuation of any single one of these factors at a single location can be considered negligible, they did have a combined effect that was clearly observed during the 2013 flood. Several alternative options were evaluated for modeling the peak discharge attenuation that occurred. These options included using the Modified Puls Routing method, adding generic storage nodes to represent floodplain storage at bridges or head gate diversions, or assuming minor channel losses to represent flood attenuation. The first two options required developing storage-discharge functions for the floodplain which would require a significant effort in data collection and would be difficult to calibrate. The final option was used because of its simplicity; a loss percentage was entered for a routing reach which reduced the peak discharge and acted as a surrogate for floodplain attenuation. Although the explicit causes of attenuation were not modeled directly, the combined effects on the peak discharge downstream were accounted for. The calibrated channel loss percentage was roughly based on channel length, floodplain width, and number of irrigation diversion/bridges within the channel reach.

It should be noted that this floodplain attenuation method was only used in calibration of the 2013 flood model. The types of floodplain attenuation discussed above are not accounted for in the predictive storm models since there is no guarantee that irrigation systems, bridge constrictions, or gravel pits in the overbank areas will remain in the same condition for perpetuity.

Calibration of the Phase 2 model along St. Vrain Creek was dependent on two primary factors. The first was the inflow hydrographs from Upper St. Vrain Creek (218 square miles), Upper Lefthand Creek (58 square miles) and Boulder Creek (447 square miles) provided in Appendix D.3. The second factor was the major tributaries and local drainage basins within the Phase 2 study area. The upper watersheds represented by inflow hydrographs experienced heavier rainfall during the 2013 flood and the discharges to the Phase 2 model tended to dominate the peak discharges downstream in St. Vrain Creek. Peak discharges from the Phase 2 local tributaries were generally smaller and peaked earlier than the discharge from the upper watersheds limiting the overlap. There was overlap though between St. Vrain Creek, Lefthand Creek and Boulder Creek since all three watersheds extend to the Continental Divide and travel times are fairly similar.

Based on this knowledge, the smaller Phase 2 tributaries were calibrated first based on available peak discharge estimates, gage records, and comparison of unit discharges with respect to rainfall depths/intensites. Once the tributary drainage basins in Phase 2 were initially calibrated, attempts were made to calibrate the combined flows in St. Vrain Creek. In most locations, the calibration was relatively straightforward and results were close to the observed peak discharges. However, at a few locations, the peak discharge estimates were difficult to attain even when pushing the calibration parameters well beyond acceptable limits. In some cases the peak discharge estimates fluctuated up and down without any obvious inflows or floodplain obstructions between the two locations. In these locations, the comments provided by URS for each peak discharge estimate (Appendix A) were closely evaluated to determine which estimate to weight more heavily. After several iterations of calibrating the model, a relatively close fit to the estimated peak discharges was obtained. Table 4 provides a comparison of peak discharges from the 10-day storm model to peak discharges observed during the September 2013 Flood in the St. Vrain Phase 2 study area. Calibration results for the 10-day 2013 flood event are discussed in more detail in Section 3.0 of this report.

Table 4. Comparison of Modeled Discharges to Observed Discharges

Location	Observed 2013 Discharge (cfs)	Modeled 2013 Discharge (cfs)	Percent Difference
St. Vrain Creek above Airport Road	14,000	14,100	1%
St. Vrain Creek below Highway 287	14,500	15,000	3%
St. Vrain Creek below Lefthand Creek	18,500	18,700	1%
St. Vrain Creek at Interstate 25	23,500	23,300	- 1%
St. Vrain Creek at State Highway 66	23,000	23,900	4%
St. Vrain Creek at County Road 34	27,000	23,900	- 11%
Lefthand Creek at 63rd Street	7,000	7,210	3%
Lefthand Creek at Diagonal Highway	8,700	5,930	- 32%
Lefthand Creek at Hwy 287	5,000	5,090	2%
Lefthand Creek at St. Vrain Creek	4,800	4,800	0%

2.5 Rainfall / Runoff Model for Predictive Peak Discharges

2.5.1 Overall Modeling Approach

Once the rainfall-runoff model was calibrated to represent the September 2013 rainfall and peak runoff, the model was used to predict peak discharges based on NOAA rainfall for a number of return periods to help guide the design of permanent roadway improvements in the study watersheds. This analysis of NOAA rainfall data is referred to herein as the predictive model. Several additional calibration steps were involved in this process as described below.

2.5.2 Design Rainfall

The NOAA Atlas 14, Volume 8 was used to determine point precipitation frequency estimates for each basin. Latitude and Longitude values were determined for the centroid of each basin in order to obtain point precipitation frequency estimates specific to each basin. Table 5 below and Appendix D.2 show the point precipitation values for the different basins. Table 5 also shows the 90 percent confidence intervals on the 24-hr rainfall depths which expresses some of the uncertainty. Figure D.1 in Appendix D shows the basin delineations for reference. The rainfall depths were applied to the standard 24-hour SCS Type II rainfall distribution. The 24-hour distributions were then incorporated into the HEC-HMS model to evaluate peak discharges for the predictive storms.

Table 5. Lower St. Vrain Precipitation Depths

Point Precipitation Frequency Estimates with 90% Confidence Intervals (inches)					
Model Basin	**10-yr, 24-hr**	**25-yr, 24-hr**	**50-yr, 24-hr**	**100-yr, 24-hr**	**500-yr, 24-hr**
SV77A	2.89 (2.36-3.55)	3.71 (2.97-4.84)	4.41 (3.43-5.82)	5.19 (3.89-7.03)	7.30 (5.02-10.5)
SV76	2.89 (2.36-3.56)	3.71 (2.97-4.86)	4.42 (3.43-5.84)	5.20 (3.89-7.05)	7.30 (5.02-10.5)
SV77B	2.91 (2.39-3.56)	3.73 (2.99-4.83)	4.43 (3.45-5.80)	5.20 (3.90-6.98)	7.26 (5.00-10.3)
SV84	2.87 (2.34-3.53)	3.67 (2.94-4.81)	4.37 (3.40-5.78)	5.14 (3.85-6.97)	7.21 (4.95-10.3)
SV86B	2.97 (2.44-3.60)	3.78 (3.05-4.86)	4.49 (3.50-5.82)	5.25 (3.95-6.99)	7.29 (5.02-10.3)
SV86A	2.89 (2.37-3.53)	3.70 (2.97-4.78)	4.39 (3.42-5.73)	5.14 (3.85-6.89)	7.14 (4.91-10.1)
SV87	2.90 (2.37-3.55)	3.70 (2.96-4.80)	4.39 (3.40-5.74)	5.12 (3.83-6.88)	7.08 (4.86-10.1)
SV94B	2.95 (2.44-3.58)	3.77 (3.04-4.83)	4.47 (3.49-5.78)	5.22 (3.93-6.93)	7.23 (4.98-10.1)
SV94A	2.92 (2.39-3.55)	3.72 (2.98-4.78)	4.40 (3.42-5.72)	5.13 (3.84-6.84)	7.06 (4.85-9.96)
SV89	2.90 (2.37-3.55)	3.70 (2.96-4.80)	4.39 (3.40-5.74)	5.12 (3.83-6.88)	7.08 (4.86-10.1)
SV75	2.84 (2.31-3.52)	3.64 (2.91-4.82)	4.35 (3.37-5.81)	5.13 (3.83-7.03)	7.25 (4.98-10.5)
SV93	2.88 (2.33-3.55)	3.68 (2.92-4.82)	4.37 (3.37-5.77)	5.11 (3.81-6.95)	7.11 (4.88-10.3)
SV92D	3.02 (2.50-3.64)	3.85 (3.11-4.90)	4.55 (3.57-5.85)	5.31 (4.00-6.99)	7.31 (5.04-10.2)
SV92C	2.96 (2.45-3.56)	3.77 (3.04-4.79)	4.45 (3.48-5.71)	5.19 (3.90-6.82)	7.11 (4.89-9.87)
SV92B	2.95 (2.42-3.58)	3.75 (3.00-4.81)	4.43 (3.44-5.74)	5.15 (3.85-6.85)	7.04 (4.84-9.91)
SV92A	2.92 (2.37-3.58)	3.72 (2.95-4.83)	4.40 (3.39-5.77)	5.13 (3.81-6.91)	7.04 (4.82-10.1)
SV95	2.91 (2.35-3.56)	3.70 (2.92-4.79)	4.36 (3.35-5.72)	5.07 (3.76-6.85)	6.94 (4.75-9.96)
SV97C	3.07 (2.56-3.67)	3.90 (3.16-4.91)	4.60 (3.61-5.84)	5.35 (4.04-6.96)	7.31 (5.04-10.0)
SV97B	2.98 (2.45-3.61)	3.79 (3.04-4.83)	4.47 (3.48-5.75)	5.19 (3.89-6.85)	7.08 (4.86-9.87)
SV97A	2.94 (2.39-3.59)	3.74 (2.96-4.82)	4.40 (3.39-5.75)	5.12 (3.80-6.86)	6.97 (4.77-9.93)
SV88	2.87 (2.31-3.54)	3.65 (2.89-4.77)	4.32 (3.32-5.71)	5.04 (3.74-6.84)	6.97 (4.77-10.0)
SV96	2.88 (2.32-3.54)	3.65 (2.88-4.74)	4.30 (3.30-5.64)	4.99 (3.70-6.73)	6.81 (4.66-9.76)
SV90	2.88 (2.33-3.54)	3.65 (2.88-4.74)	4.30 (3.30-5.64)	4.99 (3.70-6.73)	6.81 (4.65-9.74)
SV85	2.86 (2.30-3.52)	3.62 (2.85-4.71)	4.26 (3.27-5.61)	4.96 (3.68-6.70)	6.79 (4.65-9.73)
SV83	2.85 (2.29-3.52)	3.62 (2.86-4.73)	4.27 (3.28-5.64)	4.98 (3.70-6.75)	6.88 (4.71-9.86)
SV80C	2.85 (2.29-3.51)	3.60 (2.83-4.68)	4.23 (3.24-5.56)	4.91 (3.64-6.62)	6.68 (4.57-9.55)
SV80B	2.84 (2.28-3.49)	3.59 (2.83-4.67)	4.23 (3.24-5.56)	4.92 (3.65-6.64)	6.75 (4.62-9.65)
SV81B	2.84 (2.28-3.50)	3.57 (2.81-4.63)	4.18 (3.20-5.49)	4.83 (3.58-6.51)	6.54 (4.47-9.32)
SV81A	2.81 (2.26-3.47)	3.55 (2.79-4.61)	4.16 (3.19-5.47)	4.82 (3.57-6.50)	6.56 (4.49-9.34)
SV80A	2.83 (2.27-3.48)	3.58 (2.82-4.65)	4.21 (3.23-5.54)	4.90 (3.64-6.61)	6.73 (4.61-9.60)
SV73	2.80 (2.26-3.46)	3.54 (2.80-4.61)	4.17 (3.20-5.48)	4.85 (3.60-6.53)	6.65 (4.56-9.46)
SV74	2.78 (2.24-3.43)	3.50 (2.76-4.54)	4.11 (3.16-5.39)	4.78 (3.54-6.41)	6.52 (4.47-9.23)
SV72	2.78 (2.24-3.42)	3.51 (2.78-4.55)	4.13 (3.18-5.41)	4.81 (3.57-6.45)	6.60 (4.52-9.35)
SV71B	2.84 (2.29-3.51)	3.61 (2.85-4.72)	4.27 (3.28-5.64)	4.99 (3.70-6.76)	6.90 (4.73-9.90)
SV71A	2.82 (2.27-3.48)	3.57 (2.82-4.66)	4.22 (3.24-5.56)	4.92 (3.65-6.65)	6.79 (4.65-9.69)
SV70	2.78 (2.25-3.42)	3.52 (2.78-4.57)	4.14 (3.19-5.43)	4.82 (3.59-6.48)	6.64 (4.55-9.42)
SV69	2.81 (2.26-3.46)	3.56 (2.81-4.64)	4.20 (3.23-5.53)	4.90 (3.64-6.62)	6.77 (4.64-9.66)
SV68	2.79 (2.25-3.43)	3.53 (2.79-4.58)	4.16 (3.20-5.46)	4.84 (3.60-6.52)	6.68 (4.58-9.49)

Due to the size of the St. Vrain watershed (approximately 978 square miles) it was necessary to consider area correction of the rainfall depths prior to generating runoff hydrographs. Therefore, depth-area reduction factors (DARF) were applied to NOAA point precipitation estimates. The depth-area reduction factor accounts for the gradual decrease in precipitation depth with increasing distance from the storm centroid and corrects the NOAA point precipitation estimate to the average rainfall that would occur over the spatial extent of the storm. While DARF curves provided in NOAA Atlas 2 were used in the Phase 1 hydrologic analysis, the NOAA Atlas 2

DARF curves only cover drainage areas up to 400 square miles. As total drainage areas of the Big Thompson River, Boulder Creek, and St. Vrain Creek each exceeded 400 square miles, CDOT and CWCB contracted AWA to derive a site-specific 24-hour DARF curve for use in the hydrologic analysis of these large watersheds. A memo documenting AWA's work is provided in Appendix C.

AWA analyzed nine storm events along the Front Range of the Rocky Mountains extending from northern New Mexico through southern Canada, including the September 2013 event. Each storm event utilized in this analysis represented meteorological and topographical characteristics that were similar to each other and to the September 2013 event. These storms were selected to derive storm specific DARFs. The individual storm DARFs were then utilized to derive a site-specific set of 24-hour DARF values to be used in the Phase 2 hydrologic analysis along the northern Front Range of Colorado (Big Thompson River, Boulder Creek, and St Vrain Creek). These site-specific storm based 24-hour DARF values were used to extend those provided in NOAA Atlas 2 for area sizes greater than 400 square miles. This analysis resulted in 24-hour DARF values that varied significantly from NOAA Atlas 2 values, demonstrating the need for the updated analysis to capture the unique storm characteristics along the Front Range and to more accurately capture the DARFs for larger basins in the region.

To avoid significant reductions in the predicted 10, 4, 2, 1, and 0.2 percent annual chance peak discharges at the interfaces between the Phase 1 and Phase 2 study areas that would occur if the site-specific AWA DARF curve was strictly adopted for the Phase 2 hydrology analysis, a transition curve between the higher NOAA Atlas 2 DARF curve and the AWA site-specific DARF curve was developed. The transition curve started at 315 square miles which allowed for a consistent approach to be used between the two study phases and all the watersheds, as a DARF of 0.92 was utilized to estimate predictive hydrology for a drainage area of 315 square miles in the Phase 1 hydrologic analysis of the Big Thompson River. The transition curve then dropped down and tied into the AWA curve at 500 square miles providing a smooth transition between the two curves. This transition curve was tested at several design points with areas between 315 and 500 square miles and it produced reasonable results when compared against current regulatory values and expected unit discharges. For modeling purposes, a step function was developed to break the combined DARF curve into about a dozen area increments. The stepped area increments reasonably represent the actual DARF value for all of the modeled nodes (within 1%) and significantly reduces the number of model runs necessary to produce results at each node. Figure 3 below shows the various DARF curves, the model nodes for each watershed, and the stepped area increments used to represent each model node. As evident in Figure 3, the transition curve is conservative with respect to the AWA curve. Table 6 provides the area increments and resulting DARF values used in the Phase 2 hydrologic analysis. These new DARF values were developed specifically for this study and are not recommended in other locations without further evaluation.

For the 24-hr storm duration, rainfall depths are reduced by as much as 34% depending on the drainage area. For tributary areas less than 10 square miles, no area correction was applied. Between 10 and 30 square miles, a 2% reduction was applied to all upstream basins. Between 30 and 50 square miles, a 4% reduction was applied to all upstream basins. This process continues as shown in Table 6 for all nodes in the model to determine the appropriate peak discharge. Downstream of the confluence with Boulder Creek, a 34% reduction in rainfall is applied to all basins in the model to determine the effective peak discharges from I-25 downstream to the confluence with the South Platte River. This results in unadjusted rainfall depths being used to generate peak discharges in the headwater areas, while the area corrected rainfall depths are used as the design points move progressively downstream along St. Vrain Creek. This process is described in more detail in Appendix D.2. Appendix D.4 shows the appropriate DARF value for all model nodes.

Figure 3. Depth-Area Reduction Factor Curves

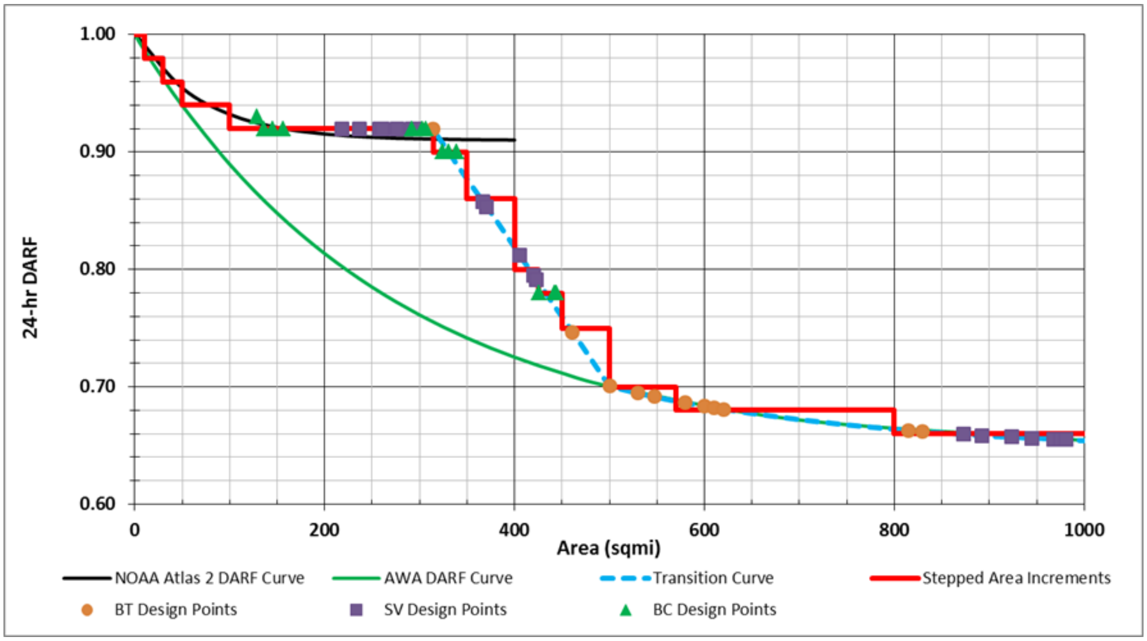


Table 6. Stepped Area Increments for DARF Application

Area Range (square miles)		24-hour DARF
Low	High	
0	10	1.00
10	30	0.98
30	50	0.96
50	100	0.94
100	315	0.92
315	350	0.90
350	400	0.86
400	425	0.80
425	450	0.78
450	500	0.75
500	570	0.70
570	800	0.68
800	1000	0.66

Depending on watershed characteristics and influences such as large flood control reservoirs, the peak discharge at a study location may not be the result of a general storm distributed over the entire watershed area, but rather a more intense storm concentrated over a smaller portion of the watershed. Therefore, as part of this study, spatially concentrated storms were evaluated using the predictive model to determine if a more intense rainfall over a smaller portion of the watershed would produce higher peak discharges. This critical storm analysis was done by delineating several potentially critical storm areas across the St. Vrain Creek watershed. These critical areas included:

- Only the Phase 2 Study Area without inflows from Phase 1 (Upper St. Vrain Creek or Upper Lefthand Creek) or Boulder Creek.
- Phase 2 Study Area with inflows from Upper St. Vrain Creek, but no inflows from Upper Lefthand Creek or Boulder Creek.
- Phase 2 Study Area with inflows from Upper St. Vrain Creek and Upper Lefthand Creek but no inflows from Boulder Creek.
- Phase 2 Study Area with inflows from part of Upper St. Vrain Creek and part of Upper Lefthand Creek but no inflows from Boulder Creek. A north and south line was drawn at approximately the location of the Button Rock Dam. All of the basins to the east of the line were assumed to be covered by the storm (approximately 49 square miles from North and South St. Vrain Creeks and approximately 31 square miles from Lefthand Creek).

After delineating the potentially critical storm areas, rainfall depths for the individual basins were adjusted using the DARF for the tributary area covered by the storm, rather than the total tributary drainage area upstream of that point; thus the peak intensity of rainfall increased in basins as a result of the increased DARF. The model results from the spatially concentrated storms yielded peak discharges along St. Vrain Creek that were equal to or smaller than the general storm spread over the entire watershed.

In addition to the 24-hour critical storm analysis, a 6-hour storm duration was also evaluated using the spatially concentrated storm areas described above. This was checked because in smaller basins the shorter, more intense design storms often produce larger peak discharges. St. Vrain Creek peak discharge results from the 6-hour critical storm analysis were significantly less than those from the 24-hour storms since the DARF adjustments are larger for shorter duration storms and the tributary hydrographs are less likely to overlap and combine as they move downstream in the watershed. Even for the smaller tributary basins, the 6-hour storms did not produce higher discharges in the Phase 2 Study Area.

2.5.3 Model Calibration

In order to calibrate the predictive model, it was necessary to adjust the 10-day calibrated CN values to account for the difference in initial abstractions between the 10-day storm and a 24-hour storm. As discussed in the Phase 1 Hydrologic Evaluation Reports, the calibrated CN values for the 10-day storm are highly dependent on the rainfall early in the storm that saturates the soil prior to the peak rainfall occurring. This initially raised some concerns about the applicability of the CN infiltration method. Known weaknesses of the CN infiltration method are that rainfall intensity is not considered and the default initial abstraction does not depend upon storm characteristics or timing. Therefore, three other infiltration options in HEC-HMS (constant loss, exponential loss, and Green-Ampt) were evaluated in the Phase 1 study to determine if they responded differently to the 10-day vs. 24-hr rainfall storms. Optimization routines in HEC-HMS were utilized to compare the different infiltration methods to determine which best matched observed runoff in the 2013 Flood. Based on the optimization results it was determined that the CN Method was actually able to produce the best fit to the observed data. Additional detail regarding comparisons of the different infiltration methods can be found in the Phase 1 Hydrologic Evaluation Reports. Although the CN method has its weaknesses, it is suitable for large return period storm events. Additionally, since it is being used as a calibration parameter, the actual selection of default values is not critical.

In order to address the 10-day storm vs. NOAA 24-hour rainfall duration, the maximum 24-hour period of rainfall was extracted from the 10-day period of data and used to re-calibrate the model. The maximum 24-hour period of rainfall was determined by finding the maximum 24-hour rainfall depth and start time for each of the individual basins in the watershed (performed using a VBA macro developed in Excel). Based on the maximum 24-hour time periods for each of the individual basins, a common 24-hour period that best represented the entire watershed was selected. This 24-hour period, which started at 1 P.M. (MST) on Wednesday, September 11, 2013, was then extracted from the 10-day rainfall record for each basin. The next step was to determine what adjustment in CN values was necessary to match the estimated 2013 Flood peak discharges using only the maximum 24-hour period of rainfall. This served to create an upper bound on the Max24hr CN calibration since the difference between the average 10-day rainfall (7.59 inches) and the average 24-hour maximum rainfall (3.38 inches) for the St. Vrain Watershed was 4.21 inches. Therefore, it should be expected that high Max24hr CN values would be necessary to produce the same peak discharges when using less than half of the rainfall total.

The next step was to consider the percentage of rainfall that becomes runoff during the peak of the storm for both the 10-day model and the Max24hr model. Therefore, a ratio of total runoff (inches) divided by total rainfall (inches) was determined for each individual basin in the 10-day model. These ratios were then multiplied by the maximum 24-hour rainfall depths for each basin to determine the corresponding runoff depth expected for each basin during the 24-hour period of maximum rainfall. The goal was to maintain consistency between the amount of rainfall that infiltrated and the amount that became runoff during the most intense period of the 2013 Flood event. The final step was to iteratively determine the Max24hr CN values necessary to produce the expected runoff depths for each individual basin. Appendices D.4 through D.8 include the model results for the Max24hr rainfall period utilizing the CN values required to match the 2013 Flood as well as the runoff/rainfall ratio determined CN values.

Using the calibrated Max24hr runoff/rainfall ratio model, the NOAA 24-hour rainfall depths and SCS Type II storm distributions were applied for each of the return periods. As a reasonableness check, the predictive model results were compared to expected unit discharges and the updated flood frequency analyses. These checks served to further validate that the CN values from the calibrated Max24hr rainfall model were better able to reflect the difference between the rainfall distributions from the 2013 Flood and the SCS 24-hr storm distributions. Results from the predictive models are discussed in more detail in Section 3.0 of this report.

3.0 HYDROLOGIC MODEL RESULTS

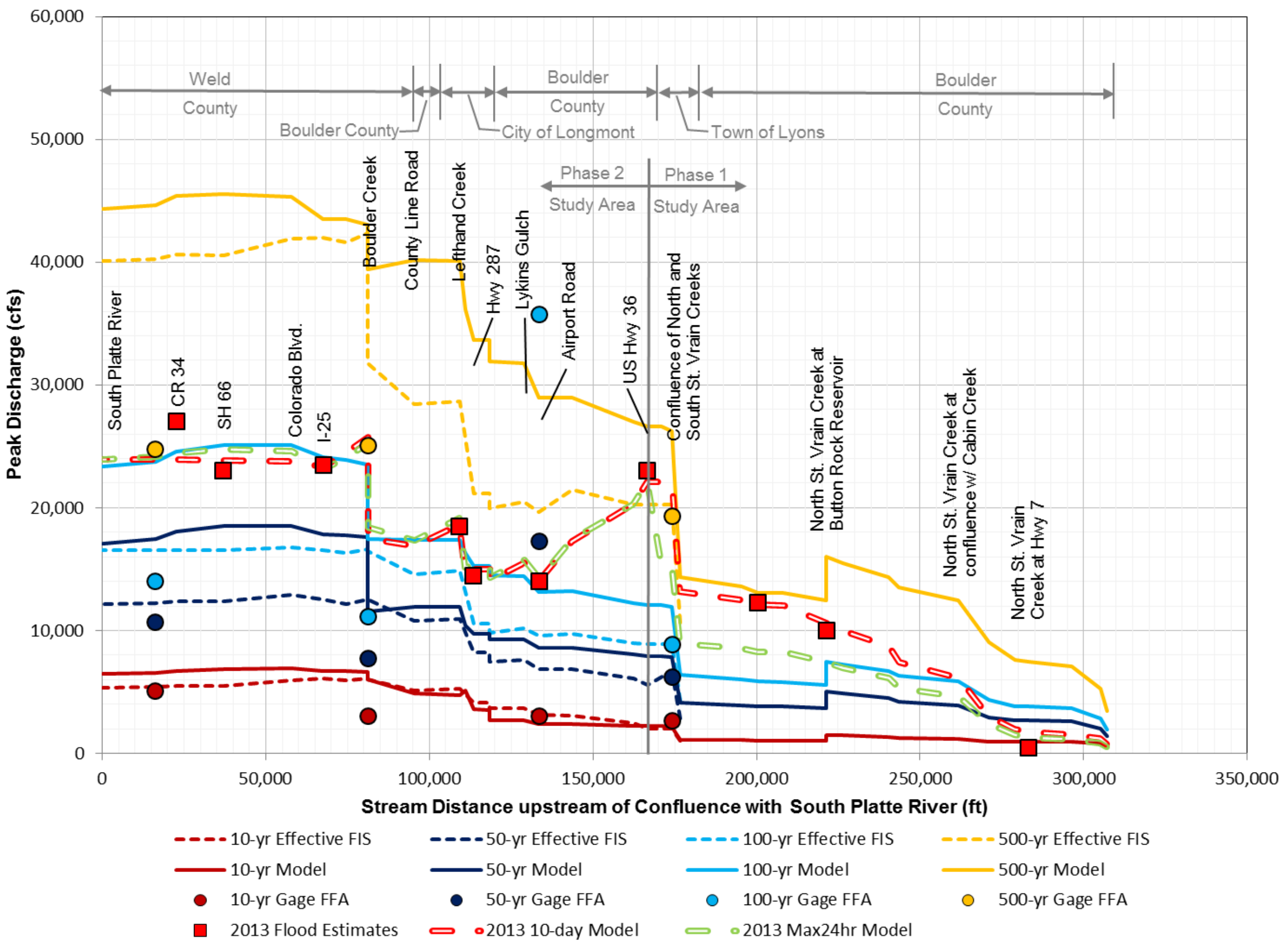
Table 7 below and the expanded table in Appendix D.5 show results at selected locations along the main stem of the St. Vrain Creek (from headwaters on North St. Vrain Creek to the South Platte River), Lefthand Creek (from headwaters to St. Vrain Creek), and Dry Creek No. 2.

Table 7. Hydrologic Model Peak Discharge Results

Description	Area (sq. mi.)	Estimated Peak (cfs)	2013 Flood Calibrated Model 10-day (cfs)	2013 Flood Max 24hr Period CN Calibrated (cfs)	NOAA Design Storms (CN Calib & DARF) 10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)
North SVC headwaters	13		773	511	560	970	1,390	1,910	3,480
North SVC at confluence with Cony Creek	21		1,178	838	770	1,370	2,010	2,810	5,300
North SVC at Copeland Falls	29		1,551	1,152	950	1,750	2,590	3,660	7,060
North SVC at Hwy 7 (Peak Estimation Point #63 by Bob Jarrett)	**33**	**450**	**1,713**	**1,279**	**960**	**1,780**	**2,670**	**3,800**	**7,420**
North SVC at confluence with Horse Creek below Hwy 7	37		1,972	1,481	920	1,740	2,660	3,820	7,640
North SVC at confluence with Rock Creek	53		3,061	2,418	950	1,870	2,940	4,330	9,020
North SVC at confluence with Cabin Creek	76		6,162	4,649	1,200	2,450	3,920	5,840	12,500
North SVC at Coulson Gulch confluence	83		7,441	5,335	1,280	2,630	4,220	6,300	13,500
North SVC at confluence with Dry SVC	91		8,706	6,139	1,350	2,790	4,490	6,700	14,400
North SVC inflow to Button Rock Reservoir	98		10,007	6,997	1,460	3,010	4,850	7,250	15,500
North SVC inflow to Button Rock Reservoir	**101**	**10,000**	**10,591**	**7,389**	**1,500**	**3,110**	**5,000**	**7,470**	**16,000**
Discharge from Button Rock Reservoir	**101**		**10,591**	**7,389**	**1,060**	**2,220**	**3,670**	**5,600**	**12,500**
North SVC below Longmont Reservoir at Hwy 36	111		12,023	8,240	1,050	2,290	3,790	5,820	13,000
North SVC at Peak Discharge Estimation Point #58 by Bob Jarrett	**112**	**12,300**	**12,094**	**8,270**	**1,060**	**2,300**	**3,800**	**5,840**	**13,100**
North SVC at Apple Valley Road	118		12,501	8,529	1,080	2,380	3,940	6,050	13,600
North SVC above confluence with South SVC	124		13,182	8,952	1,120	2,500	4,160	6,390	14,300
Confluence of North and South SVC	215		21,827	14,879	2,180	4,790	7,830	11,900	26,200
St Vrain Creek at confluence with Stone Canyon	218		22,102	15,016	2,200	4,860	7,940	12,100	26,600
Upper SVC Model Inflow Hydrographs	**218**	**23,000**	**22,127**	**22,127**	**2,200**	**4,860**	**7,950**	**12,100**	**26,600**
SVC below Union Road	222		20,271	20,320	2,210	4,910	8,070	12,300	27,000
SVC upstream of 75th Street	237		17,253	17,399	2,360	5,290	8,580	13,200	29,000
SVC above Golden Ponds (near Airport Rd)	**237**	**14,000**	**14,113**	**14,232**	**2,350**	**5,280**	**8,570**	**13,200**	**29,000**
SVC at Golden Ponds (Lykins Ditch Confluence)	259		15,531	15,743	2,690	5,760	9,270	14,400	31,700
SVC above Dry Creek #1 Confluence	262		14,122	14,323	2,700	5,750	9,290	14,500	31,900
SVC at BNSF Railroad	275		14,976	15,256	3,520	6,020	9,730	15,200	33,600
SVC below Hwy 287 (Main St.) at Dry Creek #1 Old Channel	**276**	**14,500**	**14,990**	**15,273**	**3,590**	**5,990**	**9,720**	**15,200**	**33,700**
SVC at Martin Street (The Slough)	296		14,771	15,128	5,150	6,720	10,400	16,300	36,200
SVC at Confluence with LHC	**368**	**18,500**	**18,651**	**19,172**	**4,740**	**7,370**	**11,900**	**17,400**	**40,100**
SVC upstream of Dry Creek #2 (County Line Rd.)	371		16,894	17,377	4,870	7,380	12,000	17,400	40,200
SV upstream of Boulder Creek	424		17,458	18,395	6,050	9,270	11,500	17,400	39,400
SVC at Confluence with Boulder Creek (Hwy 119)	870		25,757	25,222	6,650	11,600	17,600	23,500	43,000
SVC Upstream of I-25	881		24,534	24,187	6,730	11,800	17,800	23,900	43,500
SVC at I-25	**889**	**23,500**	**23,316**	**23,062**	**6,740**	**11,900**	**17,800**	**24,100**	**43,500**
SVC at Colorado Blvd.	921		23,750	24,586	6,900	12,400	18,500	25,100	45,400
SV at SH 66	**942**	**23,000**	**23,869**	**24,731**	**6,840**	**12,400**	**18,500**	**25,100**	**45,500**
SV at County Road 34	**965**	**27,000**	**23,893**	**24,425**	**6,710**	**12,100**	**18,100**	**24,600**	**45,400**
SV above UPRR Bridge	973		23,894	24,125	6,530	11,700	17,400	23,700	44,600
SVC at South Platte River	978		23,894	23,967	6,440	11,400	17,100	23,400	44,300
J49 (Confluence of LHC and Spring Gulch)	16		1,325	684	330	610	900	1,250	2,350
J49A (Lefthand Creek at Lickskillet Road)	**18**	**1,300**	**1,548**	**856**	**360**	**670**	**990**	**1,370**	**2,580**
Lefthand Creek above confluence w/ James Creek	23		2,626	1,619	500	920	1,360	1,890	3,540
J57 (Confluence of LHC and JC)	42		6,687	4,368	1,210	2,210	3,230	4,470	8,440
J54 (Lefthand Creek below Old Stage Road)	**47**	**3,520**	**7,734**	**5,148**	**1,260**	**2,340**	**3,440**	**4,800**	**9,220**
J67 (LHC confluence with Spruce Gulch)	51		8,461	5,539	1,340	2,490	3,680	5,150	9,980
J74 (Confluence of LHC and Geer Canyon)	58		9,394	6,332	1,460	2,740	4,080	5,770	11,400
Upper LHC Model Inflow Hydrographs	58		9,474	9,474	1,470	2,760	4,120	5,820	11,500
LHC at 63rd Street	**63**	**7,000**	**7,207**	**7,322**	**1,510**	**2,840**	**4,250**	**5,990**	**11,800**
LHC at Diagonal Highway (near Airport Rd)	**69**	**8,700**	**5,928**	**6,095**	**1,400**	**2,820**	**4,270**	**6,040**	**11,800**
LHC at Hwy 287 (Main St.)	**72**	**5,000**	**5,086**	**5,256**	**1,380**	**2,640**	**4,060**	**5,810**	**11,600**
LHC at SVC Confluence	**72**	**4,800**	**4,800**	**4,964**	**1,370**	**2,580**	**3,990**	**5,740**	**11,400**
Dry Creek #2 at Hwy 119	18		964	1,003	1,160	1,820	2,420	3,100	4,960
Dry Creek #2 at Hwy 287	30		1,893	2,105	1,840	2,700	3,600	4,620	7,440
Dry Creek No. 2 at SVC Confluence	35		2,268	2,560	2,010	2,890	3,780	4,920	8,110
Boulder Creek Model Inflow Hydrographs	447		13,890	13,890	4,790	9,000	13,700	18,500	37,300

Location descriptions and tributary drainage areas are provided for each location. Estimated peak discharge values from the 2013 flood are shown in the next column. The following two columns present the calibrated model results for the full 10-day rainfall period and the maximum 24-hour rainfall period, respectively. The last five columns present the NOAA 24-hour Type II distribution storms with area correction for the 10-, 25-, 50-, 100- and 500-year storms. The expanded table in Appendix D.5 also includes approximate river stationing, the corresponding model node for each location, the 2013 Effective FIS peak discharges, and the updated flood frequency analysis results at corresponding locations for the 10-, 50-, 100- and 500-year recurrence intervals. It should be noted that effective peak discharge locations were matched as close as possible to the model locations, but in some instances they may be a fair distance apart. Refer to Table 1 for the actual location descriptions and tributary drainage areas for the FIS peak discharges. Appendix D.4 includes the full set of model results for all nodes in the model. As shown in Table 7, the calibrated model matched the peak discharge estimates within 20 percent at all observed locations in the Phase 2 Study Area. The one exception is the peak discharge estimate at the Diagonal Highway on Lefthand Creek which did not account for the impacts of the backwater effects caused by the railroad bridge downstream of the highway. There are a few locations in the Upper St. Vrain Creek (Phase 1) and Upper Lefthand Creek (Phase 1) that exceeded 20 percent differences and descriptions of those differences are provided in the Phase 1 Hydrologic Evaluation Reports. Peak discharge profiles for St. Vrain Creek and Lefthand Creek are shown on Figures 4 and 5, respectively. The profile plots are also provided in Appendices D.6 and D.7 at a larger scale.

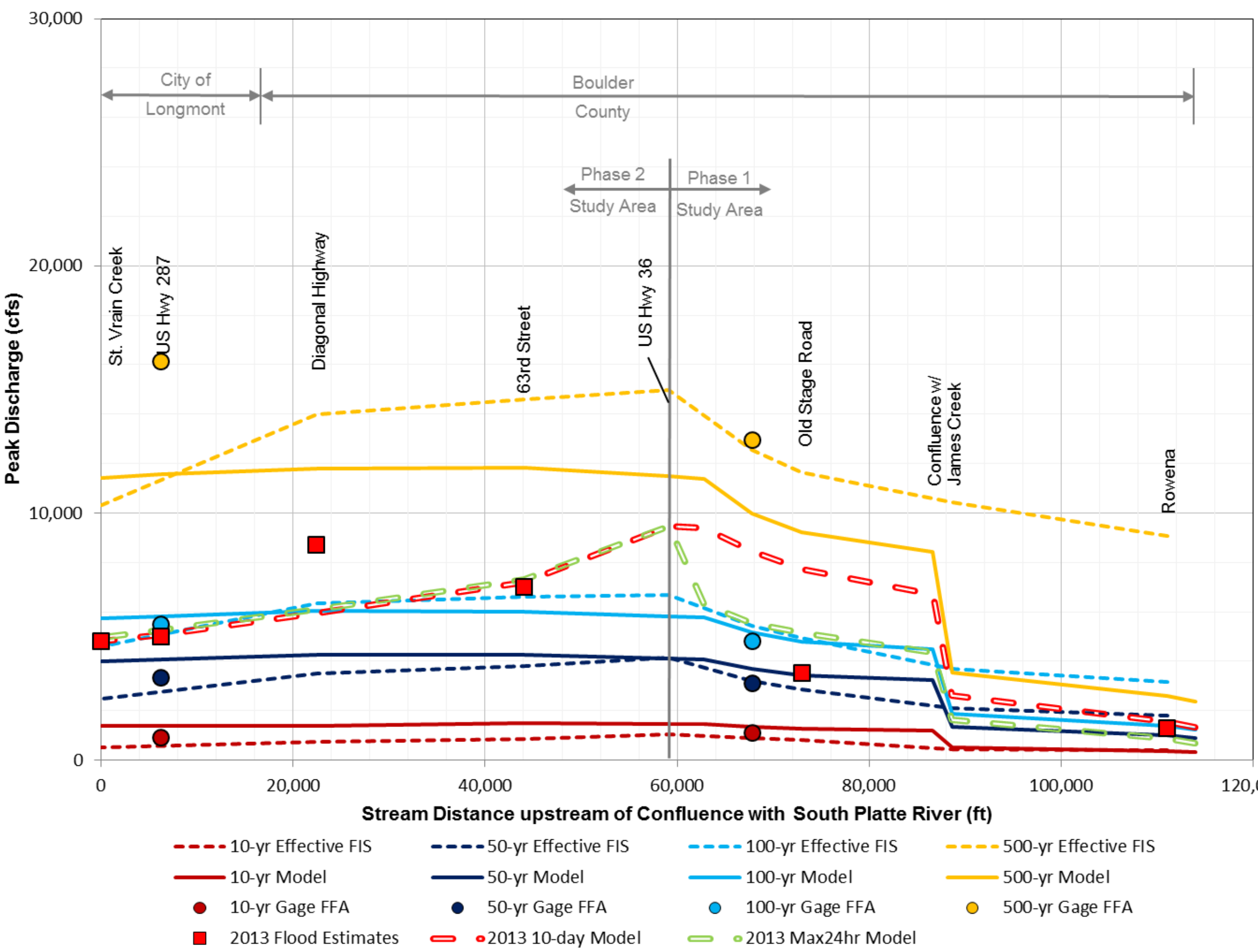
Figure 4. Peak Discharge Profiles for St. Vrain Creek and North St. Vrain Creek



The Effective FIS peak discharges are plotted as thin dashed lines. The corresponding predictive model results for the NOAA 24-hr Type II distribution storms are plotted as solid lines in the same color as the FIS discharges. The thick dashed red line is the calibrated 2013 Flood model using the full 10-day rainfall period and the thick dashed green line is the calibrated model for the maximum 24-hour rainfall period in the 2013 Flood. The estimated peak discharges and flood-frequency results are plotted as points on the profile plots.

As seen on Figure 4 for St. Vrain Creek, the calibrated 2013 flood model results for the 10-day rainfall period and the maximum 24-hour rainfall period are almost identical within the Phase 2 Study Area. It should be noted that the peak discharge in St. Vrain Creek during the 2013 Flood was driven primarily by the discharge from the upper St. Vrain Creek watershed above Lyons. Between Lyons and Longmont this peak discharge was attenuated significantly as a result of the broad undeveloped floodplain, several gravel pits, and a split flow that occurred near Hygiene and diverted flow to the north of the St. Vrain Creek channel where it then flowed east following the railroad tracks toward Longmont. As a result, the large peak discharge experience in Lyons was somewhat dissipated before reaching Longmont.

Figure 5. Peak Discharge Profiles for Lefthand Creek



There are several tributaries that join St. Vrain Creek between Lyons and Interstate 25 that had a noticeable impact on the peak discharges observed in the 2013 Flood. Lefthand Creek (72 square miles) and Boulder Creek (447 square miles) had the pronounced impact as shown in

Figure 4. Downstream of I-25 the peak discharge during the 2013 Flood remained relatively constant.

As seen in Figure 5 for Lefthand Creek, the calibrated 2013 flood model results for the 10-day rainfall period and the maximum 24-hour rainfall period are almost identical within the Phase 2 Study Area. The peak discharge during the 2013 Flood attenuated from the mouth of the canyon at Highway 36 to the confluence with St. Vrain Creek. Much of this attenuation is a direct result of the long, narrow shape of the basin in this reach and the lack of additional inflows to sustain the peak discharge. As discussed above, the only peak discharge estimate not matched within 20 percent by the model was at the Diagonal Highway where the hydraulic model used to develop the peak discharge estimate did not account for the backwater effects of the downstream railroad bridge.

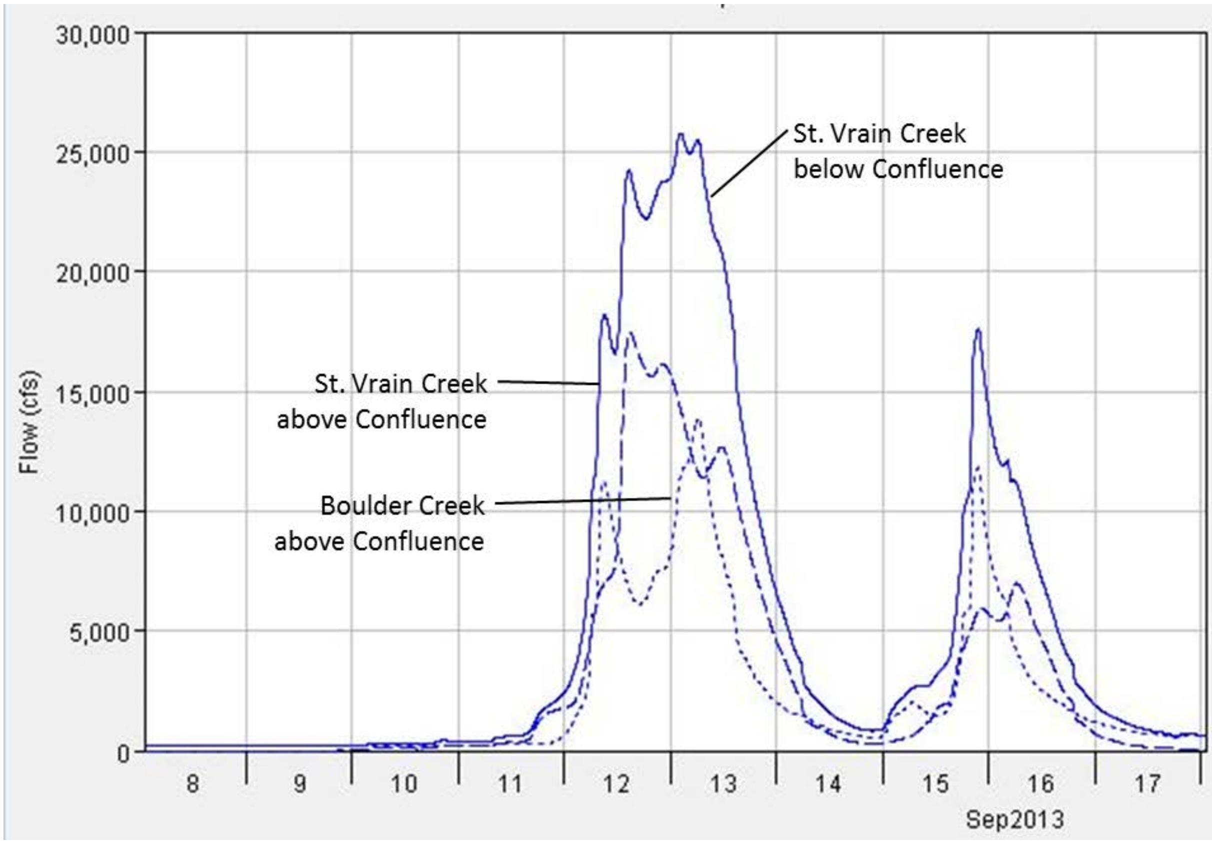
Appendix D.9 provides several plots from the HEC-HMS model which show locations where the model was calibrated to the 2013 Flood. The first plot compares the partial gage record from the USGS gage on St. Vrain Creek in Lyons to the upstream model node on St. Vrain Creek at the Highway 36 Bridge below Lyons. The peak discharge measurements are limited since the gage was washed out but the stage measurements provide a good indication of the timing of the rising limb of the hydrograph and it compares favorably with the calibrated model results. The modeled peak discharge also compares well with the peak discharge estimate developed by Bob Jarrett as part of the Phase 1 Hydrologic Evaluation (estimate discussed in detail in Section 2.2 of Phase 1 Report).

Two of the next three plots in Appendix D.9 show the modeled hydrographs along St. Vrain Creek at Airport Road and Highway 287 where peak discharge estimates were determined for the 2013 Flood. The other plot is at the CDWR gage near Hover Road which recorded the rising limb of the hydrograph. The timing of the recorded discharge and stage measurements matches well with the model results at this location.

The next four plots in Appendix D.9 show the modeled hydrographs along Lefthand Creek at 63rd Street, the Diagonal Highway, Highway 287, and Ken Pratt Boulevard where discharge estimates were determined for the 2013 Flood. The remaining plots in Appendix D.9 are all on St. Vrain Creek downstream of the confluence with Lefthand Creek. The first of these is at the Highway 119 Bridge just downstream of Lefthand Creek where the stream gage is located. Stage measurements were recorded during the 2013 Flood and the timing of the rising limb is in good agreement with the model results. This plot also shows how the hydrographs from St. Vrain Creek (Element R2120) and Lefthand Creek (J92A) overlap resulting in an increase in the downstream peak discharge on St. Vrain Creek.

The next plot in Appendix D.9 shows the modeled hydrograph on St. Vrain Creek just upstream of the confluence with Boulder Creek where the USGS gage recorded the first few hours of the 2013 Flood rising limb before washing out. The recorded discharge was limited at the gage but the rising limb of the modeled hydrograph matched up well indicating that the timing of the peak discharges at the confluence with Boulder Creek are accurate. The peak discharges on St. Vrain Creek and Boulder Creek overlapped considerably in the 2013 Flood as indicated by the downstream peak discharge estimate at Interstate 25 and the model calibration. This is not surprising considering the relatively similar travel times and the overall storm pattern across the watershed. Figure 6 shows the overlapping of the modeled hydrographs at the confluence with Boulder Creek for the 2013 Flood model.

Figure 6. Model Results at Confluence of St. Vrain and Boulder Creek in 2013 Flood



The next three plots in Appendix D.9 show comparisons between the model results and peak discharge estimates at Interstate 25, State Highway 66, and County Road 34. The last plot in Appendix D.9 is shows a comparison between model results and the partial record at the USGS gage at the mouth of St. Vrain Creek near Platteville. The timing of the rising limb for both the discharge and stage measurements line up well with the model.

A concerted effort was made not to over calibrate the model to match all peak discharge estimates. Instead, a systematic approach was taken in the calibration process to ensure a consistent method was used throughout all of the watersheds studied. The goal was to obtain the best overall fit to the majority of the peak discharge estimates rather than try to match them all at the expense of calibration parameters being pushed beyond a reasonable range. The systematic approach prevents individual basins in the model from being biased toward unique occurrences associated with this particular storm event. Although the model has been calibrated to the 2013 flood event, the end goal is to develop a hydrologic model capable of representing storms of various magnitudes.

The calibrated model results for the NOAA 24-hour predictive storms on St. Vrain Creek and Lefthand Creek are also shown on Figures 4 and 5, respectively. The predictive model peak discharges for the various return periods were compared to the results from the updated FFA as well as to current regulatory discharges.

On St. Vrain Creek (Figure 4 or Appendix D.6), the predictive model results between Lyons and the confluence with Boulder Creek had the same general shape as the current regulatory discharges but they were offset by 15 to 30 percent. This offset originated in Lyons at the confluence of South St. Vrain Creek and North St. Vrain Creek, primarily due to increases in the discharge from North St. Vrain Creek as described in the Phase 1 Hydrologic Evaluation Report. This peak discharge increase in Lyons translated downstream through Longmont for all of the recurrence intervals (discharge profiles generally move parallel to each other). As described in Section 2.3, the FFA at the gage upstream of Longmont only included 12 years of record and only the 10-year results are worth comparing based on the confidence limits. The 10-year FFA results at this gage matched up well with both the predictive model results and the current regulatory discharges. The FFA for the St. Vrain Creek gage just upstream of the confluence with Boulder Creek produced results that were well below both the predictive model results and the current regulatory discharges. As described in Section 2.3, the record only included 34 years and the highest recorded discharge was only 3,600 cfs, five times less than the 2013 peak discharge estimate. The 2013 estimate was treated as an outlier in the FFA and the results are considered low.

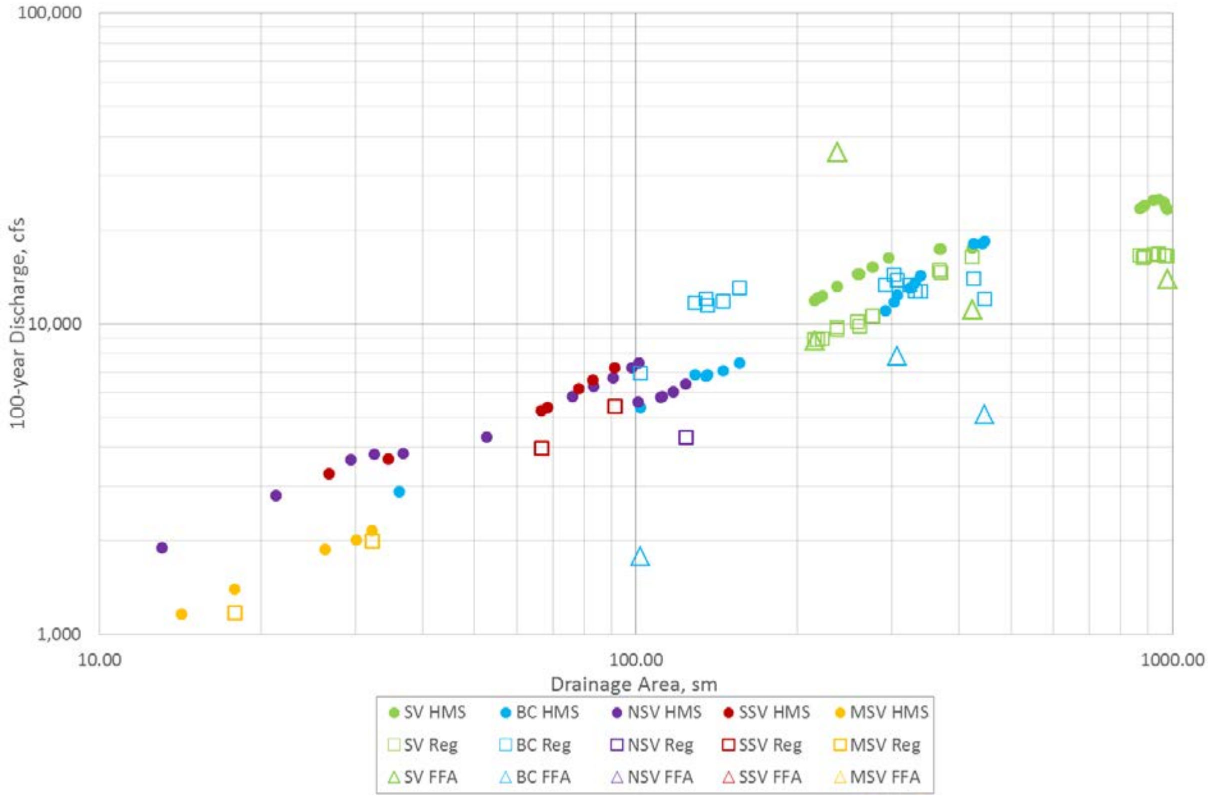
At the confluence with Boulder Creek, the predictive model shows a jump in the peak discharge profile on St. Vrain Creek due to the overlap in hydrographs at this location. St. Vrain Creek (424 square miles) and Boulder Creek (447 square miles) have similar tributary areas and travel times so it is not surprising that the peak discharges overlap at the confluence. In contrast, the current regulatory discharges upstream and downstream of the confluence are almost identical. This was a direct result of the modeling approach used in the 1981 USACE Report discussed in Section 2.1. The regulatory peak discharges on St. Vrain Creek above the Boulder Creek confluence were produced by centering a 6-hour storm over the St. Vrain watershed above the confluence with Boulder Creek, but excluding the area above Button Rock Dam (109 square miles) and assuming very little discharge from North St. Vrain Creek. From the confluence with Boulder Creek to the mouth of St. Vrain Creek, the peak discharges were developed by applying a 6-hour storm to the entire St. Vrain Creek watershed including Boulder Creek. A 6-hour storm results in more intense rainfall and produces runoff hydrographs with a tall, narrow shape (2-3 hour peaks). As these individual hydrographs are routed downstream they are less likely to overlap with other hydrographs which results in smaller peak discharges for large watersheds. In contrast, a 24-hour storm produces broader hydrographs which are more likely to overlap downstream in a large watershed. This likely explains why there is no rise in the current regulatory peak discharges on St. Vrain Creek at the confluence with Boulder Creek.

The peak discharges on St. Vrain Creek downstream of the confluence with Boulder Creek remain relatively constant all the way to the South Platte River. A slight attenuation of peak discharges is observed in this reach but the model also accounts for inflow from smaller tributaries such as Godding Hollow which tend to balance it out. The FFA for the St. Vrain Creek gage at the mouth produced results that were well below both the predictive model results and the current regulatory discharges. Although the gage has 87 years of record, the annual maximum peaks are highly influenced by irrigation practices in the watershed as indicated by the highest recorded annual peak of 11,300 cfs for an 978 square mile watershed (12 cfs/mi^2). The 2013 peak discharge estimate of 27,000 cfs shows that these irrigation practices though are not sufficient control large floods, hence the assumption that their effects are ignored in the predictive model.

On Lefthand Creek (Figure 5 or Appendix D.7), the predictive model results between Highway 36 and the confluence with St. Vrain Creek matched up well with the current regulatory discharges. The regulatory peak discharges attenuated from 6,700 cfs at Highway 36 to 4,610 cfs at the confluence with St. Vrain Creek. The 1981 report states that the reduction of overbank storage by future development would tend to reduce the attenuation affect, thus causing an increase in peak discharges in the lower reaches of the study. The predictive model does not account for overbank storage in the lower reaches since this section has been channelized and is designed to contain the 100-year peak discharge, resulting in slightly higher peak discharges at the confluence with St. Vrain Creek.

The predictive peak discharges were also compared against flood frequency results and current regulatory discharges to get a sense for how the different sources of discharge estimates compare (see Figure 7).

Figure 7. Comparison of 100-year Discharges in the St. Vrain Watershed



Watershed (color):
SV = St. Vrain Creek (green)
BC = Boulder Creek (blue)
NSV = North St. Vrain Creek (purple)
SSV = South St. Vrain Creek (red)
MSV = Middle St. Vrain Creek (orange)

Analysis Method/Data Source (marker shape):
HMS = HEC-HMS Calibrated Model (filled circle)
Reg = FIS Regulatory Peak Discharge (square)
FFA = Flood Frequency Analysis (triangle)

The following observations can be made from Figure 7 regarding the Phase 2 Study Area:

1. Compared to the modeled discharges, more scatter is associated with the current regulatory discharges and flood frequency discharge estimates.
2. The current regulatory discharges on St. Vrain Creek below the confluence with Boulder Creek (far right side) start to drop whereas the predictive model continues the linear trend.

Appendix D.8 includes two additional plots of discharge versus area. The first includes discharges in the Lefthand Creek watershed. The second compares discharges on the Big Thompson River, Little Thompson River, St. Vrain Creek, Lefthand Creek, and Boulder Creek. The common trend further supports the predictive model results over the scatter in current regulatory peak discharges.

4.0 CONCLUSIONS AND RECOMMENDATIONS

This report documents a hydrologic investigation of Lower St. Vrain Creek (South Platte River upstream to Highway 36 at Lyons) associated with the extreme flood event of September, 2013. Peak discharges experienced during the flood were estimated and compared to current regulatory discharges, shown in Table 8 below. Based on the current regulatory discharges, the September 2013 flood ranged from a 100-year event to greater than a 500-year event in some locations. However, based on the original modeling assumptions for North St. Vrain Creek and the Boulder Creek confluence used to develop the regulatory peak discharges it is recommended that the actual recurrence interval of the 2013 flood be based on the updated predictive discharges developed in this evaluation.

Table 8. Estimate of September 2013 Peak Discharge Recurrence Interval based on Current Regulatory Discharges

	2013 Effective FIS Peak Discharge				Ayres 2013 Updated				2013 Flood	2013 Flood
	Approximate Location for Comparison				Flood Frequency Analysis				Estimated	Estimated
Description	10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	Peak Discharge (cfs)	Recurrence Interval (yrs)
St. Vrain Creek at Hwy 36 below Lyons	2,040	6,670	8,880	20,260					23,000	> 500 Year
St. Vrain Creek at Airport Road	3,160	6,890	9,580	19,680	2,990	17,250	35,720	186,890	14,000	100 to 500
St. Vrain Creek at Highway 287	4,110	8,240	10,580	21,200					14,500	100 to 500
St. Vrain Creek below Lefthand Creek	5,250	10,950	14,850	28,670					18,500	100 to 500
St. Vrain Creek above Boulder Creek	6,010	12,500	16,440	31,790	3,050	7,720	11,150	25,100		
St. Vrain Creek at Interstate 25	6,070	12,500	16,510	41,960					23,500	100 to 500
St. Vrain Creek at State Highway 66	5,920	12,900	16,760	41,900					23,000	100 to 500
St. Vrain Creek at County Road 34	5,520	12,400	16,560	40,590					27,000	100 to 500
St. Vrain Creek at Gage near Mouth	5,410	12,200	16,540	40,230	5,090	10,650	13,990	24,790		
Lefthand Creek at Highway 36	1,035	4,145	6,700	14,990						
Lefthand Creek at 63rd Street	860	3,800	6,600	14,590					7,000	~ 100
Lefthand Creek at Diagonal Highway	750	3,500	6,330	13,990					8,700	100 to 500
Lefthand Creek at Highway 287					920	3,330	5,490	16,150	5,000	~ 100
Lefthand Creek at South Pratt Parkway	520	2,480	4,610	10,320					4,800	~ 100

An updated flood frequency analysis was also performed as part of this study to reflect annual peak flows that have occurred since prior gage analyses, including estimated peak discharges from the 2013 Flood. Backup information associated with the gage analyses for the St. Vrain

Creek and Lefthand Creek gages are provided in Appendix B. Table 8 shows a summary of the updated flood frequency analyses for the St. Vrain watershed. The flood frequency analysis results tended to be low on St. Vrain Creek due to irrigation impacts. The FFA on Lefthand Creek though matched well with the predictive model results.

A HEC-HMS rainfall/runoff model was developed and calibrated to match the peak discharge estimates obtained for the 2013 flood event. The first step in this process was to calibrate rainfall information representing the September storm to match available ground data throughout the study watersheds. This is described in Section 2.4.5. The rainfall data was incorporated as 5-minute incremental rainfall hyetographs for a 10-day period around the 2013 flood event. The second step was to incorporate inflow hydrographs for Upper St. Vrain Creek, Upper Lefthand Creek and Boulder Creek which were developed in separate models. The third step was to calibrate the model by adjusting Curve Number, channel roughness and channel losses to obtain a best fit of the model results to the peak discharge estimates. This model was calibrated to the full 10-day period. The fourth step was to apply NOAA point precipitation depths for various recurrence intervals using a 24-hour SCS Type II rainfall distribution to develop predictive peak discharges. To better represent a 24-hour storm as opposed to the long duration September event, the model was re-calibrated based on the maximum 24-hour period of rainfall from the 2013 flood event. Once the curve numbers were adjusted to provide a best fit with the 2013 peak discharge estimates, the design rainfall (adjusted using DARF curves) was applied. The results of this predictive model are summarized in Table 7 and in Appendix D. Table 9 compares the predictive peak discharges from this modeling effort to current regulatory discharges for the 100-year event.

Table 9. 100-year Modeled Peak Flows Compared to Current Regulatory Discharges

Location	Current Regulatory Discharge (cfs)	Modeled Discharge (cfs)	Percent Difference
St. Vrain Creek at Hwy 36 below Lyons	8,880	12,100	+36%
St. Vrain Creek at Airport Road	9,580	13,200	+38%
St. Vrain Creek at Highway 287	10,580	15,200	+44%
St. Vrain Creek below Lefthand Creek	14,850	17,400	+17%
St. Vrain Creek above Boulder Creek	16,440	17,500	+6%
St. Vrain Creek at Interstate 25	16,510	24,100	+46%
St. Vrain Creek at State Highway 66	16,530	25,100	+52%
St. Vrain Creek at County Road 34	16,560	24,600	+49%
St. Vrain Creek at South Platte River	16,520	23,400	+42%
Dry Creek No. 1 at St. Vrain Creek	2,315	2,750	+19%
Spring Gulch (The Slough) at St. Vrain Creek	3,650	4,340	+19%
Lefthand Creek at Highway 36	6,700	5,820	- 13%
Lefthand Creek at 63rd Street	6,600	5,990	- 9%
Lefthand Creek at Diagonal Highway	6,330	6,040	- 5%
Lefthand Creek at St. Vrain Creek	4,610	5,740	+25%
Dry Creek No. 2 at St. Vrain Creek	2,600	4,920	+89%
Boulder Creek at St. Vrain Creek	12,000	18,500	+54%

The assumptions and limitations of various hydrologic methodologies used for development of the current regulatory discharges and for those used in this study were closely reviewed, compared, and contrasted. Based on this evaluation, the results of the current rainfall-runoff model using the 24-hour NOAA rainfall are viewed as suitable for use by CDOT in the design of permanent roadway improvements along St. Vrain Creek. In addition, the results of this modeling effort will be made available to local agencies for their consideration in revising discharges currently used for regulatory purposes. As described below, the rainfall/runoff model results better reflect the peak discharges in North St. Vrain Creek (Phase 1) and the overlap in hydrographs at the Boulder Creek confluence. Therefore it is recommended that the model results be considered for adoption as the updated regulatory peak discharges along St. Vrain Creek. It should be noted that this study was focused on peak discharge estimation in St. Vrain Creek and Lefthand Creek and was not developed with the intention of replacing regulatory values in the smaller tributaries. Additional analysis is recommended for smaller tributaries to evaluate shorter, more intense storms.

The 35 to 45 percent difference in 100-year peak discharges between Lyons and Longmont can be attributed to the fact that the current regulatory peak discharges were based on the assumption that Button Rock Dam would store runoff from North St. Vrain Creek and this tributary area was not included in the original model used to develop discharge estimation. The 2013 flood is evidence that this assumption was not conservative enough and that significant peak discharges from the reservoir can occur causing flood damage downstream. In contrast, the predictive model developed as part of this study only accounts for attenuation of peak discharges as they pass through the Button Rock Dam Spillway, conservatively assuming the reservoir is full prior to the start of the storm.

The 40 to 50 percent difference in 100-year peak discharges downstream of the Boulder Creek confluence can be attributed to the fact that the current regulatory peak discharges were based on a 6-hour storm over the entire St. Vrain watershed (including Boulder Creek). The current regulatory peak discharges upstream and downstream of Boulder Creek are essentially identical which indicates that the Boulder Creek 6-hour hydrograph peak does not overlap at all with the St. Vrain Creek 6-hour hydrograph peak. This is largely because a shorter, more intense rainfall produces a tall, narrow discharge hydrograph which is less likely to overlap with other downstream discharge hydrographs in the model. In contrast, the predictive model developed as part of this study used a 24-hour storm over the entire St. Vrain watershed (including Boulder Creek). The longer duration storm produces peak discharge hydrographs with a much broader shape and more potential to overlap other hydrographs downstream. In the case of the Boulder Creek confluence, the predictive model resulted in a combined peak discharge that was approximately 65 percent of the direct sum of the two tributary peak discharges. This indicates that the two peak discharge hydrographs overlapped but the instantaneous peak discharges were offset slightly.

Based on the predictive model discharges for the return periods analyzed, as shown in Table 10 below, the peak discharge observed along St. Vrain Creek during the September 2013 flood event was approximately a 1 percent annual chance peak discharge (100-year storm) downstream of Lyons. Lefthand Creek experienced between a 0.2 percent annual chance peak discharge and a 2 percent annual chance peak discharge from upstream to downstream based on the predictive model.

Table 10. Estimate of September 2013 Peak Discharge Recurrence Interval based on Model Results

Location	Drainage Area (mi^2)	Measured Discharge (cfs)	Annual Chance Peak Discharge (cfs)					Estimated Recurrence Interval (yr)
			10%	4%	2%	1%	0.2%	
St. Vrain Creek at Hwy 36 Bridge (D-15-I)	218	23,000	2,200	4,860	7,950	12,100	26,600	100 to 500
St. Vrain Creek at Airport Road	237	14,000	2,360	5,280	8,570	13,200	29,000	~ 100
St. Vrain Creek at Highway 287	276	14,500	3,590	5,990	9,720	15,200	33,700	~ 100
St. Vrain Creek below Lefthand Creek	368	18,500	4,740	7,370	11,900	17,400	40,100	~ 100
St. Vrain Creek at Interstate 25	889	23,500	6,740	11,900	17,800	24,100	43,500	~ 100
St. Vrain Creek at State Highway 66	942	23,000	6,840	12,400	18,500	25,100	45,500	~ 100
St. Vrain Creek at County Road 34	965	27,000	6,710	12,100	18,100	24,600	45,400	~ 100
Lefthand Creek at 63rd Street	63	7,000	1,510	2,840	4,250	5,990	11,800	100 to 500
Lefthand Creek at Diagonal Highway	69	8,700	1,400	2,820	4,270	6,040	11,800	100 to 500
Lefthand Creek at Highway 287	72	5,000	1,380	2,640	4,060	5,810	11,600	50 to 100
Lefthand Creek at Ken Pratt Blvd.	72	4,800	1,370	2,580	3,990	5,740	11,400	50 to 100

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TECHNICAL APPENDICES

Appendix A

2013 Peak Discharge Estimates

URS

Technical Memorandum

Date: Thursday, February 12, 2015

To: Steven Griffin, CDOT- Region 4
Kevin Houck, Colorado Water Conservation Board

From: William Carrier, P.E.

Subject: **ESTIMATED PEAK DISCHRGES – PHASE 2**

Introduction

In late summer 2013, the Colorado Front Range experienced an extensive rainstorm event spanning approximately ten days from September 9th to September 18th. The event generated widespread flooding as the long-duration storm saturated soils and increased runoff potential. Flooding resulted in substantial erosion, bank widening, and realigning of stream channels; transport of mud, rock and debris; failures of dams; landslides; damage to roads, bridges, utilities, and other public infrastructures; and flood impacts to many residential and commercial structures. Ten fatalities were attributed to the floods.

During and immediately following the rainstorm event, the Colorado Department of Transportation (CDOT) engaged in a massive flood response effort to protect the traveling public, rebuild damaged roadways and bridges to get critical travel corridors open again, and engage in assessments and analyses to guide longer term rebuilding efforts. As part of this effort, CDOT partnered with the Colorado Water Conservation Board (CWCB) to initiate hydrologic analyses in several key river systems impacted by the floods. The work was contracted to three consultant teams led by the following firms.

Boulder Creek, Little Thompson River	CH2M HILL
Big Thompson River, St. Vrain Creek, Lefthand Creek	Jacobs
Coal Creek, South Platte River	URS

The purpose of the analyses is to ascertain the approximate magnitude of the September flood event in key locations throughout the watershed and to prepare estimates of peak discharge that can serve to guide the design of permanent roadway and other infrastructure improvements along the impacted streams. These estimates of peak discharges for various return periods will be shared with local floodplain administrators for their consideration in revising or updating any current regulatory discharges.

The primary tasks of the hydrologic analyses include:

1. Estimate peak discharges that were believed to have occurred during the flood event at key locations along the study streams. Summarize these discharges along with estimates provided

by others in comparison to existing regulatory discharges. Document the approximate return period associated with the September flood event based on current regulatory discharges.

2. Prepare rainfall-runoff models of the study watersheds, input available rainfall data representing the September rainstorm, and calibrate results to provide correlation to estimated peak discharges.

3. Prepare updated flood frequency analyses using available gage data and incorporate the estimated peak discharges from the September event.

4. Use rainfall-runoff models to estimate predictive peak discharges for a number of return periods based on rainfall information published by the National Oceanic and Atmospheric Administration (NOAA) [NOAA Atlas 14, Volume 8, Updated 2013]. Compare results to updated flood frequency analyses and unit discharge information and calibrate as appropriate.

The hydrologic analyses were divided into two phases of work. Phase 1 focused on the mountainous areas in the upper portion of the watersheds, extending from the upper divides of the Big Thompson River, Little Thompson River, St. Vrain Creek, Lefthand Creek, Coal Creek, and Boulder Creek watersheds to the mouth of their respective canyons. The Phase 1 analyses have been documented in six reports with the following titles and dates.

1. Hydrologic Evaluation of the Big Thompson Watershed, August 2014
2. Little Thompson River Hydrologic Analysis Final Report, August 2014
3. Hydrologic Evaluation of the St. Vrain Watershed, August 2014
4. Hydrologic Evaluation of the Lefthand Creek Watershed, August 2014, revised December 2014
5. Coal Creek Hydrology Evaluation, August 2014
6. Boulder Creek Hydrologic Evaluation Final Report, August 2014

Copies of these Phase 1 reports can be downloaded from the CWCB website at the following link:

http://cwcb.state.co.us/water-management/flood/pages/2013floodresponse.aspx

Phase 2 of the hydrologic analyses focused on the plains region of the Big Thompson River, Boulder Creek, Little Thompson River, and St. Vrain Creek from the downstream limit of the Phase 1 studies at the mouth of the canyons to the downstream confluences of the watersheds with their respective receiving streams. The hydrologic analyses were contracted to two consultant teams led by the following firms:

Boulder Creek, Little Thompson River	CH2M HILL
Big Thompson River, St. Vrain Creek	Jacobs

Phase 2 hydrologic analyses for each of the watersheds included flows from the original Phase 1 watersheds, as appropriate: the downstream reach of the Big Thompson River was modeled to include flows from the Little Thompson River. Likewise, the downstream reach of St. Vrain Creek included flows from Lefthand Creek and Boulder Creek, with Boulder Creek in turn receiving flows from Coal Creek.

This Memorandum documents the Phase 2 the high water estimation at designated locations along the watersheds. The purpose of the analyses is to ascertain the approximate magnitude of the September flood event in key locations throughout the watersheds and to prepare estimates of peak discharge that can serve to guide the design of permanent roadway and other infrastructure improvements along the impacted streams.

Methodology

Collection of Data:

URS sent a survey team to each bridge location that was to be calibrated with the high flow. . At each location, the team surveyed at least four cross sections that included the main channel and the floodplain. The locations were surveyed even though pre-flood models existed as the flood changed the topography of the landscapes. A minimum of four cross sections is are needed to properly evaluate flows by the modeling program, HEC-RAS, in order to properly evaluate flows at each location; a cross section directly upstream and downstream of the bridge, a cross section located upstream of the bridge roughly the distance of the bridge opening upstream of the bridge (1:1 opening), and a downstream cross section located about four times the bridge opening downstream of the bridge (4:1 opening). These distances are based on approximate expansion and contraction zones as recommended by the HEC-RAS manual. Additional cross sections were surveyed at a location if deemed necessary due to increased complexity at a location such as drop structures near the bridge or bends in the area.

During the surveys, the team looked for evidence of high water marks from the September 2013 floods. This included debris in bushes, trees, bridges, or a high point on the ground. These points were recorded during the survey as high water marks. In order to help with calibration, the locations of these points were near the surveyed cross sections.

In addition, information about the bridges was collected in order to properly model each location. The information collected included, the width of the bridge, the length of the bridge opening, the number of piers, the width of the piers, the location of the piers, abutment information, the distance from the bottom of the channel to the low chord of the of the bridge (the bridge opening), the distance from the bottom of the bridge to top of the guard rails, and any other bridge information deemed necessary for use in the modeling software.

Processing of Data:

Once the data was collected, it was transformed from the local surveying system to the Northern Colorado State Plane System where each point in the cross section had a northing, an easting, and an elevation. The surveyed cross sections and high water marks were exported into ESRI shapefiles. These were then reviewed for accuracy and completeness in ArcMap. The data was converted into excel format and exported to HEC-RAS. The left side facing downstream of each cross section was initially set as Station 0. There were about 30 to 50 surveyed points for each cross section. The distances between the cross sections were used to assign the river station with the most downstream cross section arbitrarily labeled as station 1000.

In some cases, the field surveyed cross sections did not extend far enough to contain flows in the modeled cross section. This occurred in areas where the floodplain was extremely wide, exceeding 2,000 feet in width or in locations that were adjacent to rock and gravel quarry ponds. In these instances, the surveyed cross sectional data was supplemented with post flood LiDAR data. The LiDAR was used to create a digital elevation model (DEM) to extract elevation points.

<u>HEC-RAS Modeling:</u>

HEC-RAS, Version 4.1.0, is a 1-dimensional step backwater river analysis system created by the United States Army Corps of Engineers. It was selected due to wide spread use, prominent use in previous models at the same locations, and the many tools for bridge modeling that exist in this software.

Many of the locations had existing HEC-RAS (or HEC-2) models from when the bridges were designed and constructed and were provided by CDOT. In these cases, the bridge data was already available and stations were adjusted to reflect these models. For all locations, the new surveyed cross sections were added into the HEC-RAS model. The bridge data was also verified with the field survey data. For locations without existing models, the bridge data recorded in the field was included as well.

The Manning's "n" values in the model were selected based upon field conditions and existing model values. In order to test the sensitivity of the flow in relation to the Manning's value, the Manning's value was increased and decreased in at least two (2) models on each stream, Big Thompson, Little Thompson, and St. Vrain. Results of this sensitivity analysis are summarized below.

The contraction and expansion coefficients were selected based on recommendations used in the HEC-RAS manual. To properly model bridges, ineffective areas were added to the upstream and downstream of bridges to account for the flow contraction and expansion at the bridge openings. For upstream of the bridge, there was a 1:1 contraction ratio meaning at the bridge the ineffective area would extend at a 45 degree angle to the bridge. Downstream of the bridge, a 3:1 expansion ratio was modeled. Generally, the ineffective areas extend for the two cross sections upstream and downstream of the river. In some cases, they were extended into additional cross sections depending on the width of the floodplain and cross section versus the bridge opening.

The bridges were modeled using the Energy Equation with over topping weir coefficient of 2.6. The energy Equation was selected as the High Flow Bridge Modeling Method. This method was selected as the majority of the bridges modeled were not overtopped, and as a result pressure and/or weir flow was not present.

Once the model parameters were complete, the estimated flow at each location was adjusted until the model water surface elevation approached the high water marks. In the case where the high water marks couldn't be matched well with the all of the cross sections, emphasis was placed on the cross section just downstream or upstream of the bridge The downstream locations provided a better representation of free flow during the flood event as compared to the upstream locations that could have potentially had backups and created artificially high debris marks.

For each model, subcritical and subcritical flow regimes were run and each calibrated to the surveyed high water mark.

Results

Most of the sites had consistent correlation between the field observations and the results of the model at each location. Generally, the calculated water surface elevations were within 0.1 feet of the observed high water elevation with a few exceptions. Subcritical flow modeling produced a more consistent match of water surface values. This could be attributed to the mild slopes of the channel in the lower reaches located in the plains and the wide floodplains. In some locations such as at Coal and Rock Creek, running the model as supercritical resulted in more accurate results as both of these tributaries have steeper slopes and more incised channels.

For some sites, the HEC-RAS model was unable to match the field observations. This was mainly due to overtopping of the bridge or nearby road. The high water survey occurred months after the floods and in some cases emergency repairs had been performed making it difficult to locate high water locations. There were also few photos from which to estimate the flood widths. For the points that overtopped, the high water mark was assumed as the top of the bridge rail.

The models had little sensitivity to changes in the Manning's n values. For the models tested, a 0.01 change in the Manning's value resulted in variance of less than 5% in the modeled flows. This held true regardless of the magnitude of the flows from the smaller flows 1,500 cfs to larger flows exceeding 20,000 cfs.

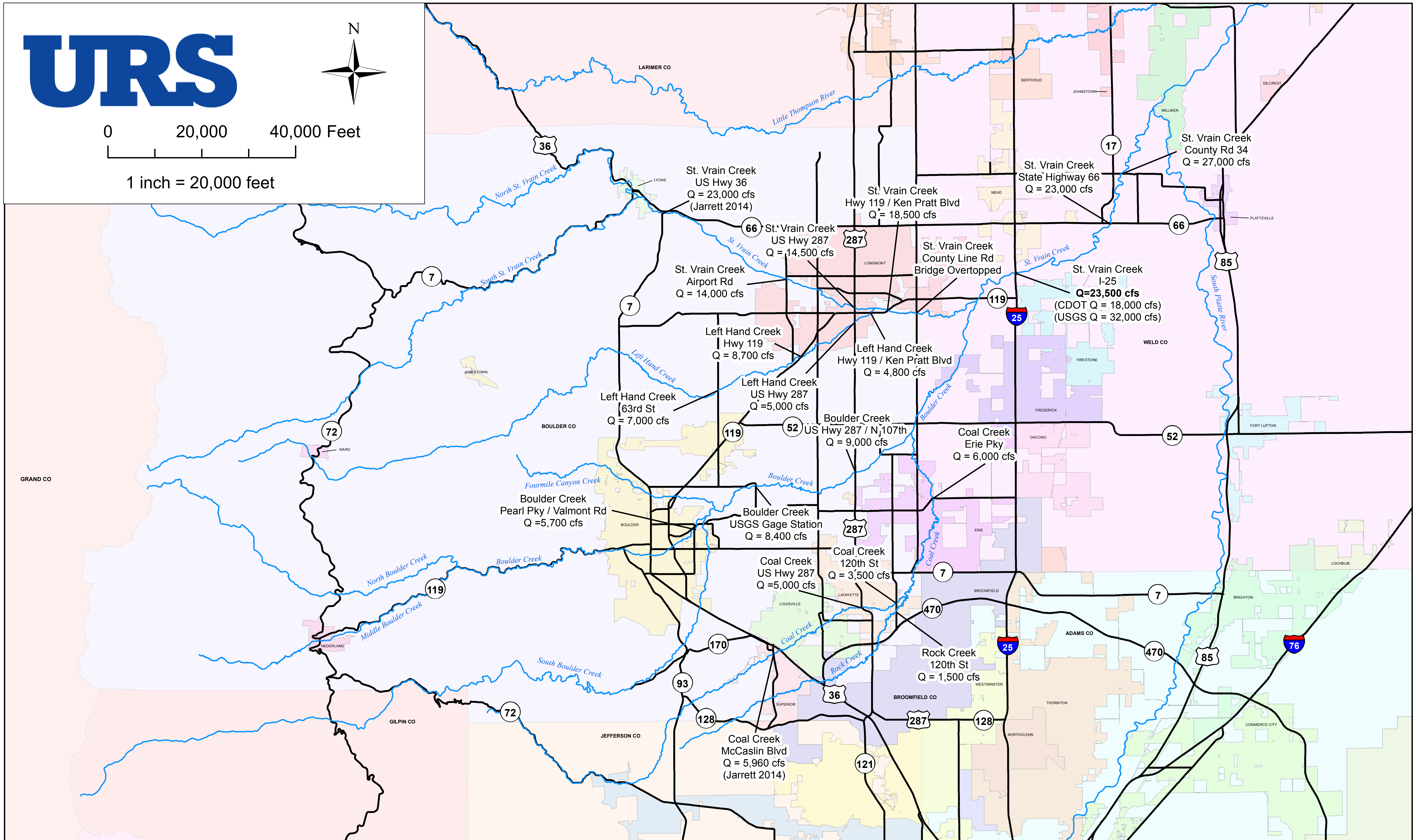
The following table summarizes the discharge estimates, the high water marks, and the calculated water surface elevation, and comments regarding each location.

Summary of Estimated Discharges for September 2013

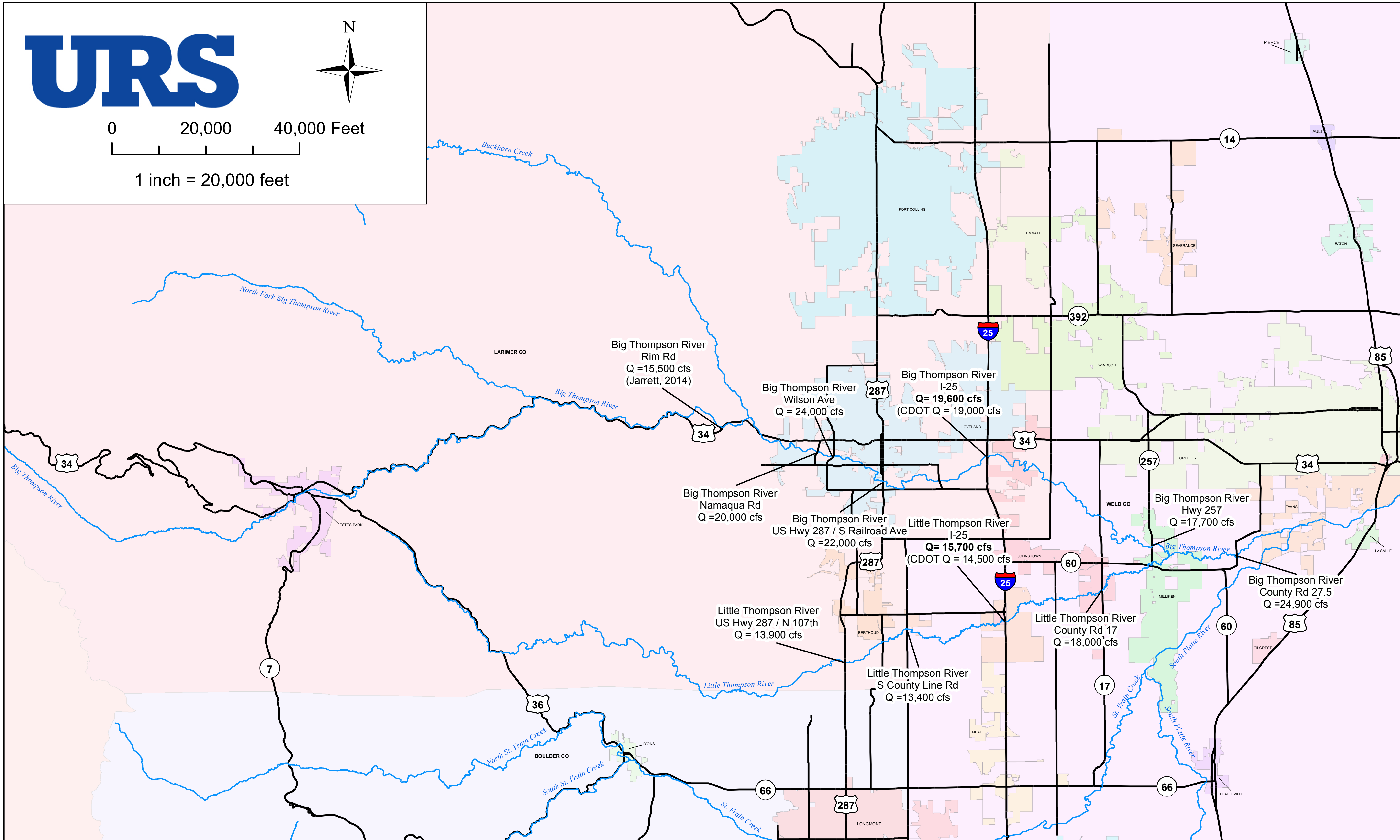
Location			Discharge (cfs)	High Water Elevation (ft) NAVD 88	Water Surface Elevation (ft) NAVD 88	Comments
Little Thompson						
1	At N 107th Crossing (287)		13,900	4998.73	4998.74	
2	At S County Line Road Crossing	FEMA Point	13,400	4938.17	4938.58	
3	At I-25 Crossing		15,700	4857.11	4857.12	
4	At County Road 17 Crossing		18,000			Bridge overtopped/unreliable
Big Thompson						
1	Namaqua Road *		20,000	5002.42	5002.04	Area very hard to calibrate given ponds and overtopping.
2	Wilson Avenue*		24,000	4990.07	4990.26	Flows rates based on downstream ponds being full.
3	S. Railroad Avenue or Hwy 287	FIS Location	22,000	4933.3	4933.3	
4	I-25	FIS Location	19,600	4849.91	4849.97	3,000 cfs overtopped I-25 north of cross section.
5	County Line Road (Larimer-Weld)	FIS Location	8,800	4813.44	4813.47	Unreliable results. Bridge was overtopped.
6	U/S of Confl with Little Thompson (Hwy 257, CR 21)		17,700	4746.7	4746.73	
7	D/S of Confl with Little Thompson (CR 25)					No Model
8	County Road 27.5		24,900	4701.93	4701.93	
Boulder Creek						
1	Boulder Creek at Pearl Pky / Valmont Road	FEMA Point	5,700	5200.51	5200.49	4300 cfs at subcritical flows
2	Boulder Creek at N 107 Street/Boulder 287		9,000	5016.35	5016.38	
3	Coal Creek at Bridge Street (N of Erie)	FEMA Point				No Model
4	Coal at Erie		6,000	5021.267	5021.66	
5	Coal Creek at Highway 287		5,000	5206.66	5206.65	Possible attenuation /blowouts DS of structure
6	Coal Creek at the Confluence with Rock Creek	FEMA Point				No Model
7	Rock Creek at S 120th Street	FEMA Point	1,500	5149.65	5149.8	
8	Coal Creek At 120th		3,500	5140.59	5140.5	
Lefthand						
1	N. 63rd St.		7,000	5159.71	5159.7	
2	Diagonal Highway (Hwy 119 near Airport Road)		8,700	5019.09	5019.07	Model does not account for influence of railroad bridge.
3	Hwy 287 (Main Street)		5,000	4950.17	4950.7	
4	U/S of Confl with St. Vrain (Hwy 119/Ken Pratt Blvd.)	FIS Location	4,800	4937.36	4937.36	
St. Vrain						
1	85th Street/Airport Road	FIS Location	14,000	5027.85	5027.77	No Bridge in HEC-RAS model.
2	U/S of Confluence w/ Lefthand Creek (US Hwy 287)		14,500	4948.87	4949.37	
3	D/S of Confl. w/ Lefthand Creek and UIS of Confl w/ Boulder Creek (Hwy 119/Ken Pratt Blvd)		18,500	4924.81	4924.29	
4	County Line Road (Boulder-Weld)	FIS Location				Not a good point-road washed out around the bridge, downstream work completed.
5	D/S of Confl. w/ Boulder Creek (1-25)		23,500	4834.93	4834.73	
6	State Hwy 66 (CR 30)		23,000	4791.11	4791.13	
7	Country Road 34		27,000	4770.88	4770.88	

*Recommended flow value of 22,000 cfs.

Peak Flow Discharge for the September 2013 Floods - St. Vrain Creek Drainage Area



Peak Flow Discharge for the Spetember 2013 Floods - Big Thompson River Drainage Area



As previously mentioned, for most locations high water elevation observed in the field correlated well with the calculated water surface elevations in the models. There were a few exceptions. A summary of the model results for each stream reach are included below.

Little Thompson River

1. North 107th Crossing (US Hwy 287) - The cross section directly upstream of the bridge was calibrated to the high water mark. The calculated water surface for the downstream cross sections did not match well with the surveyed high water marks. This was due to the bridge overtopping and may have resulted in a split flow into the adjacent farmland.
2. S County Line Road – No issues. The model correlated very well.
3. I-25-For this location there were three bridges modeled, North I-25, South I-25 and the frontage road to the east. This location gave good results which allowed it to be calibrated at three different high water marks. Both cross sections on either side of the frontage road were calibrated and the most upstream cross section was calibrated.
4. County Road 17- This road overtopped and as a result gave unreliable results.

Big Thompson River

The Namaqua Road and Wilson Avenue locations were very difficult to determine flow rates. The locations have numerous quarry ponds directly upstream and downstream of each location. When the sections were modeled, the water surface elevation was assumed to be 1 foot below the pond embankment. Because these ponds occupy approximately 1,500 feet of the floodplain, the actual water surface elevation plays a large role in the flow calculation. A 1 foot increase or decrease in the water surface of the ponds varies the flow by approximately 1,000 cfs. In addition, flows jumped the northern bank upstream of the Namaqua Road crossing.

1. Namaqua Road – Flows estimated at 20,000 cfs but, it is recommended that flows be averaged with Wilson Avenue crossing. Suggested value of 22,000cfs.
2. Wilson Avenue – See Namaqua Road note.
3. Hwy 287 – Model correlated well to high water marks.
4. I-25- This location has three different bridges, North I-25, South I-25 and the frontage road. For modeling purposes, the cross section between the two I-25 bridges was calibrated to the high water location. The water surface elevation was 4849.9’. The flow value includes 3,000 cfs that overtopped I-25 north of the cross section road.
5. County Line Road (Larimer-Weld) - This didn’t yield reliable results as it overtopped the road.

6. Hwy 257-This location was calibrated to the section just upstream of the bridge to a water surface elevation of 4746.73’.

7. Downstream confluence with Little Thompson – No model developed.

8. County Road 27.5 – Model match field observations.

The cross sections at Namaqua Road, Wilson Avenue and US Hwy 287 were supplemented with LiDAR data to fully contain the flow and be calibrated correctly.

<u>Boulder Creek:</u>

1. Boulder Creek at Pearl Parkway and Valmont Road - This section was calibrated to the upstream section.

2. Boulder Creek at 287 - This section was calibrated to just downstream of the bridge and has an extra cross section both down and upstream.

<u>Rock and Coal Creek:</u>

3. Coal Creek at Bridge Street (N of Erie) – No Model developed due to limited access.

4. Coal Creek at Erie - At this location three bridges were modeled: one for a pedestrian bridge before the road, one for the road, and one for a railroad bridge downstream. It was calibrated to the cross section just before the road bridge. Reliability of the estimated flow is questionable due to the complexity of the model.

5. Coal Creek at Highway 287- Here there was some attenuation possible as well as blowouts of downstream of the structure.

6. Coal Creek at the Confluence with Rock Creek – No Model developed as high water elevation could no e determined.

7. Rock Creek at 120th Street - An additional cross section was modeled upstream of the bridge. The calibration point here is the cross section just downstream of the bridge.

8. Coal Creek at 120th- An additional cross section was modeled upstream of the bridge. The calibration point here is the cross section just downstream of the bridge.

The calibration of the confluence of Coal Creek and Rock Creek was not modeled as the high water mark was difficult to establish.

<u>Lefthand Creek:</u>

1. N. 63rd St. - This location was calibrated to the most downstream cross section. The two upstream cross sections were close to the surveyed high water marks.

2. Diagonal Highway (Hwy 119 near Airport Road) - Two separate bridges were modeled for this location. The most downstream cross section was added using LiDAR data. The cross section between the two bridges was the calibration point.

3. Hwy 287- The cross section just downstream at this location was used for the calibration point.

4. Hwy 119/ Ken Pratt Blvd- This model included two additional cross sections upstream of the bridge.

St. Vrain Creek

1. Hwy 287/Airport Road – Model correlated well to the observed high water marks. However, bridge information was not available and therefore not included in the HEC-RAS model. There were no bridge as-built plans available and at the time of the survey, the creek flows were too great to safely perform a bridge survey.

2. U/S of Confluence w/ Lefthand Creek (US Hwy 287) – Model matched survey data.

3. Hwy 119/Ken Pratt Blvd. - This section had two extra cross sections upstream and downstream to help increase the accuracy of the model. The upstream cross section and the cross section just downstream of the bridge were used as calibration points.

4. County Line Road (Boulder-Weld) - This location was not modeled. The road on both sides of the bridge had washed away and there had been downstream work completed.

5. I-25 - In this location, it was modeled as two bridges. The drop structure downstream of the bridges was also added. The structure was not in the original model. The model was calibrated to the upstream face of the upstream bridge.

6. State Highway 66 (CR33) – model match field observations. The bridge was replaced as part of the emergency repairs.

7. County Road 34 – No Issues.

REFERENCES

US Army Corps of Engineers (USACE) HEC-RAS River Analysis System, Ver. 4.1.0, January 2010.

CDOT hydraulic models

Appendix A

HEC-RAS Results

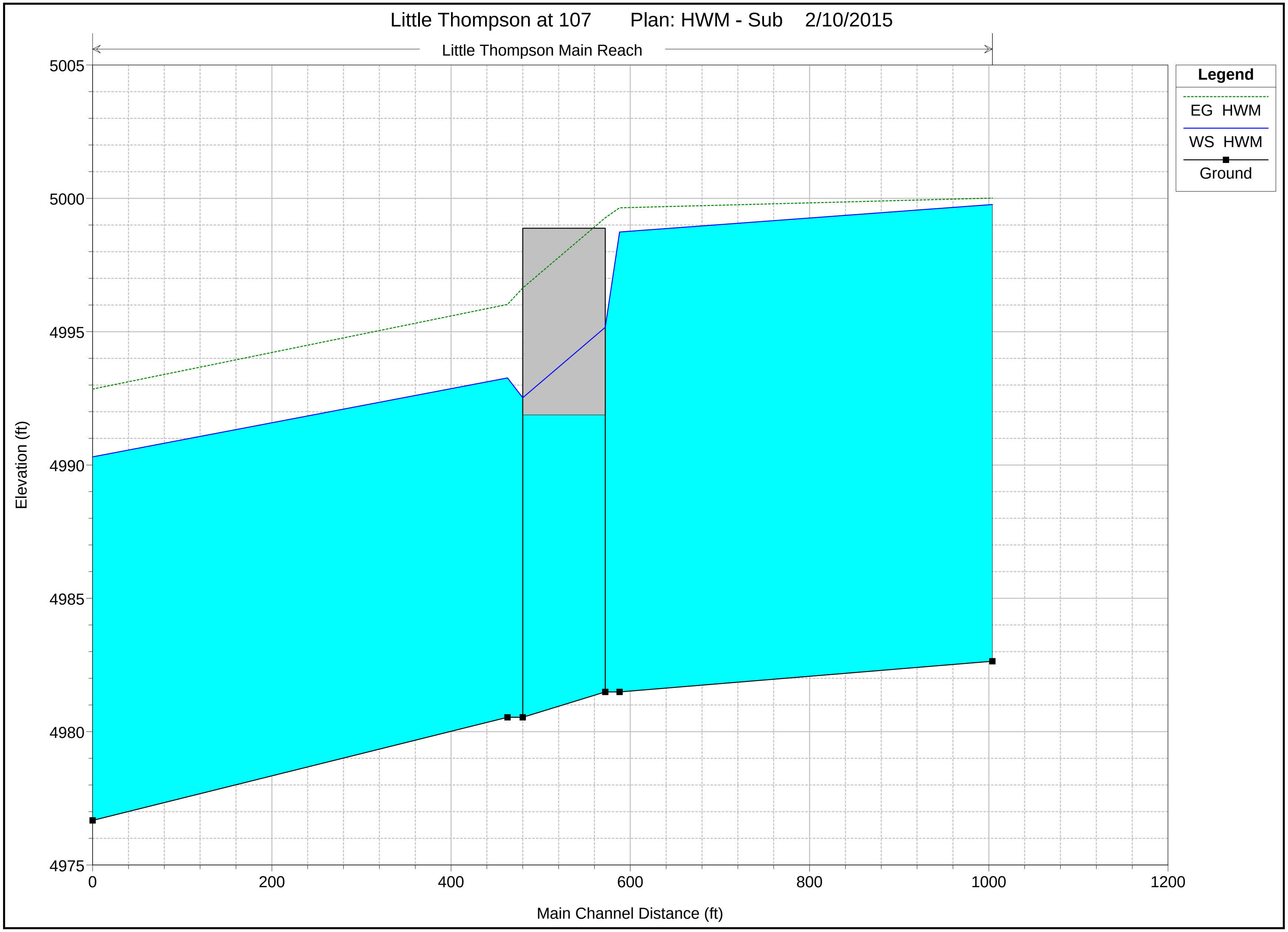
Little Thompson

HEC-RAS Results

Little Thompson at 107th

HEC-RAS Plan: HWM - Sub   River: Little Thompson   Reach: Main Reach   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Reach	1104	HWM	13900.00	4982.64	4999.77	4992.93	5000.01	0.000459	4.65	3771.28	337.29	0.22
Main Reach	688	HWM	13900.00	4981.49	4998.74	4992.27	4999.65	0.001247	8.07	1892.70	224.74	0.36
Main Reach	625		Bridge									
Main Reach	563	HWM	13900.00	4980.54	4993.26	4992.22	4996.02	0.006745	13.63	1086.86	158.47	0.77
Main Reach	100	HWM	13900.00	4976.67	4990.30	4990.30	4992.85	0.006698	14.99	1316.99	224.29	0.76

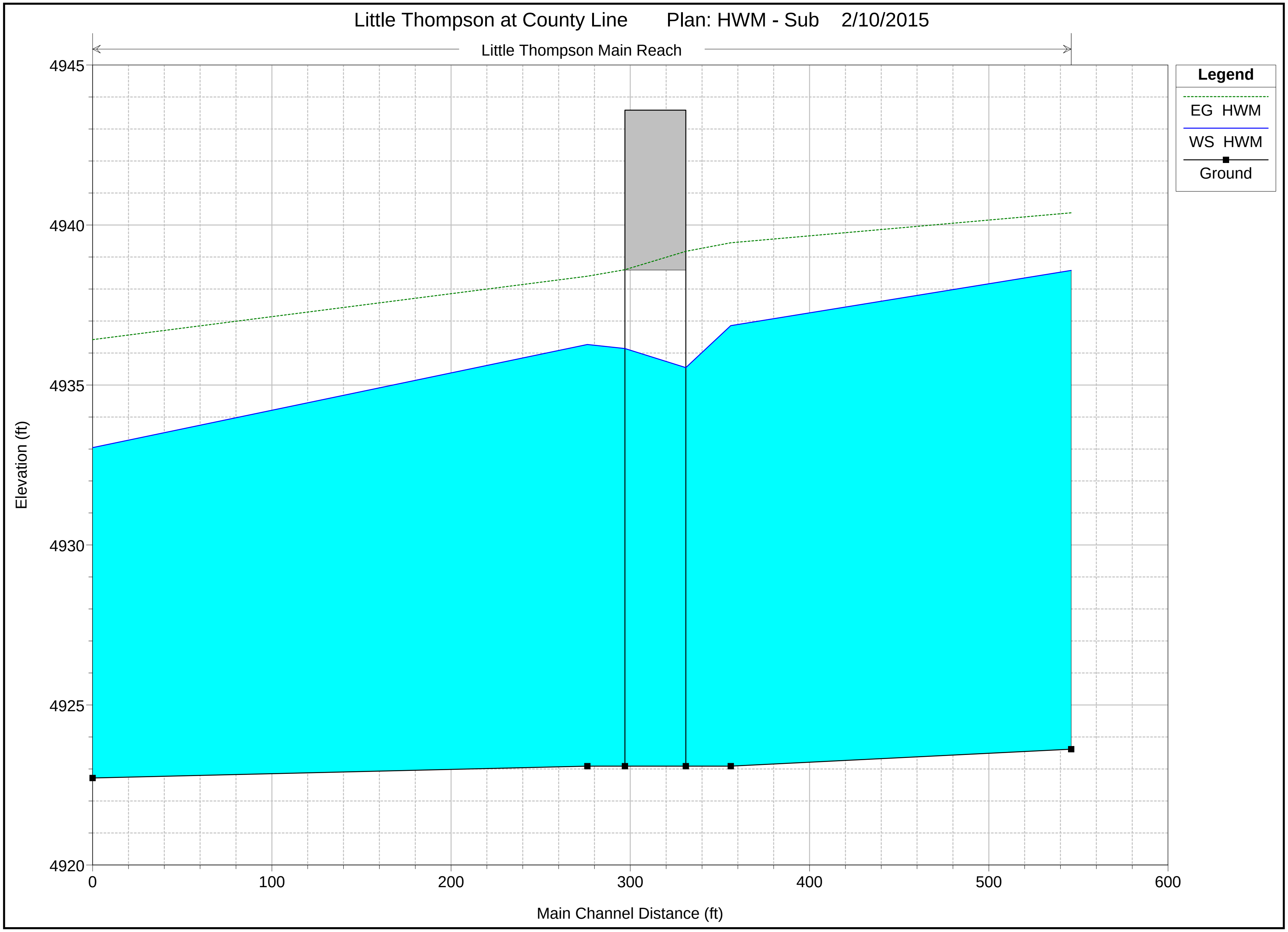

Little Thompson at 107   Plan: HWM - Sub   2/10/2015
Little Thompson Main Reach
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
5005
5000
4995
4990
4985
4980
4975
0
200
400
600
800
1000
1200
Main Channel Distance (ft)

Little Thompson at S County Line

HEC-RAS Plan: HWM - Sub   River: Little Thompson   Reach: Main Reach   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Reach	675	HWM	13400.00	4923.62	4938.58	4936.06	4940.38	0.003837	12.09	1361.73	144.75	0.59
Main Reach	460	HWM	13400.00	4923.09	4936.85	4934.82	4939.44	0.005135	13.29	1078.23	153.78	0.69
Main Reach	420		Bridge									
Main Reach	381	HWM	13400.00	4923.09	4936.27	4933.71	4938.40	0.004802	11.87	1152.04	150.34	0.65
Main Reach	100	HWM	13400.00	4922.72	4933.04	4933.04	4936.42	0.010002	15.26	983.02	159.49	0.92

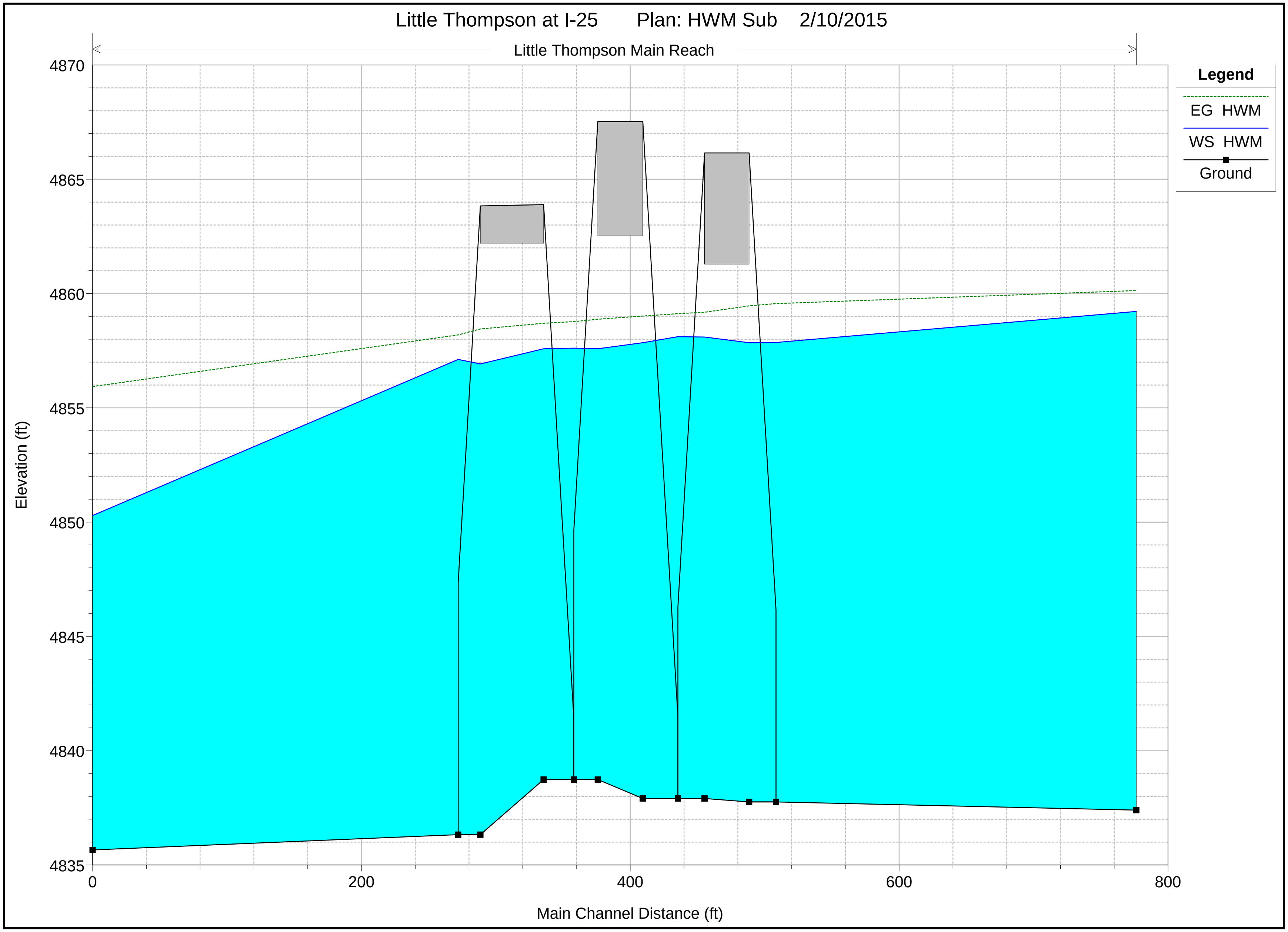
Little Thompson at S County Line



Little Thompson at I-25

HEC-RAS Plan: HWM - Sub   River: Little Thompson   Reach: Main Reach   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Reach	2595.635	HWM	15700.00	4837.40	4859.22	4853.35	4860.13	0.001358	8.57	2069.01	219.51	0.35
Main Reach	2327.635	HWM	15700.00	4837.76	4857.86	4853.99	4859.56	0.002620	12.67	1744.94	170.12	0.52
Main Reach	2307.635		Bridge									
Main Reach	2254.640	HWM	15700.00	4837.91	4858.12	4850.56	4859.12	0.001423	8.17	2010.57	152.53	0.38
Main Reach	2228.62		Bridge									
Main Reach	2177.256	HWM	15700.00	4838.74	4857.61	4851.13	4858.77	0.001664	10.19	2053.02	182.47	0.43
Main Reach	2154.767		Bridge									
Main Reach	2091.285	HWM	15700.00	4836.33	4857.12	4850.02	4858.19	0.001588	10.42	2114.56	177.43	0.42
Main Reach	1819.285	HWM	15700.00	4835.66	4850.29	4850.29	4855.93	0.010311	20.10	881.95	82.62	0.99

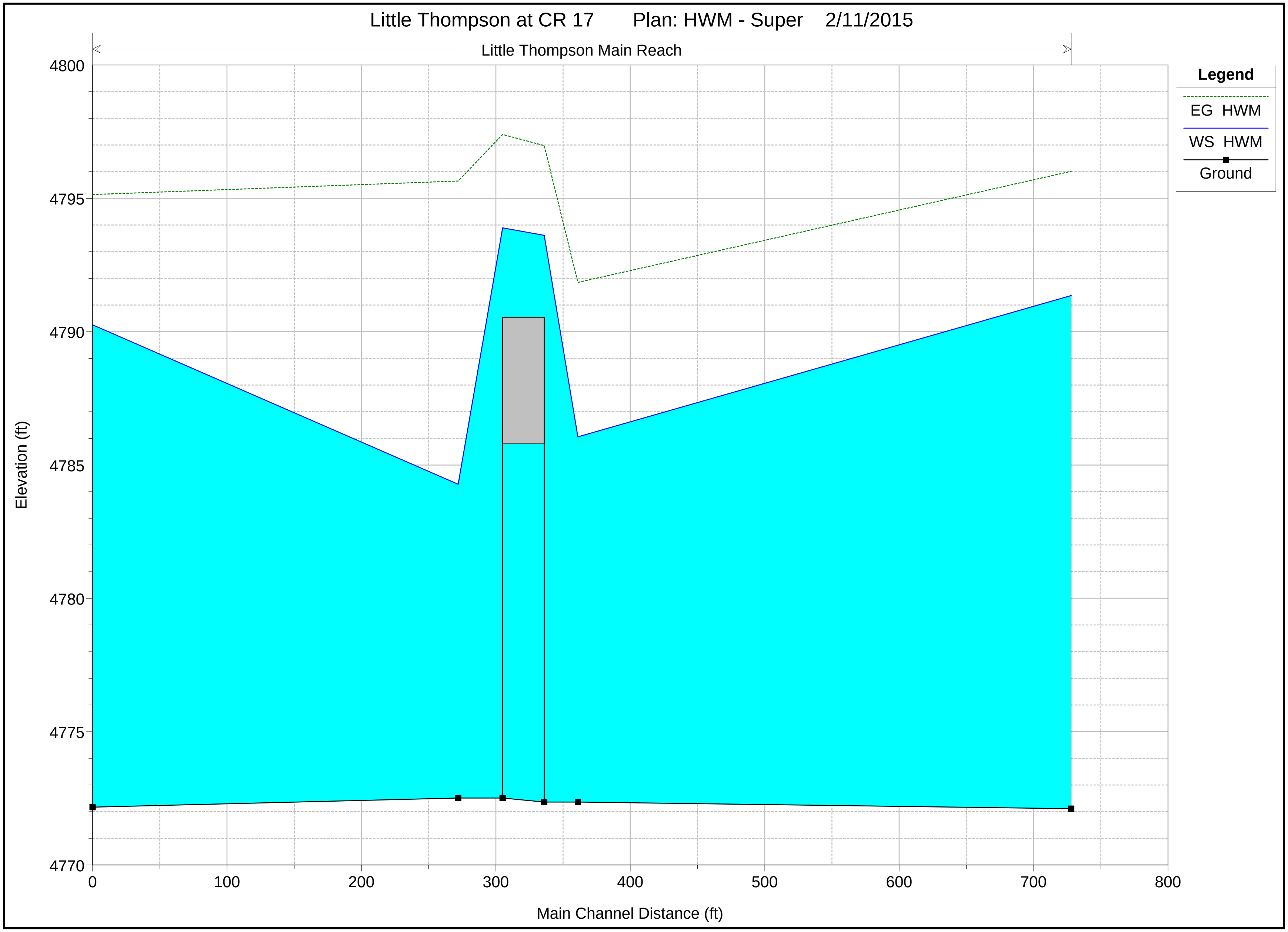


Little Thompson at I-25 Plan: HWM Sub 2/10/2015
Little Thompson Main Reach
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4870
4865
4860
4855
4850
4845
4840
4835
0
200
400
600
800
Main Channel Distance (ft)

Little Thompson at CR 17

HEC-RAS Plan: HWM - Super River: Little Thompson Reach: Main Reach Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Reach	828	HWM	18000.00	4772.11	4791.35	4791.35	4796.01	0.009333	19.37	1163.14	119.08	0.89
Main Reach	461	HWM	18000.00	4772.36	4786.06	4786.87	4791.85	0.013259	19.61	962.00	131.26	1.07
Main Reach	417		Bridge									
Main Reach	372	HWM	18000.00	4772.51	4784.28	4787.77	4795.65	0.032953	27.31	689.21	139.47	1.61
Main Reach	100	HWM	18000.00	4772.17	4790.26	4790.26	4795.14	0.009435	20.71	1130.05	108.67	0.91


Little Thompson at CR 17 Plan: HWM - Super 2/11/2015
Little Thompson Main Reach
4800
4795
4790
4785
4780
4775
4770
Elevation (ft)
0
100
200
300
400
500
600
700
800
Main Channel Distance (ft)
Legend
EG HWM
WS HWM
Ground

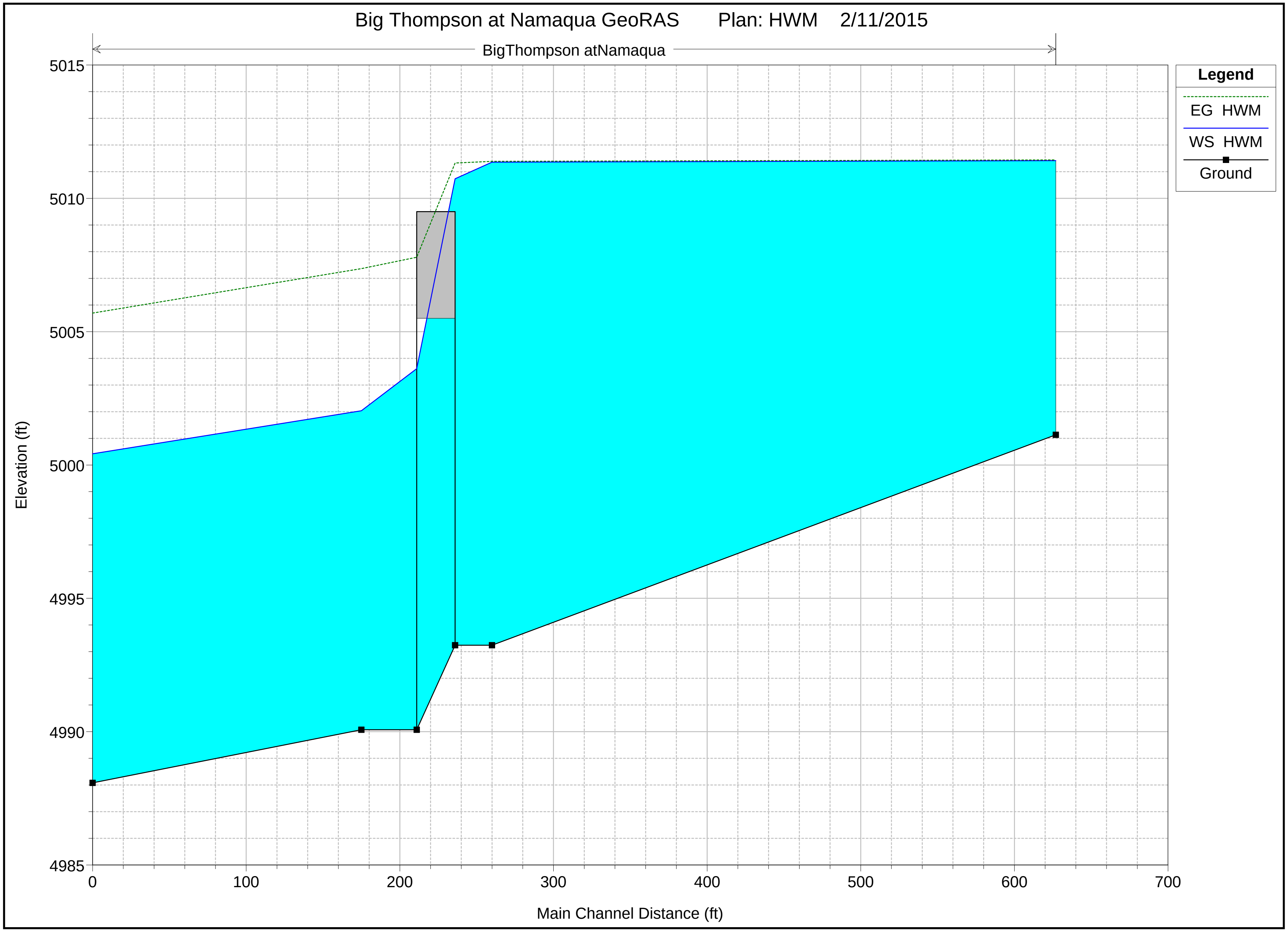
Big Thompson

HEC-RAS Results

Big Thompson at Namaqua

HEC-RAS Plan: HWM - Sub   River: BigThompson   Reach: atNamaqua   Profile: HWM

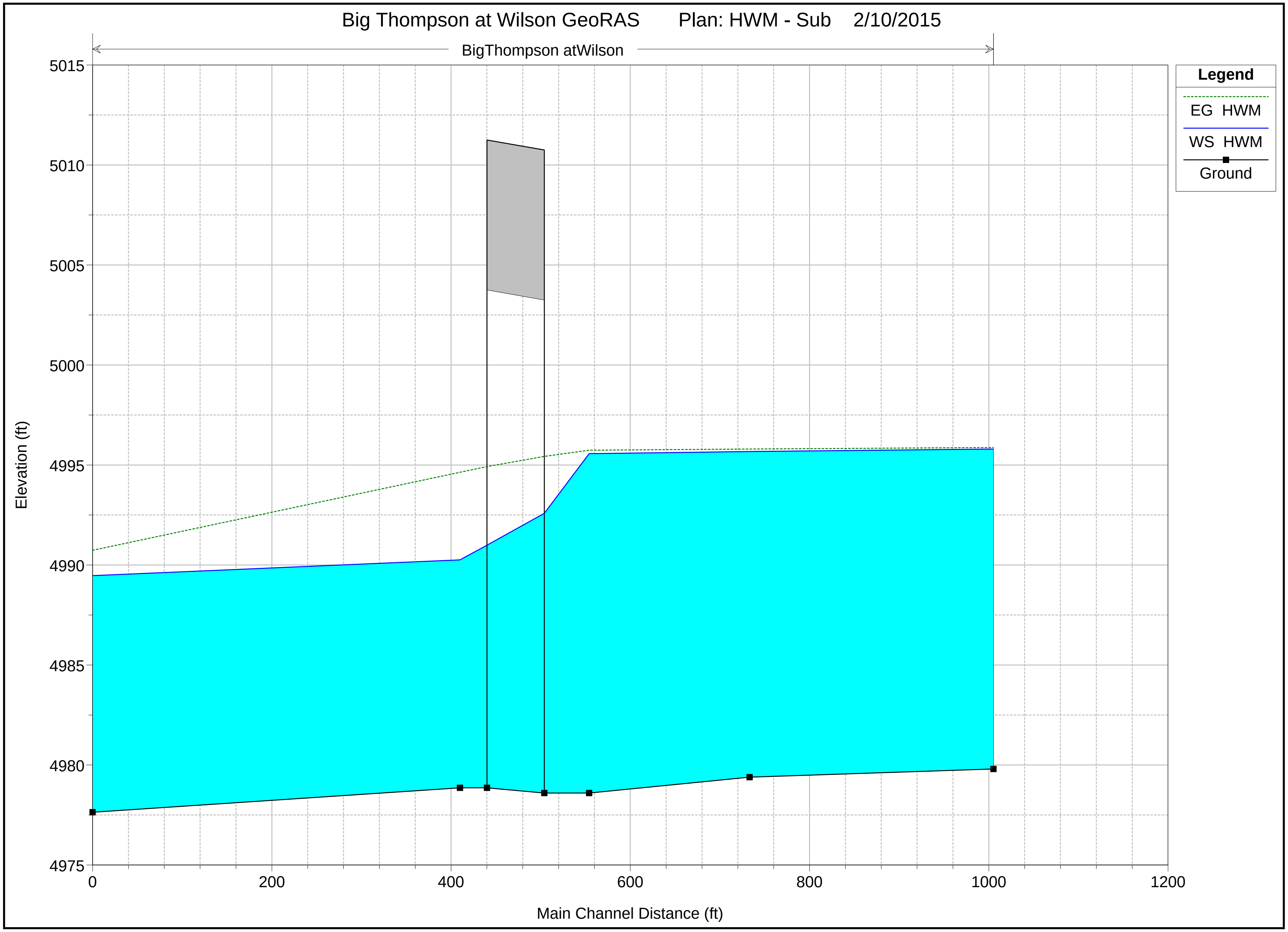
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
atNamaqua	906.0839	HWM	20000.00	5001.13	5011.41	5007.42	5011.44	0.000185	2.62	16549.89	3325.48	0.15
atNamaqua	539.4245	HWM	20000.00	4993.24	5011.35	5005.94	5011.39	0.000110	2.89	17214.33	2673.16	0.12
atNamaqua	512.7339		Bridge									
atNamaqua	454.7339	HWM	20000.00	4990.07	5002.04	5001.46	5007.36	0.008801	19.67	1206.31	631.62	1.04
atNamaqua	279.7508	HWM	20000.00	4988.08	5000.42	5000.42	5005.70	0.010125	18.87	1210.15	764.12	1.08


Big Thompson at Namaqua GeoRAS Plan: HWM 2/11/2015
BigThompson atNamaqua
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
5015
5010
5005
5000
4995
4990
4985
0
100
200
300
400
500
600
700
Main Channel Distance (ft)

Big Thompson at Wilson

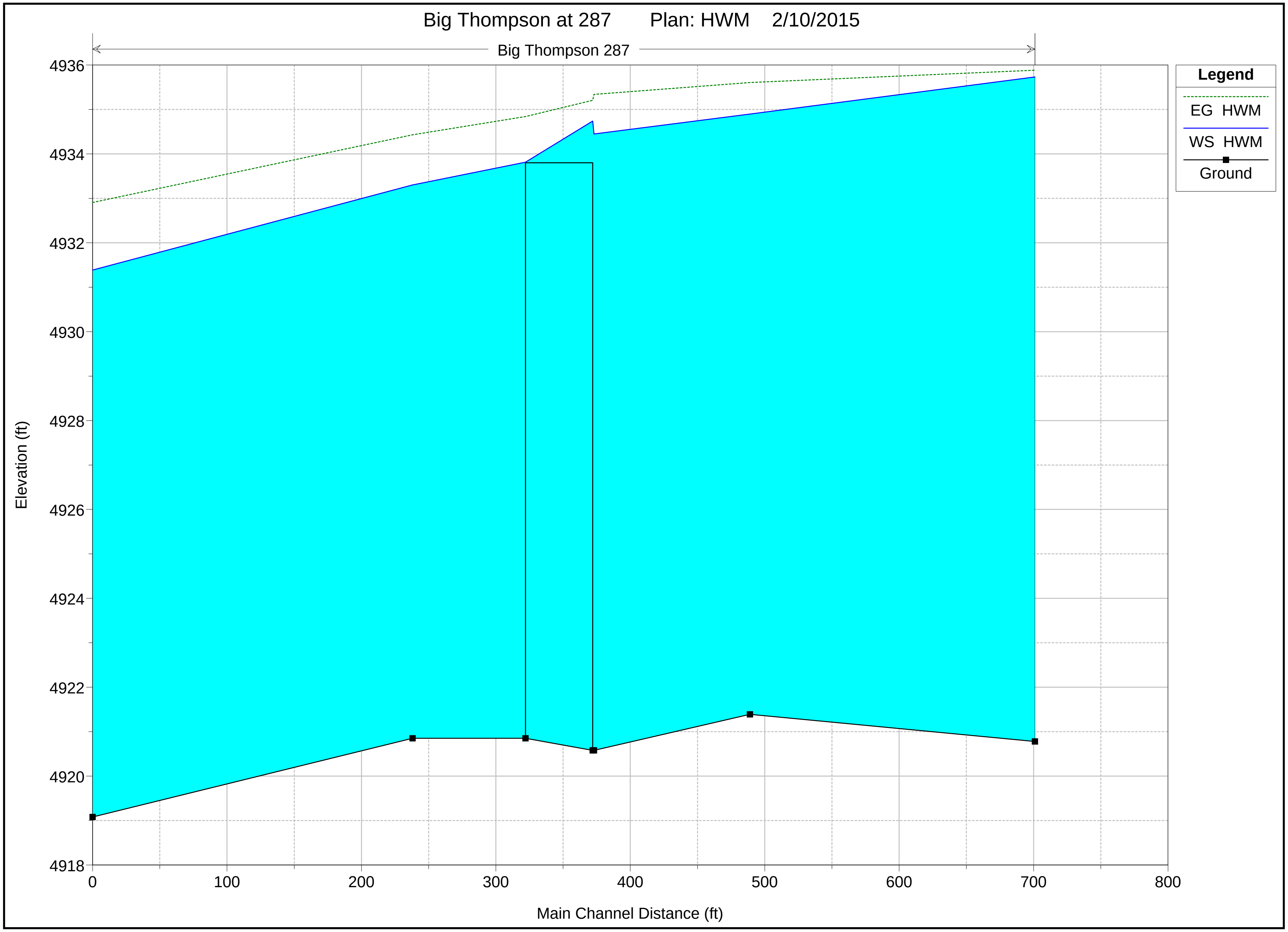
HEC-RAS Plan: HWM - Sub River: BigThompson Reach: atWilson Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
atWilson	1357.954	HWM	24000.00	4979.80	4995.80		4995.88	0.000217	3.71	14225.99	2419.93	0.17
atWilson	1085.827	HWM	24000.00	4979.39	4995.67		4995.80	0.000304	4.59	12241.83	2230.26	0.20
atWilson	906.7098	HWM	24000.00	4978.60	4995.57	4989.97	4995.74	0.000346	4.87	10839.57	1895.19	0.22
atWilson	852.6332		Bridge									
atWilson	762.6332	HWM	24000.00	4978.86	4990.26	4990.26	4994.64	0.008262	16.96	1467.20	675.66	0.97
atWilson	352.6946	HWM	24000.00	4977.64	4989.47	4989.47	4990.74	0.003542	12.20	4308.02	2150.21	0.65


Big Thompson at Wilson GeoRAS Plan: HWM - Sub 2/10/2015
BigThompson atWilson
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
5015
5010
5005
5000
4995
4990
4985
4980
4975
0
200
400
600
800
1000
1200
Main Channel Distance (ft)

HEC-RAS Plan: HWM - Sub   River: Big Thompson   Reach: 287   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
287	1201	HWM	22000.00	4920.78	4935.73	4930.86	4935.88	0.000626	5.56	9575.16	2281.27	0.27
287	989	HWM	22000.00	4921.39	4934.90	4933.99	4935.61	0.002111	9.49	5569.71	2200.01	0.50
287	873	HWM	22000.00	4920.58	4934.45	4933.82	4935.34	0.002171	10.62	5347.52	2444.13	0.53
287	872		Bridge									
287	738	HWM	22000.00	4920.85	4933.30	4933.30	4934.43	0.002943	11.69	4691.34	2662.32	0.61
287	500	HWM	22000.00	4919.08	4931.39	4931.39	4932.91	0.006289	15.01	4018.16	3198.84	0.84

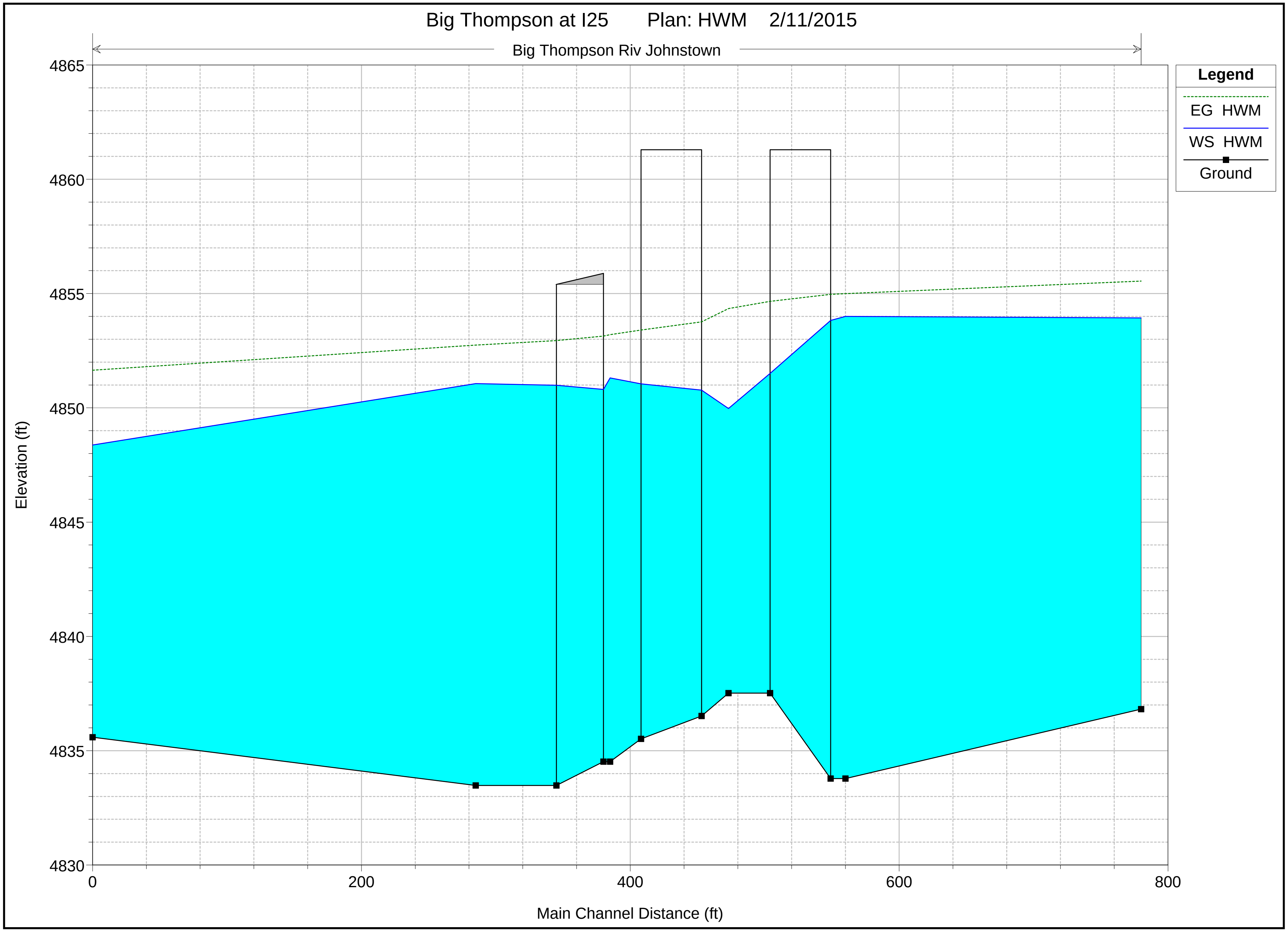


Big Thompson at 287 Plan: HWM 2/10/2015
Big Thompson 287
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4936
4934
4932
4930
4928
4926
4924
4922
4920
4918
0
100
200
300
400
500
600
700
800
Main Channel Distance (ft)

HEC-RAS Plan: HWM - Sub River: Big Thompson Riv Reach: Johnstown Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Johnstown	1780	HWM	16600.00	4836.82	4853.93	4850.12	4855.54	0.002358	12.05	1918.19	163.39	0.54
Johnstown	1560	HWM	16600.00	4833.78	4854.00	4848.48	4855.00	0.001224	8.61	2247.86	321.14	0.40
Johnstown	1548		Bridge									
Johnstown	1473	HWM	16600.00	4837.52	4849.97	4849.97	4854.34	0.007920	18.81	1142.53	133.26	0.98
Johnstown	1453		Bridge									
Johnstown	1385	HWM	16600.00	4834.52	4851.31	4846.97	4853.20	0.002352	12.69	1761.16	152.12	0.56
Johnstown	1380		Bridge									
Johnstown	1285	HWM	16600.00	4833.48	4851.06	4846.08	4852.74	0.001838	10.83	1681.49	315.43	0.49
Johnstown	1000	HWM	16600.00	4835.59	4848.37	4848.37	4851.65	0.007493	16.81	1332.76	290.97	0.93

Big Thompson at I-25

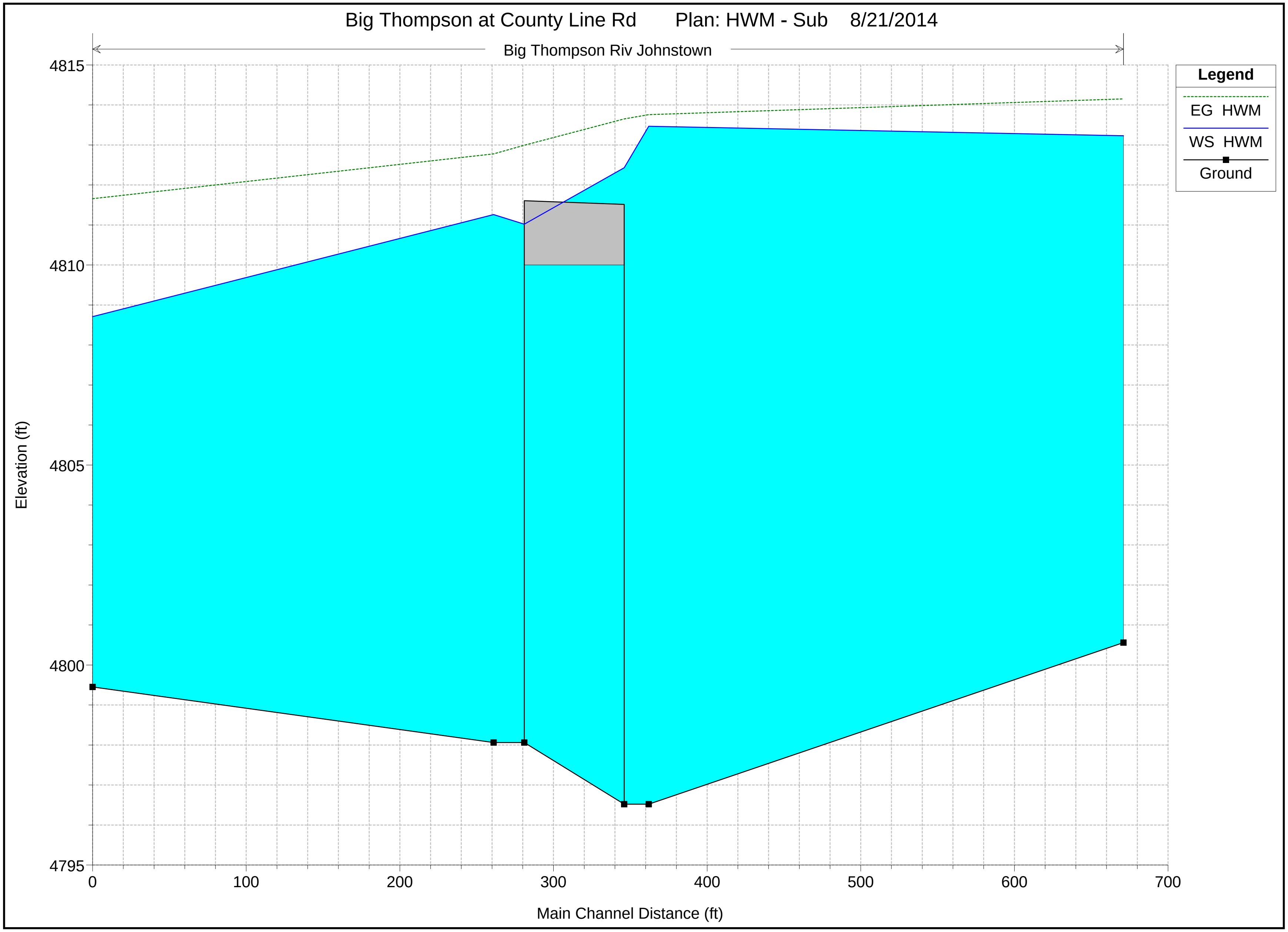


Big Thompson at County Line Road

HEC-RAS Plan: HWM Sub   River: Big Thompson Riv   Reach: Johnstown   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Johnstown	1171	HWM	8800.00	4800.56	4813.23	4810.19	4814.15	0.001898	8.71	1320.61	162.61	0.47
Johnstown	862	HWM	8800.00	4796.52	4813.47	4804.86	4813.76	0.000330	4.83	2474.43	238.52	0.21
Johnstown	846		Bridge									
Johnstown	761	HWM	8800.00	4798.06	4811.26	4806.87	4812.78	0.002140	9.91	895.04	158.55	0.50
Johnstown	500	HWM	8800.00	4799.45	4808.71	4808.71	4811.66	0.008132	14.63	714.97	178.75	0.93

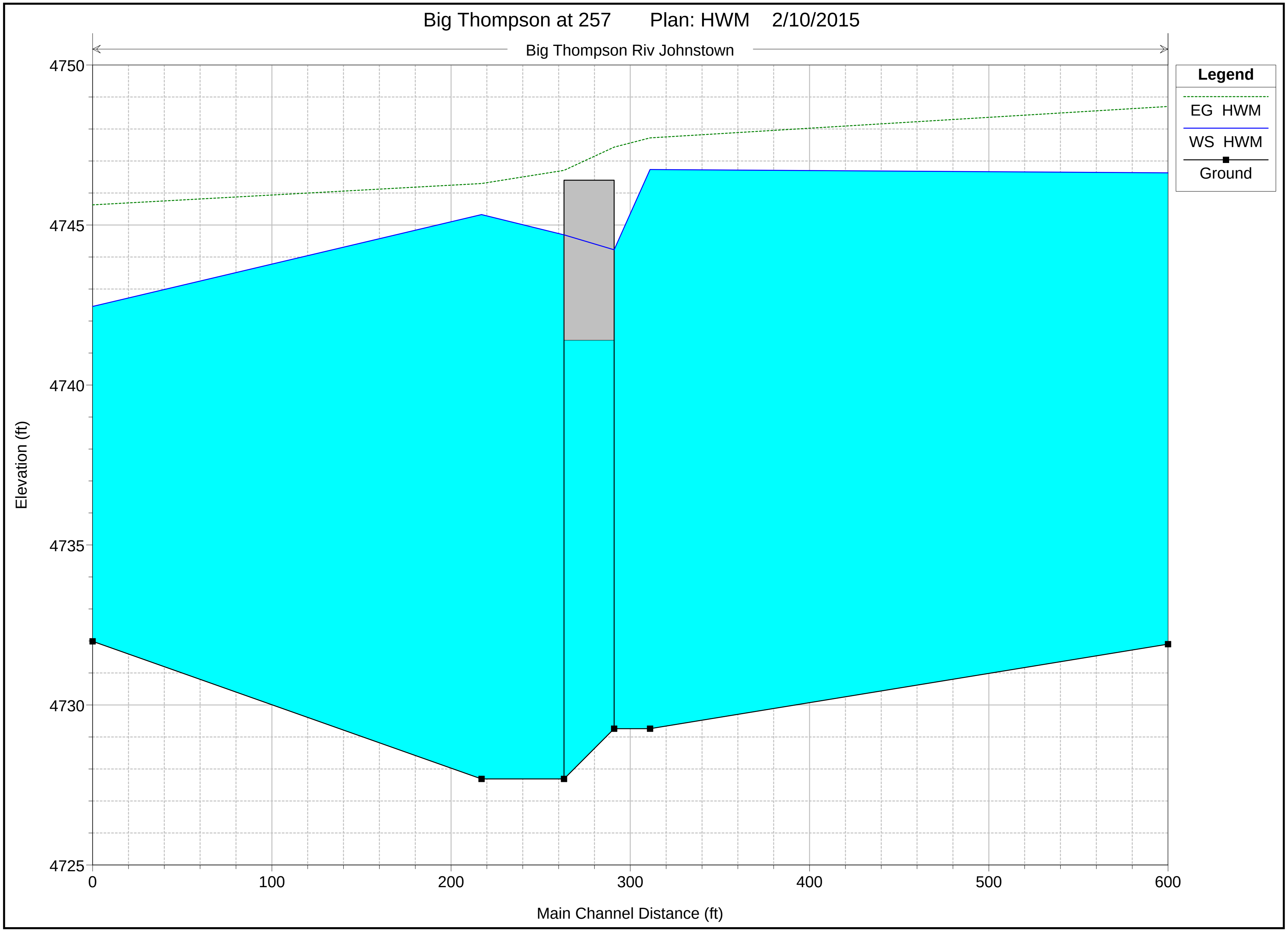
Big Thompson at County Line Road



Big Thompson at 257

HEC-RAS Plan: HWM Sub River: Big Thompson Riv Reach: Johnstown Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Johnstown	1600	HWM	17700.00	4731.90	4746.63	4744.57	4748.70	0.004722	14.95	1728.32	172.67	0.72
Johnstown	1311	HWM	17700.00	4729.26	4746.73	4740.73	4747.72	0.001324	8.02	2288.56	255.13	0.40
Johnstown	1291		Bridge									
Johnstown	1217	HWM	17700.00	4727.69	4745.32	4737.62	4746.30	0.000936	8.10	2367.79	262.00	0.35
Johnstown	1000	HWM	17700.00	4731.99	4742.45	4742.45	4745.63	0.007722	15.49	1403.46	230.45	0.93

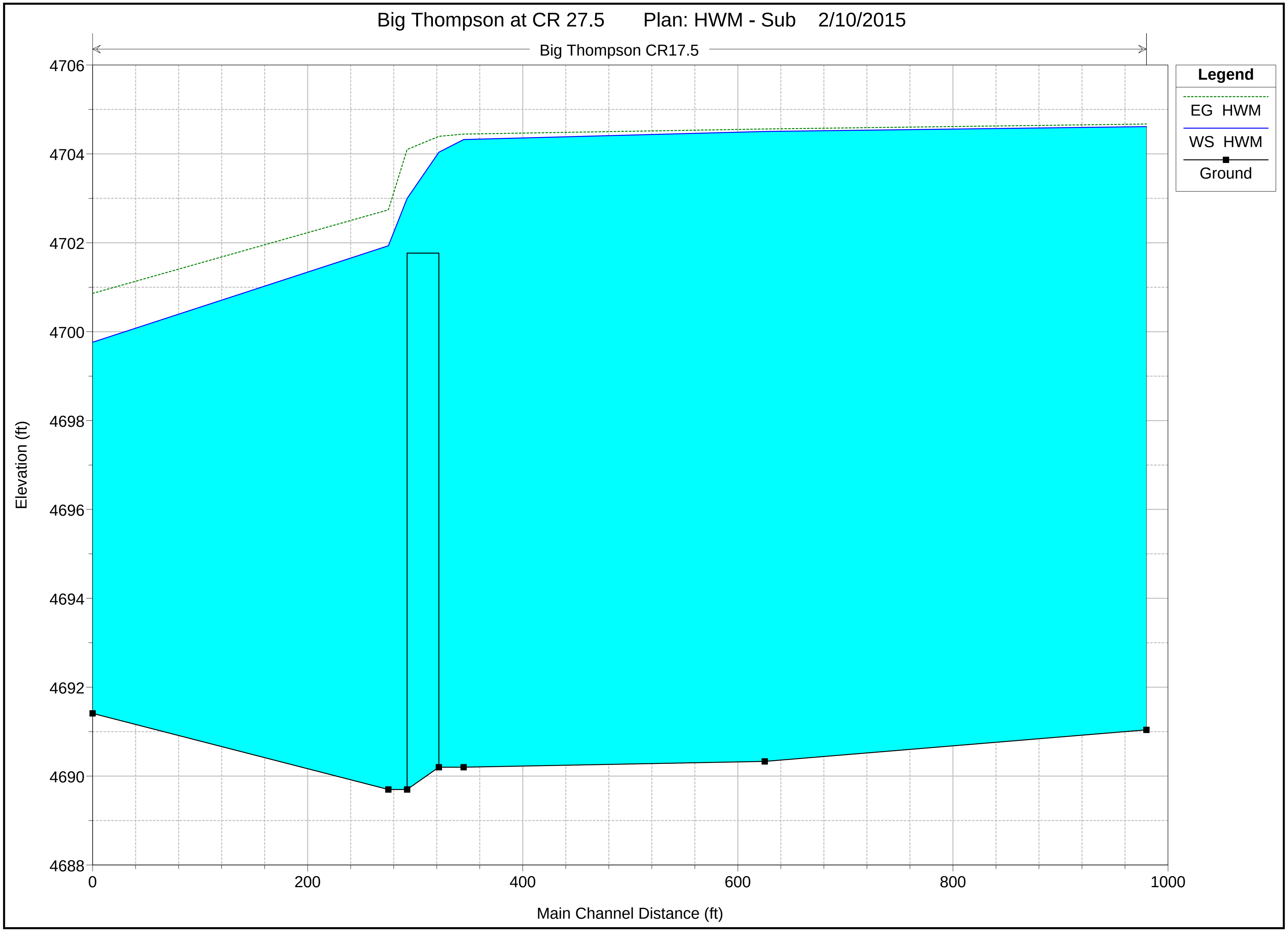

Big Thompson at 257 Plan: HWM 2/10/2015
Big Thompson Riv Johnstown
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4750
4745
4740
4735
4730
4725
0
100
200
300
400
500
600
Main Channel Distance (ft)

Big Thompson at CR 27.5

HEC-RAS Plan: HWM - Sub   River: Big Thompson   Reach: CR17.5   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
CR17.5	1480	HWM	24900.00	4691.04	4704.61	4702.12	4704.67	0.000360	3.39	15809.33	3885.42	0.20
CR17.5	1125	HWM	24900.00	4690.33	4704.50	4701.53	4704.56	0.000277	3.06	16517.65	3683.36	0.18
CR17.5	845	HWM	24900.00	4690.20	4704.32	4702.41	4704.45	0.000558	4.58	12727.37	3473.96	0.26
CR17.5	822		Bridge									
CR17.5	775	HWM	24900.00	4689.70	4701.93	4701.93	4702.74	0.005179	10.37	5764.35	3171.61	0.72
CR17.5	500	HWM	24900.00	4691.41	4699.76	4699.76	4700.86	0.009108	11.97	4175.16	2008.44	0.92

Big Thompson at CR 27.5



Technical Memorandum

Boulder Creek

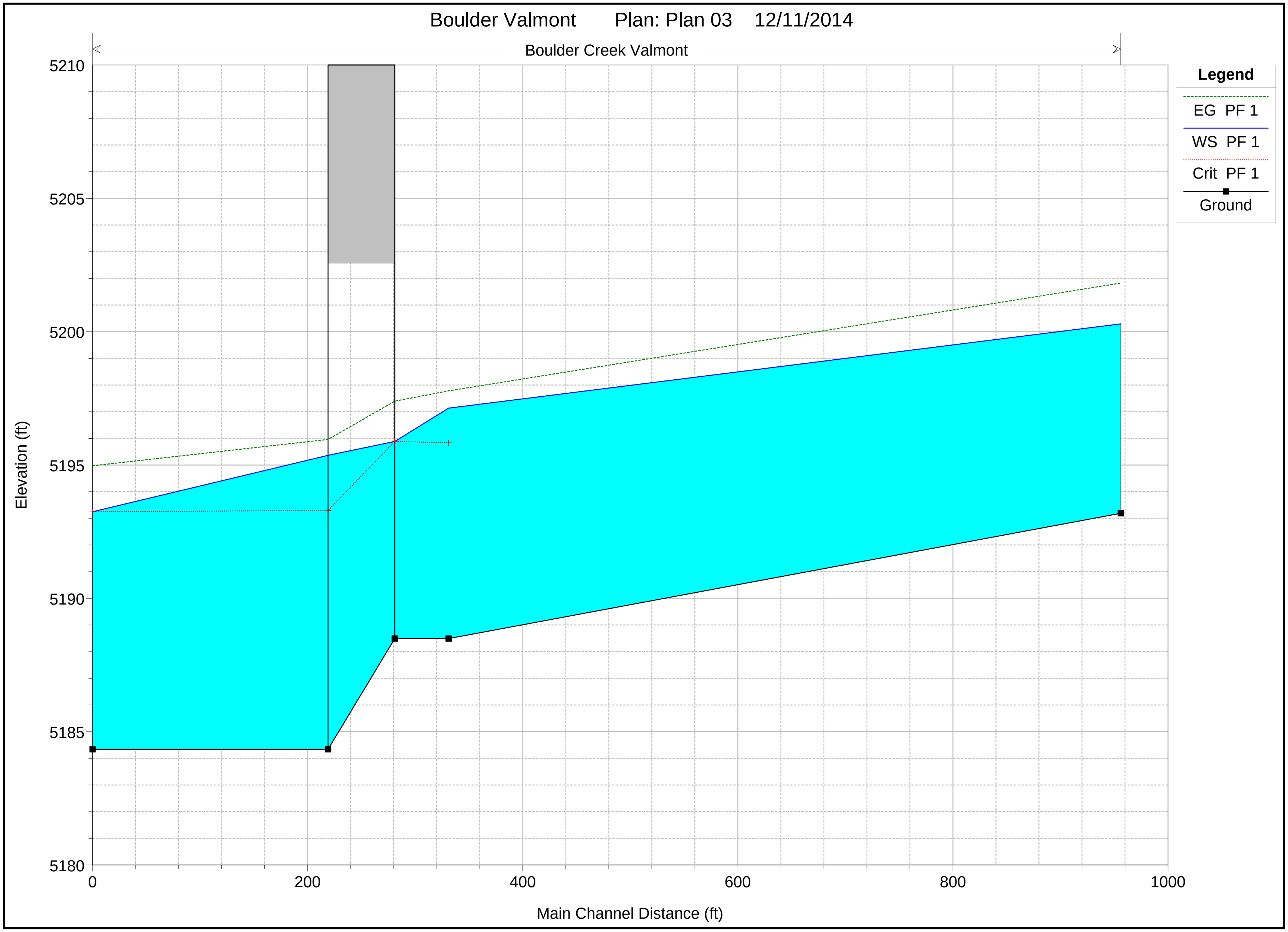
HEC-RAS Results

Boulder at Valmont

HEC-RAS Plan: Plan 03   River: Boulder Creek   Reach: Valmont   Profile: PF 1

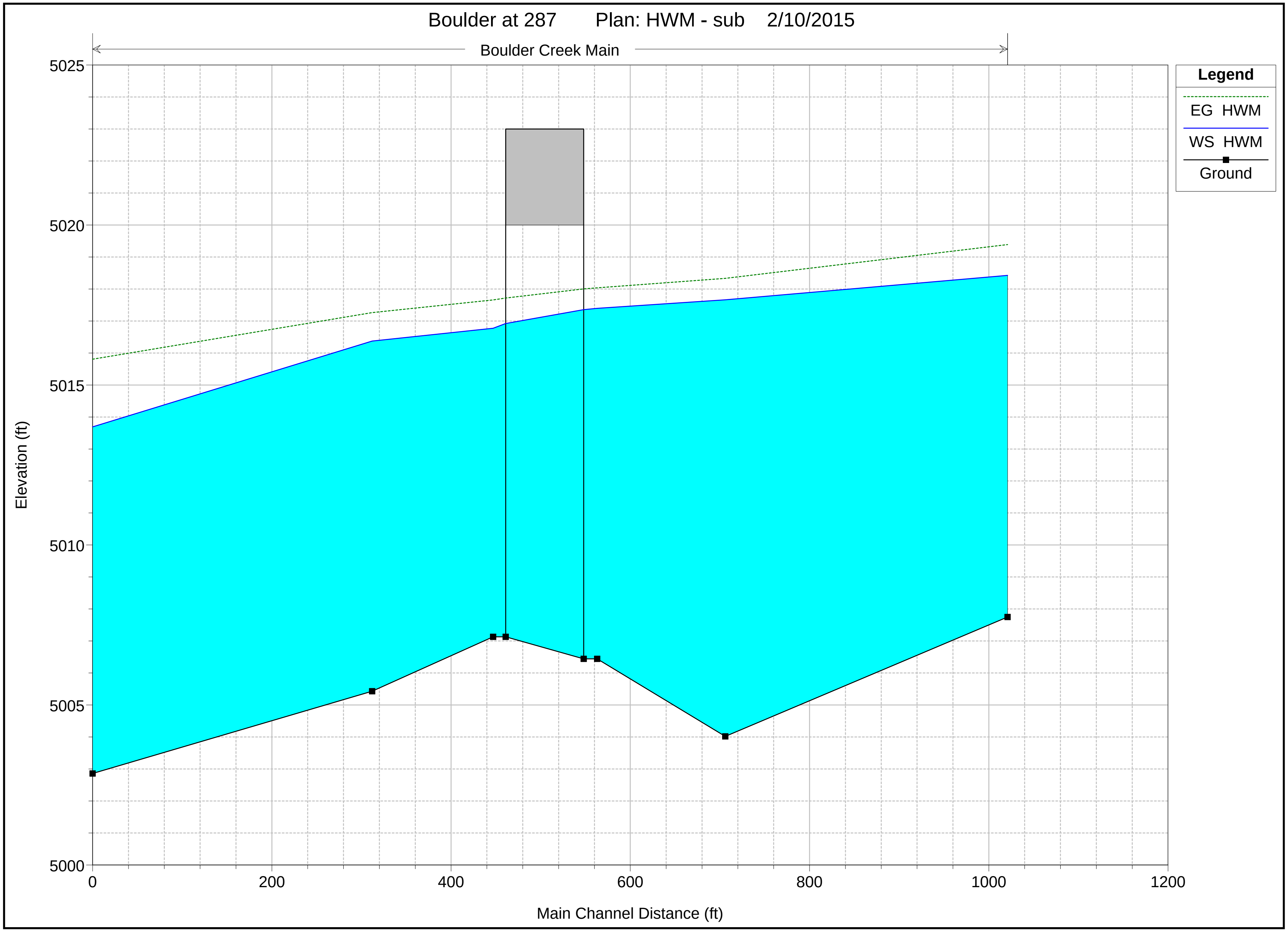
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Valmont	1956	PF 1	4300.00	5193.19	5200.29		5201.82	0.010375	10.55	460.25	109.11	0.82
Valmont	1331	PF 1	4300.00	5188.49	5197.13	5195.83	5197.78	0.003751	7.73	760.05	180.19	0.53
Valmont	1081		Bridge									
Valmont	1000	PF 1	4300.00	5184.34	5193.25	5193.25	5194.97	0.008304	11.80	513.68	155.69	0.77

Boulder at Valmont



HEC-RAS Plan: HWM - Sub River: Boulder Creek Reach: Main Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main	2021	HWM	9000.00	5007.75	5018.43	5018.10	5019.39	0.004807	10.13	1529.85	446.05	0.61
Main	1706	HWM	9000.00	5004.02	5017.66	5015.27	5018.33	0.002121	8.27	1687.08	282.04	0.43
Main	1563	HWM	9000.00	5006.44	5017.40	5014.00	5018.04	0.001876	7.35	1553.92	208.62	0.41
Main	1505		Bridge									
Main	1447	HWM	9000.00	5007.13	5016.77	5014.56	5017.66	0.003056	8.51	1324.66	208.55	0.52
Main	1312	HWM	9000.00	5005.43	5016.38	5014.19	5017.26	0.002832	8.22	1376.82	244.13	0.49
Main	1000	HWM	9000.00	5002.86	5013.69	5013.69	5015.81	0.007230	13.40	962.39	208.12	0.76



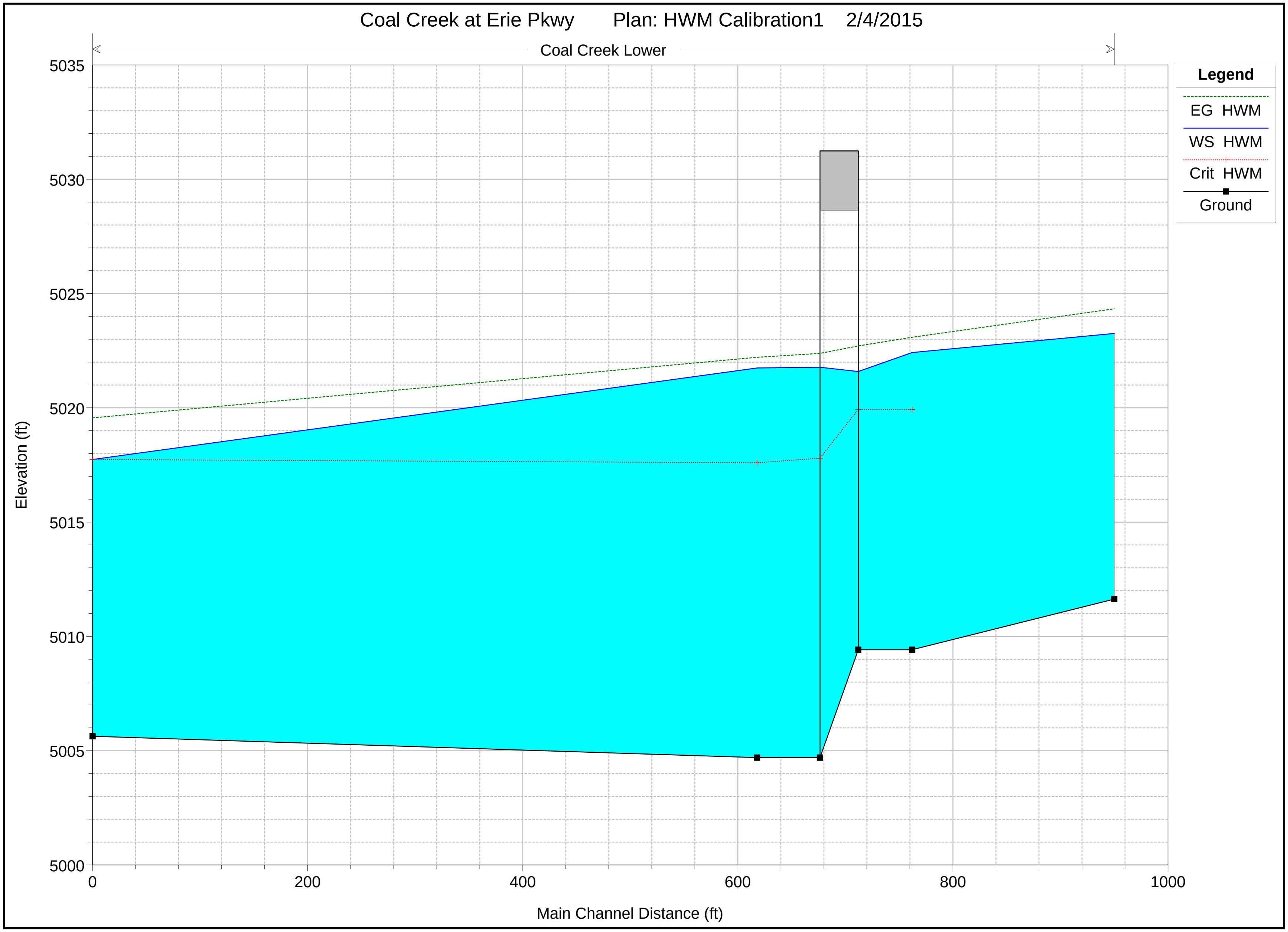
Boulder at 287 Plan: HWM - sub 2/10/2015
Boulder Creek Main
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
5025
5020
5015
5010
5005
5000
0
200
400
600
800
1000
1200
Main Channel Distance (ft)

Coal at Erie Pkwy

HEC-RAS Plan: HWM 1 Bridge   River: Coal Creek   Reach: Lower   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	19370	HWM	6000.00	5011.63	5023.26		5024.33	0.008024	9.22	740.79	143.95	0.51
Lower	19182	HWM	6000.00	5009.42	5022.42	5019.92	5023.09	0.004605	6.72	918.56	176.19	0.36
Lower	19154		Bridge									
Lower	19038	HWM	6000.00	5004.70	5021.75	5017.60	5022.21	0.001853	5.55	1155.46	204.33	0.27
Lower	18420	HWM	6000.00	5005.63	5017.74	5017.74	5019.56	0.014899	12.05	588.19	154.27	0.74

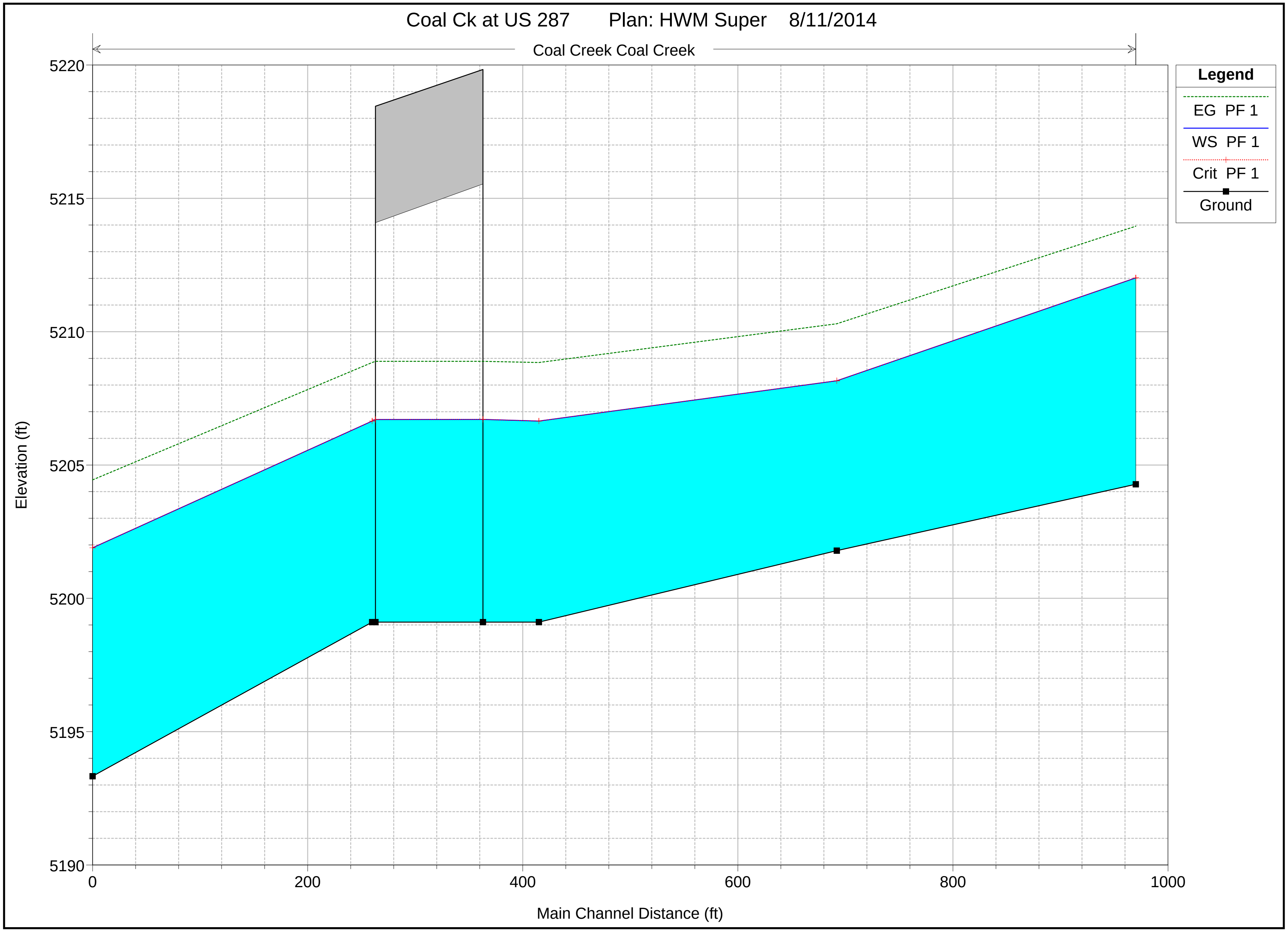
Coal at Erie Pkwy



Coal at US 287

HEC-RAS Plan: Super Crit   River: Coal Creek   Reach: Coal Creek   Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Coal Creek	82873.73	PF 1	5000.00	5204.28	5212.02	5212.02	5213.96	0.060211	12.09	458.53	114.90	0.92
Coal Creek	82595.73	PF 1	5000.00	5201.79	5208.16	5208.16	5210.30	0.067647	12.20	447.71	145.18	0.97
Coal Creek	82318.73	PF 1	5000.00	5199.11	5206.65	5206.65	5208.84	0.061620	12.75	430.27	97.03	0.94
Coal Creek	82260		Bridge									
Coal Creek	82163.73	PF 1	5000.00	5199.11	5206.65	5206.65	5208.84	0.061620	12.75	430.27	97.03	0.94
Coal Creek	81903.73	PF 1	5000.00	5193.33	5201.89	5201.89	5204.44	0.052203	13.60	411.22	79.80	0.90



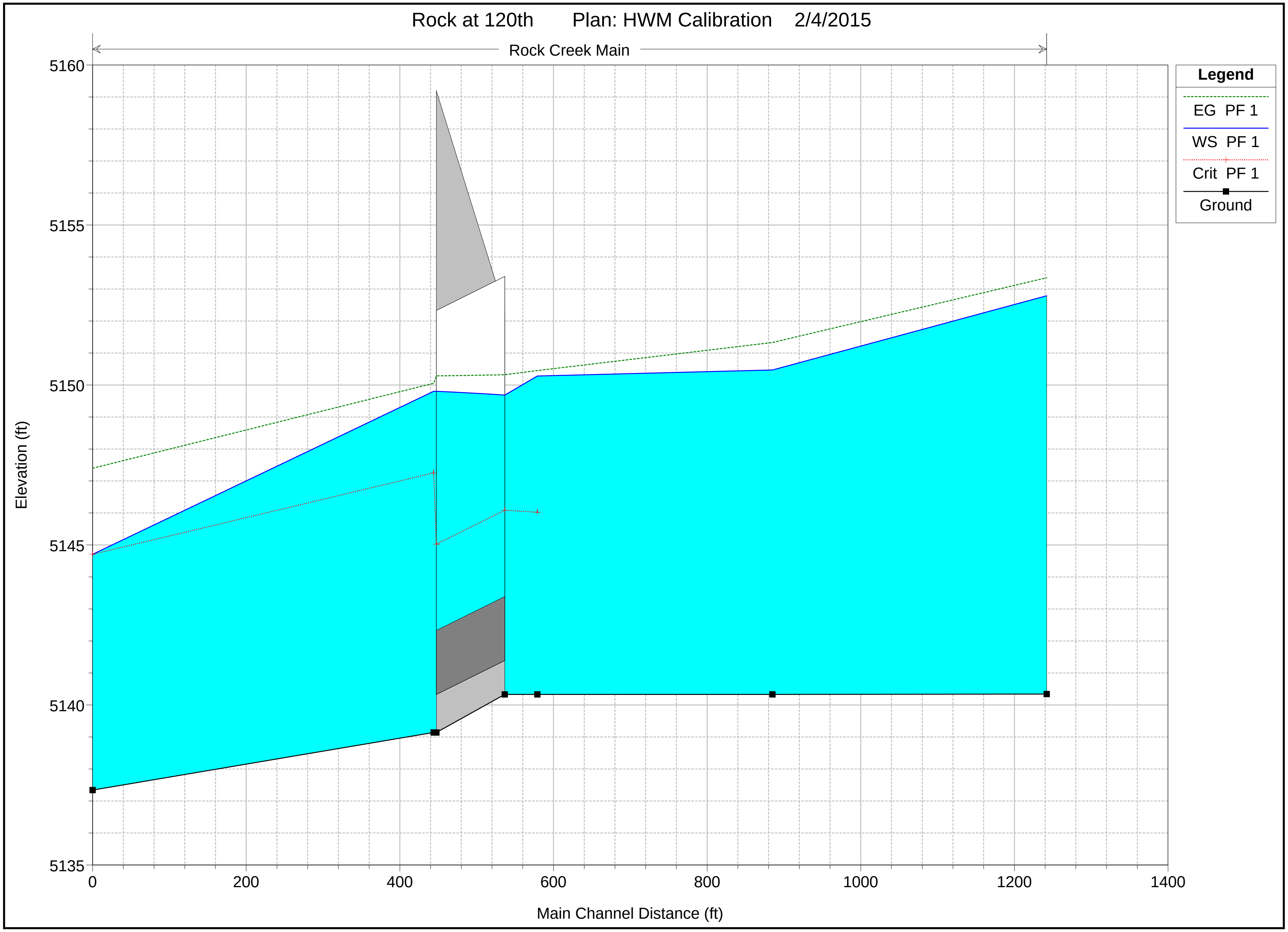
Coal Ck at US 287 Plan: HWM Super 8/11/2014
Coal Creek Coal Creek
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
5220
5215
5210
5205
5200
5195
5190
0
200
400
600
800
1000
Main Channel Distance (ft)

Rock at 120th

HEC-RAS Plan: HWM Calibration   River: Rock Creek   Reach: Main   Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main	207844	PF 1	1500.00	5140.34	5152.79		5153.35	0.004839	6.07	258.07	58.77	0.44
Main	207487	PF 1	1500.00	5140.33	5150.47		5151.33	0.006494	7.92	236.32	76.93	0.51
Main	207181	PF 1	1500.00	5140.33	5150.28	5146.02	5150.45	0.001093	3.82	485.95	83.66	0.23
Main	207119		Culvert									
Main	207046	PF 1	1500.00	5139.14	5149.80	5147.26	5150.05	0.002091	5.00	419.99	106.00	0.30
Main	206602	PF 1	1500.00	5137.34	5144.71	5144.71	5147.40	0.035533	13.48	119.51	35.59	1.07

Rock at 120th

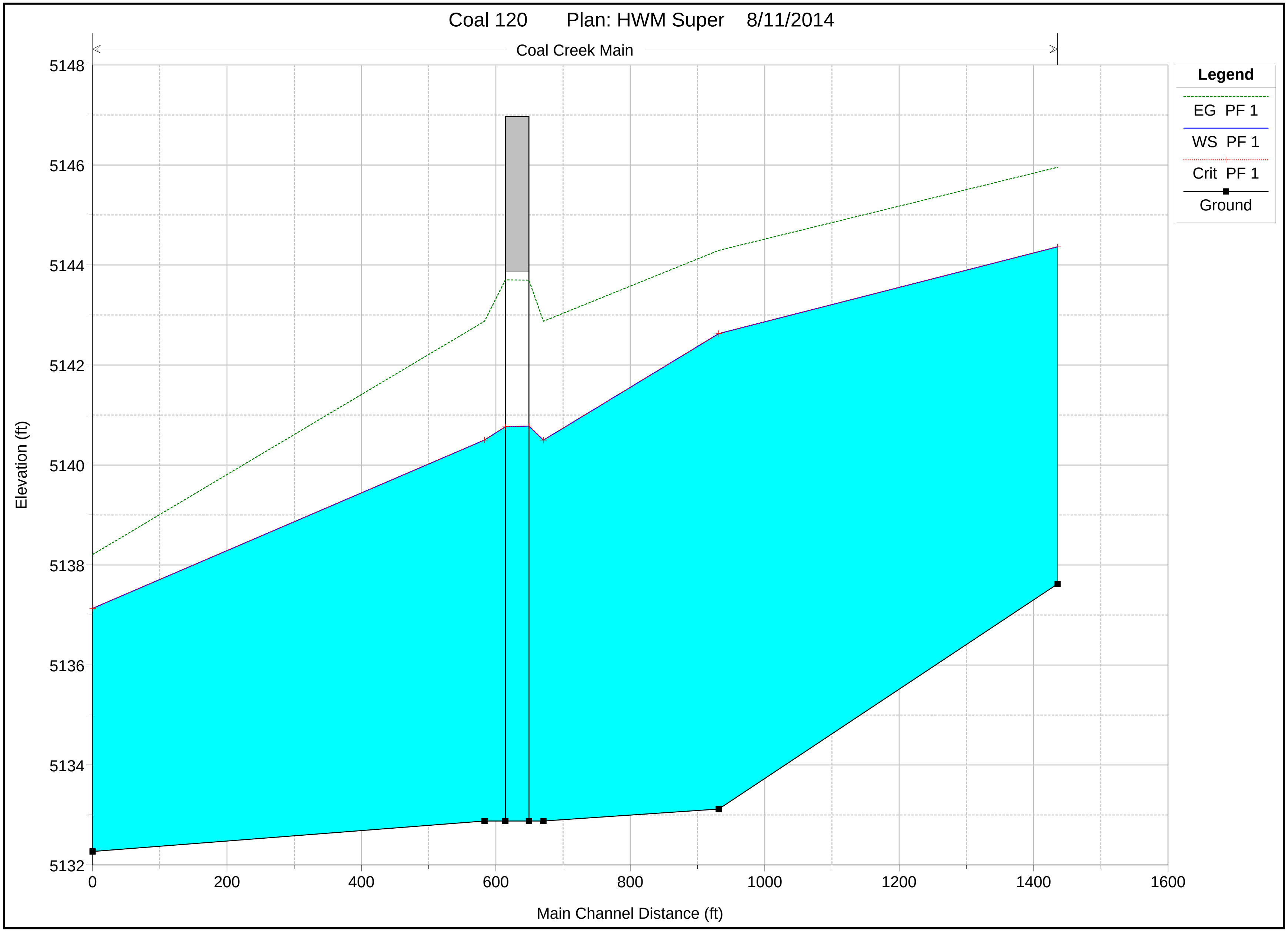


Coal at 120

HEC-RAS Plan: Super Crit   River: Coal Creek   Reach: Main   Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main	72657.6	PF 1	3500.00	5137.62	5144.36	5144.36	5145.96	0.020221	12.87	378.53	114.48	0.93
Main	72153.6	PF 1	3500.00	5133.12	5142.63	5142.63	5144.29	0.015926	13.03	397.17	112.42	0.81
Main	71892.6	PF 1	3500.00	5132.88	5140.50	5140.50	5142.87	0.019779	14.06	302.32	74.90	0.97
Main	71836		Bridge									
Main	71805	PF 1	3500.00	5132.88	5140.50	5140.50	5142.87	0.019727	14.05	302.62	74.97	0.97
Main	71222	PF 1	3500.00	5132.27	5137.13	5137.13	5138.21	0.016879	9.14	481.07	219.26	0.84

Coal at 120



Coal 120 Plan: HWM Super 8/11/2014
Coal Creek Main
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
5148
5146
5144
5142
5140
5138
5136
5134
5132
0
200
400
600
800
1000
1200
1400
1600
Main Channel Distance (ft)

URS

Technical Memorandum

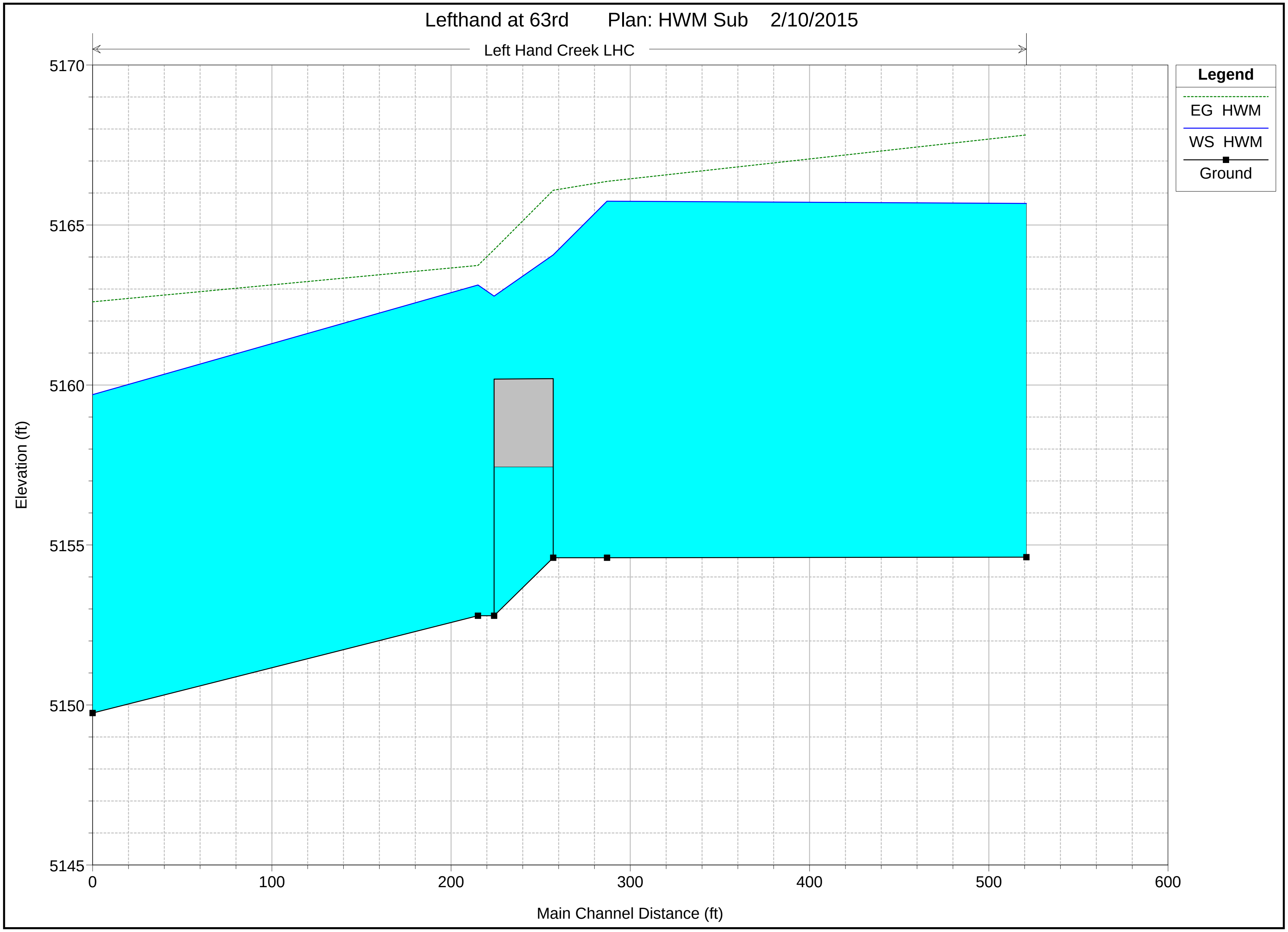
Lefthand Creek

HEC-RAS Results

Left Hand at 63rd

HEC-RAS Plan: HWM Sub River: Left Hand Creek Reach: LHC Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
LHC	1521	HWM	7000.00	5154.62	5165.67	5165.67	5167.81	0.008699	13.88	771.10	163.02	0.83
LHC	1287	HWM	7000.00	5154.60	5165.74	5161.55	5166.36	0.001709	6.44	1203.36	197.39	0.38
LHC	1257		Bridge									
LHC	1215	HWM	7000.00	5152.79	5163.13	5160.56	5163.74	0.002252	6.78	1290.28	246.23	0.43
LHC	1000	HWM	7000.00	5149.75	5159.70	5159.70	5162.60	0.010550	15.47	607.18	113.06	0.93

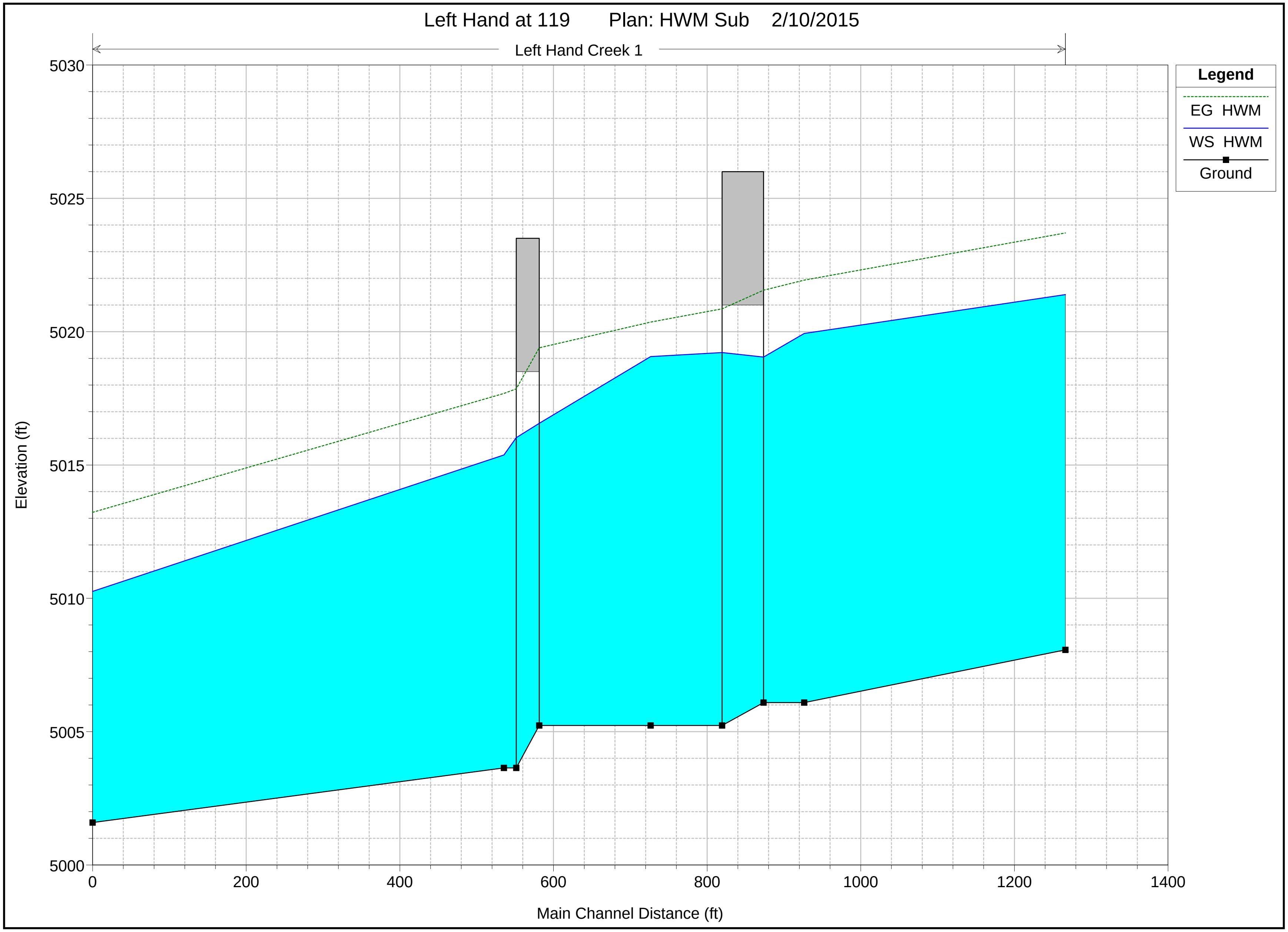

Lefthand at 63rd Plan: HWM Sub 2/10/2015
Left Hand Creek LHC
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
5170
5165
5160
5155
5150
5145
0
100
200
300
400
500
600
Main Channel Distance (ft)

Left Hand at 119th

HEC-RAS Plan: HWM Sub   River: Left Hand Creek   Reach: 1   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	1731	HWM	8700.00	5008.07	5021.39	5020.04	5023.70	0.006282	12.81	777.98	157.33	0.71
1	1391	HWM	8700.00	5006.09	5019.93	5017.19	5021.93	0.003965	11.74	858.66	131.02	0.60
1	1338		Bridge									
1	1191	HWM	8700.00	5005.23	5019.07	5016.48	5020.36	0.003057	10.27	1124.41	183.64	0.52
1	1046		Bridge									
1	1000	HWM	8700.00	5003.64	5015.38	5014.38	5017.68	0.006716	12.97	830.02	134.17	0.75
1	464.5513	HWM	8700.00	5001.59	5010.26	5010.26	5013.22	0.010261	14.17	687.36	247.13	0.92

Left Hand at 119th

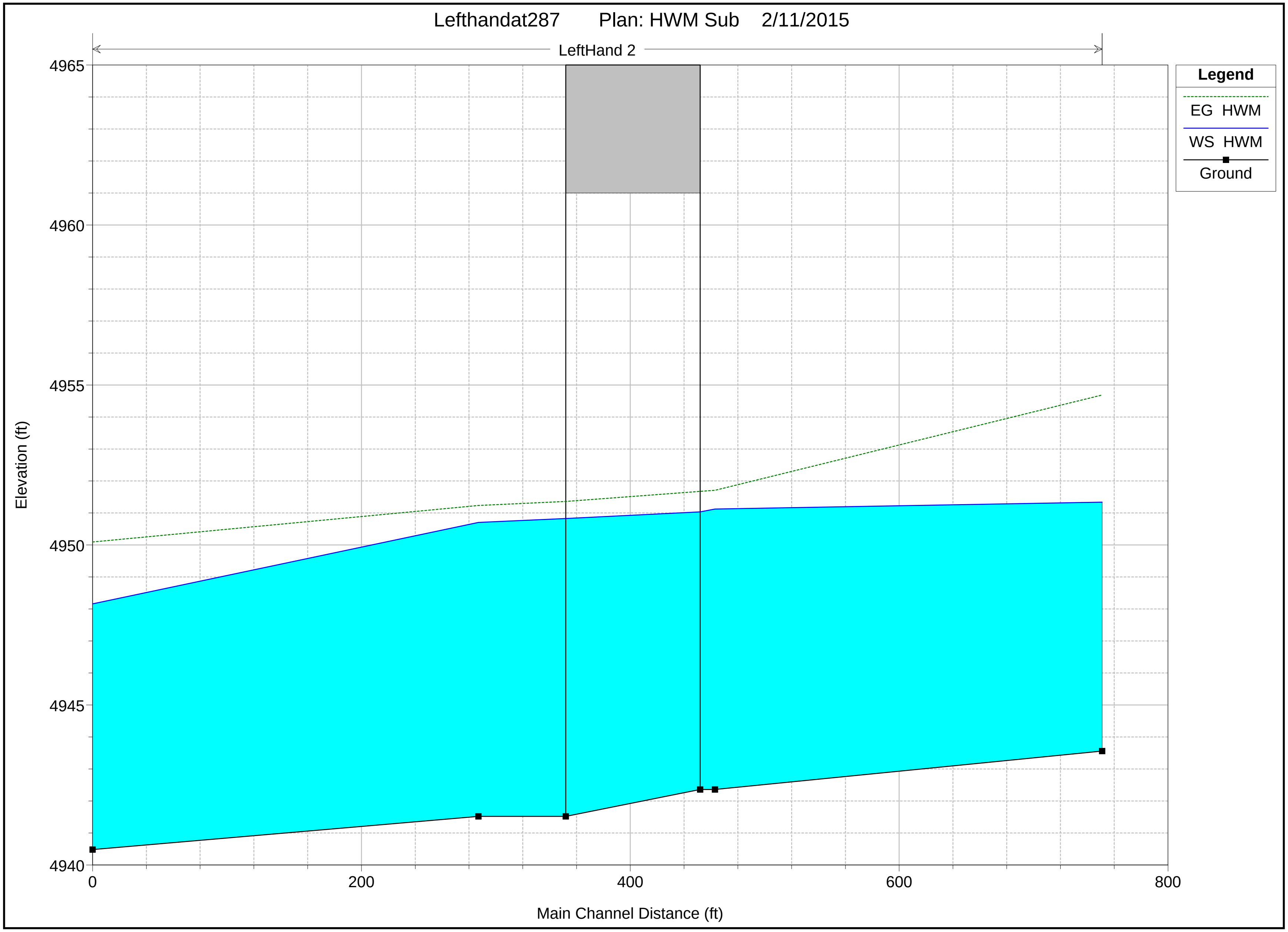


Left Hand at 287

HEC-RAS Plan: HWM Sub   River: LeftHand   Reach: 2   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
2	1751	HWM	5000.00	4943.56	4951.34	4951.34	4954.69	0.012627	14.68	340.50	70.85	0.99
2	1463	HWM	5000.00	4942.36	4951.12	4947.62	4951.71	0.001884	6.27	852.72	119.08	0.39
2	1452		Bridge									
2	1287	HWM	5000.00	4941.52	4950.70	4946.88	4951.24	0.001545	6.10	921.21	136.30	0.36
2	1000	HWM	5000.00	4940.48	4948.16	4948.16	4950.09	0.014139	12.55	499.97	152.36	0.99

Left Hand at 287

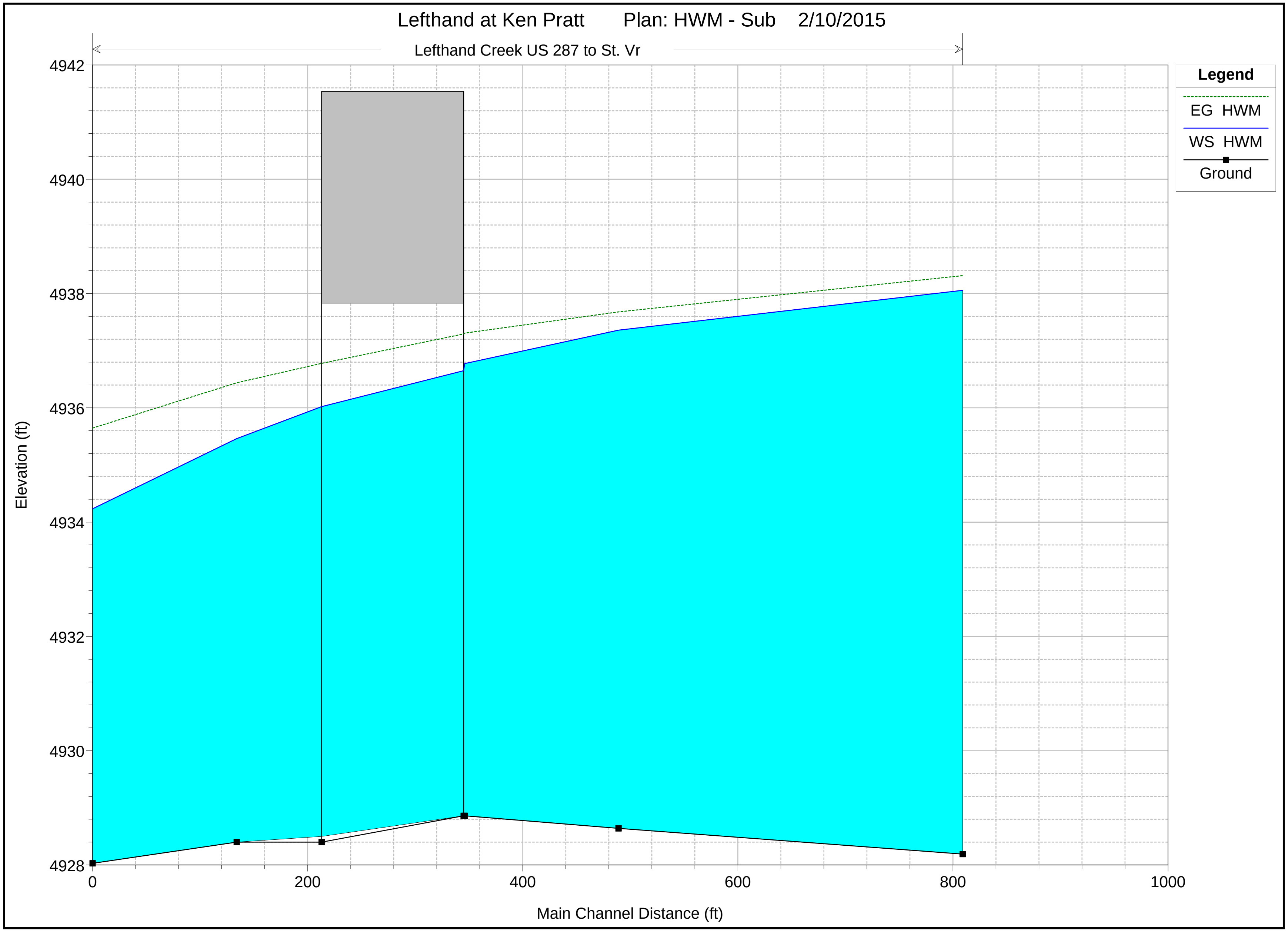


Left Hand at Ken Pratt

HEC-RAS Plan: HWM Sub River: Lefthand Creek Reach: US 287 to St. Vr Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
US 287 to St. Vr	1609	HWM	4800.00	4928.19	4938.06	4934.73	4938.31	0.001930	6.08	1304.98	232.20	0.38
US 287 to St. Vr	1289	HWM	4800.00	4928.64	4937.36	4933.80	4937.68	0.002011	6.43	1229.18	222.88	0.41
US 287 to St. Vr	1146	HWM	4800.00	4928.86	4936.77	4934.25	4937.31	0.003027	7.88	945.39	165.40	0.50
US 287 to St. Vr	1067		Bridge									
US 287 to St. Vr	934	HWM	4800.00	4928.40	4935.46	4934.11	4936.44	0.004171	9.00	724.98	147.51	0.65
US 287 to St. Vr	800	HWM	4800.00	4928.03	4934.23	4934.23	4935.65	0.007855	10.53	649.98	230.27	0.85

Left Hand at Ken Pratt



Technical Memorandum

St. Vrain Creek

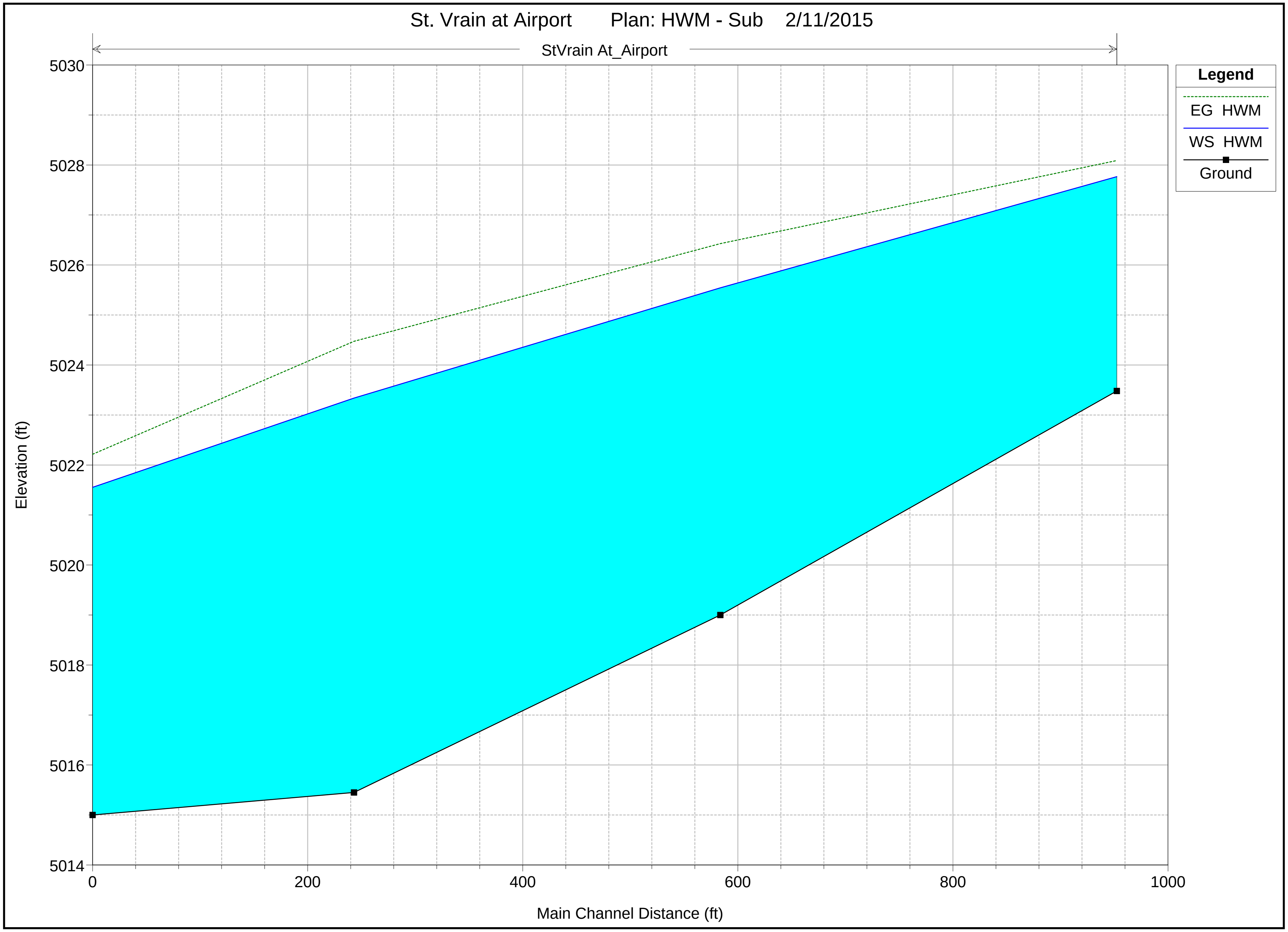
HEC-RAS Results

St. Vrain at Airport

HEC-RAS Plan: HWM Sub   River: StVrain   Reach: At_Airport   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
At_Airport	1486.316	HWM	14000.00	5023.48	5027.77	5026.83	5028.09	0.003893	6.35	4226.68	2147.94	0.58
At_Airport	1117.65	HWM	14000.00	5019.00	5025.54	5025.54	5026.43	0.004906	8.92	2986.14	1672.45	0.69
At_Airport	777.0211	HWM	14000.00	5015.45	5023.34	5023.34	5024.47	0.005145	9.85	2410.87	1751.79	0.72
At_Airport	533.9273	HWM	14000.00	5015.00	5021.55	5021.55	5022.22	0.004387	8.36	3757.45	2709.84	0.65

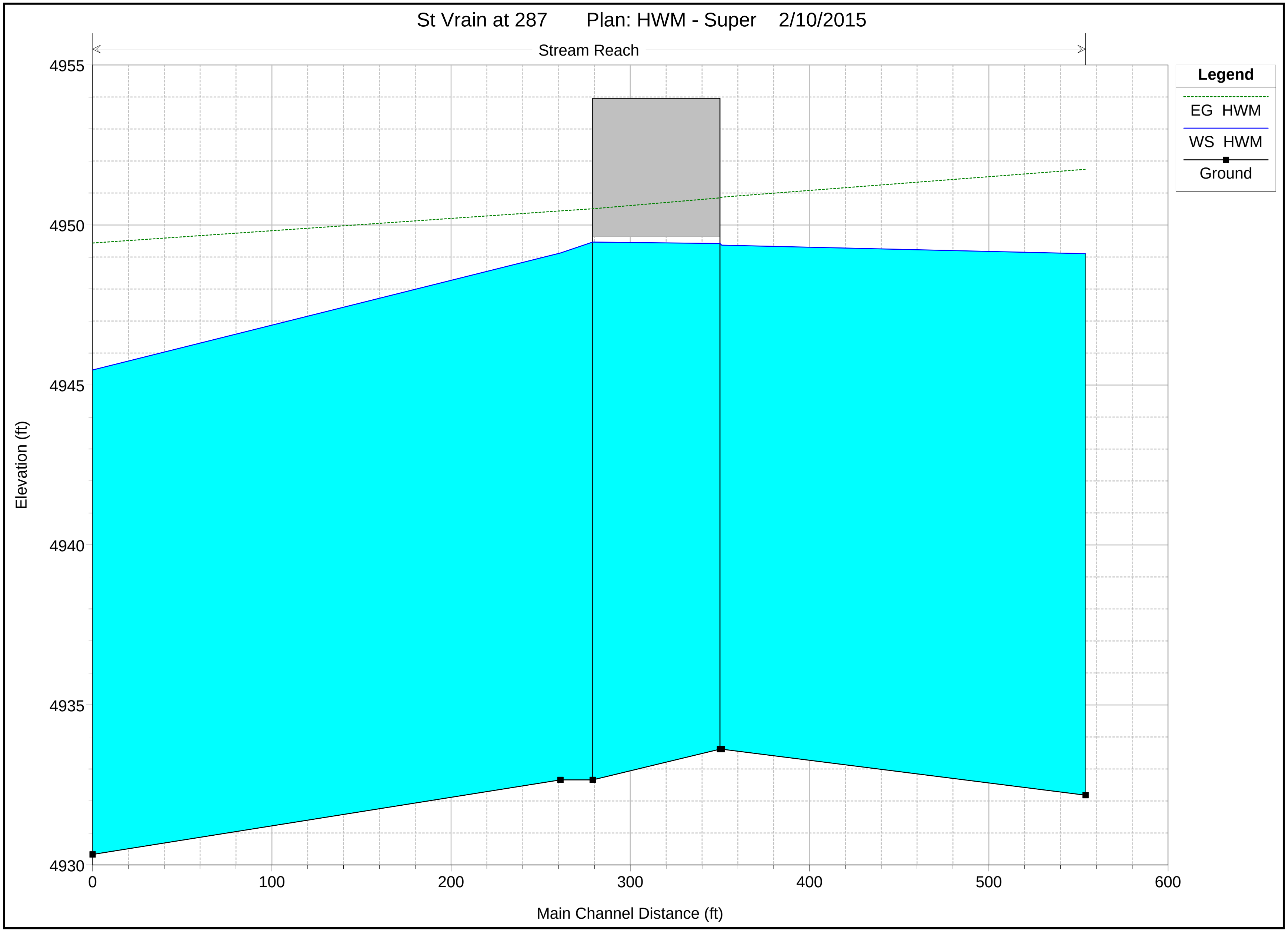
St. Vrain at Airport



St. Vrain at 287

HEC-RAS Plan: HWM - Super   River: Stream   Reach: Reach   Profile: HWM

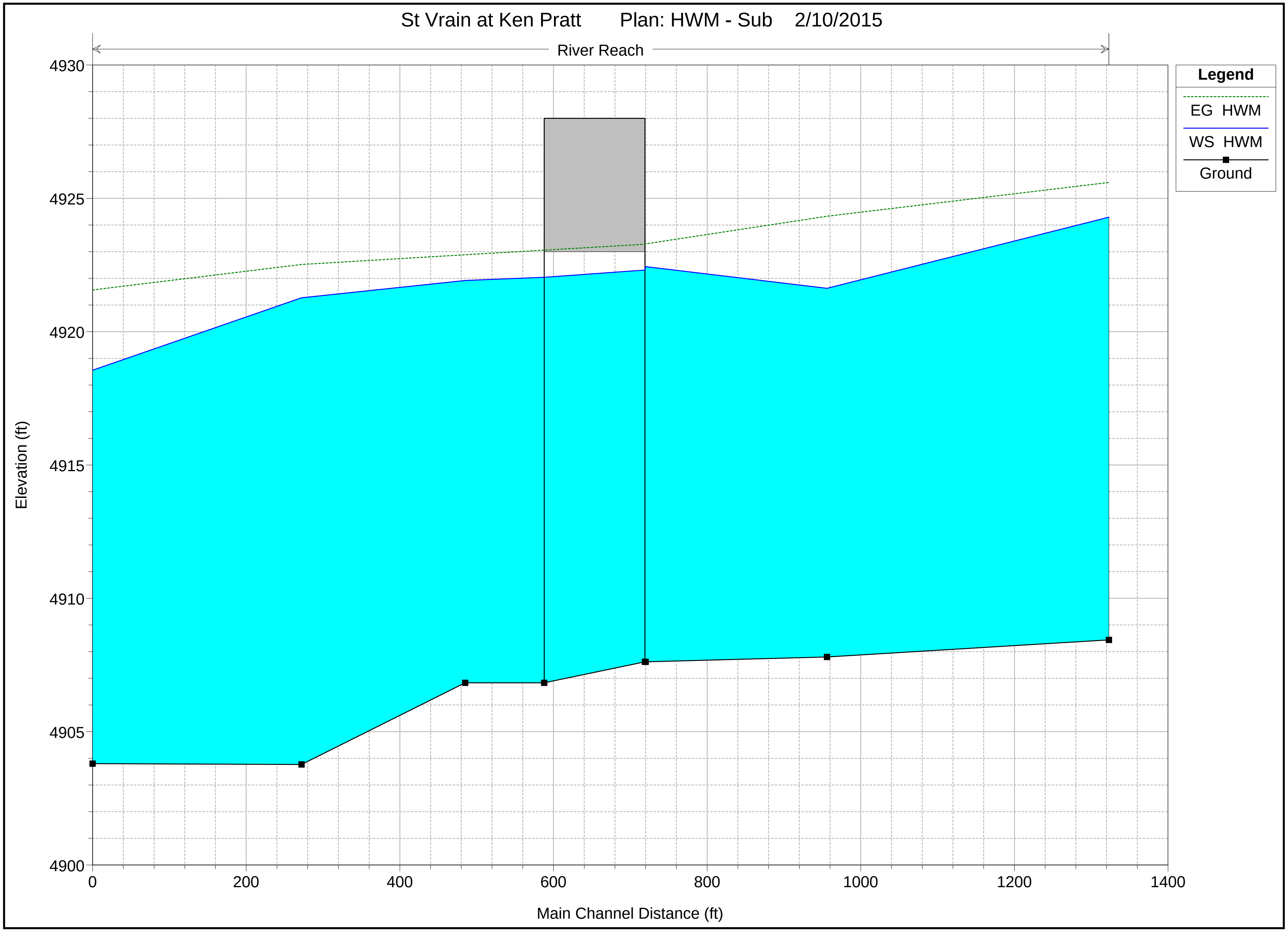
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	654	HWM	14500.00	4932.18	4949.10	4946.57	4951.74	0.003888	15.41	1423.46	148.47	0.71
Reach	451	HWM	14500.00	4933.62	4949.37	4944.80	4950.87	0.001850	10.87	1743.20	159.65	0.50
Reach	450		Bridge									
Reach	361	HWM	14500.00	4932.66	4949.13	4944.46	4950.44	0.001685	10.52	1846.43	181.47	0.48
Reach	100	HWM	14500.00	4930.33	4945.47	4945.47	4949.44	0.005772	17.65	1168.03	169.34	0.84


St Vrain at 287 Plan: HWM - Super 2/10/2015
Stream Reach
Legend
EG HWM
WS HWM
Ground
4955
4950
4945
4940
4935
4930
Elevation (ft)
0
100
200
300
400
500
600
Main Channel Distance (ft)

St. Vrain at Ken Pratt

HEC-RAS Plan: HWM - Sub   River: River   Reach: Reach   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	1378	HWM	18500.00	4908.44	4924.29	4920.97	4925.59	0.002364	11.62	2608.53	263.71	0.54
Reach	1011	HWM	18500.00	4907.80	4921.63	4920.82	4924.33	0.004165	14.69	1821.17	239.69	0.72
Reach	775	HWM	18500.00	4907.62	4922.44	4917.13	4923.29	0.001192	7.67	2657.07	290.11	0.38
Reach	774		Bridge									
Reach	540	HWM	18500.00	4906.83	4921.92	4916.94	4922.89	0.001357	8.07	2494.23	282.47	0.41
Reach	327	HWM	18500.00	4903.77	4921.27	4917.26	4922.52	0.001858	10.54	2460.61	300.95	0.49
Reach	55	HWM	18500.00	4903.80	4918.55	4918.55	4921.56	0.005052	16.26	1844.45	278.21	0.79

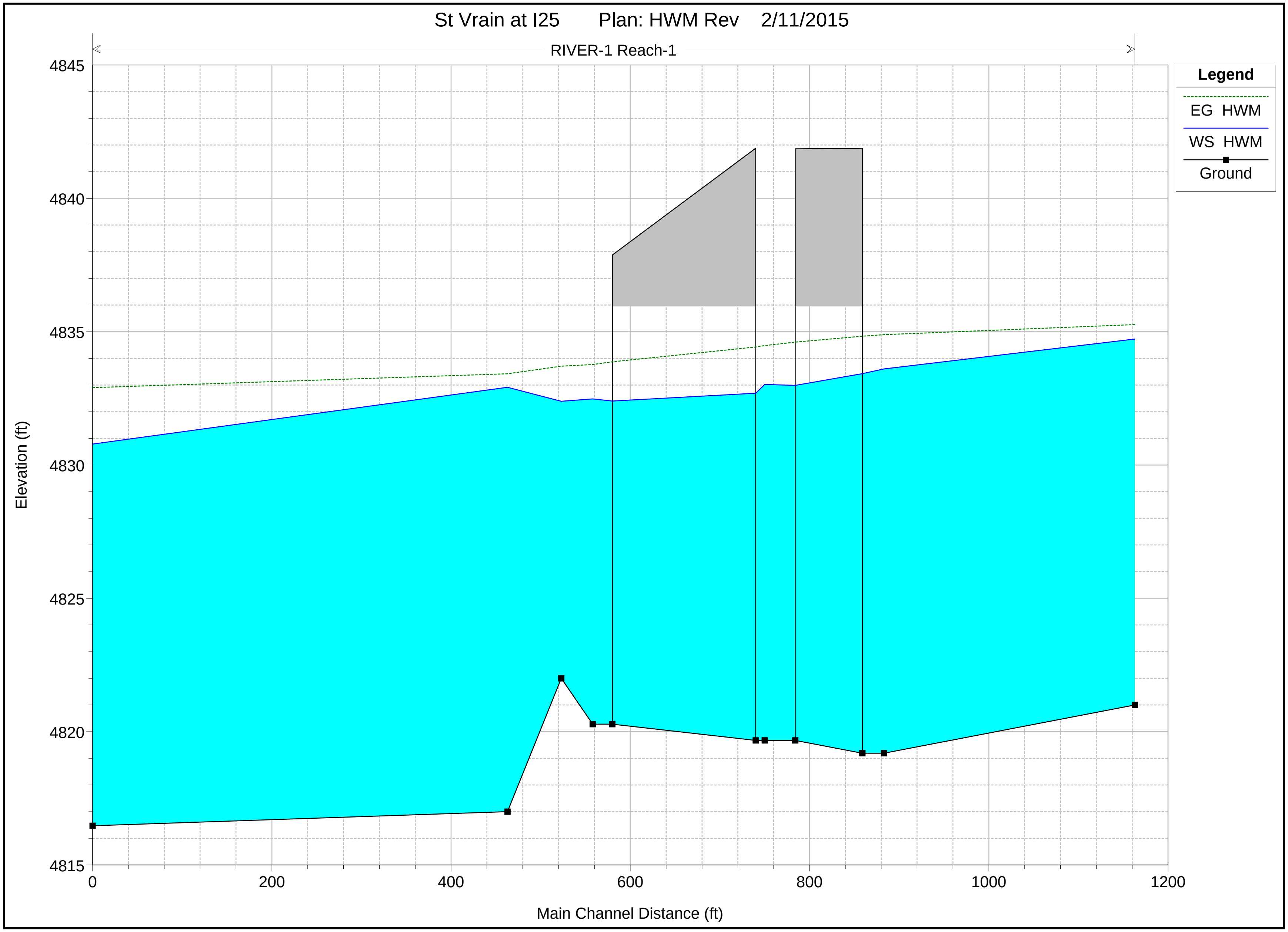

St Vrain at Ken Pratt Plan: HWM - Sub 2/10/2015
River Reach
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4930
4925
4920
4915
4910
4905
4900
0
200
400
600
800
1000
1200
1400
Main Channel Distance (ft)

St. Vrain at I-25

HEC-RAS Plan: HWM Rev River: RIVER-1 Reach: Reach-1 Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach-1	1766.598	HWM	23500.00	4821.00	4834.73	4829.05	4835.27	0.000851	6.74	4275.77	441.00	0.37
Reach-1	1486.598	HWM	23500.00	4819.19	4833.60	4829.64	4834.89	0.001444	9.37	2757.65	283.90	0.49
Reach-1	1462.498		Bridge									
Reach-1	1353.598	HWM	23500.00	4819.67	4833.02	4829.96	4834.48	0.001880	9.99	2571.49	289.39	0.55
Reach-1	1343.498		Bridge									
Reach-1	1161.598	HWM	23500.00	4820.28	4832.48	4829.18	4833.77	0.001689	9.27	2664.22	322.91	0.52
Reach-1	1126.598	HWM	23500.00	4822.00	4832.39	4829.14	4833.71	0.001722	9.36	2637.57	322.12	0.53
Reach-1	1066.598	HWM	23500.00	4817.00	4832.91	4824.14	4833.42	0.000372	5.86	4306.12	384.59	0.26
Reach-1	603.598	HWM	23500.00	4816.47	4830.78	4828.32	4832.90	0.002503	12.52	2418.26	357.78	0.65

St. Vrain at I-25

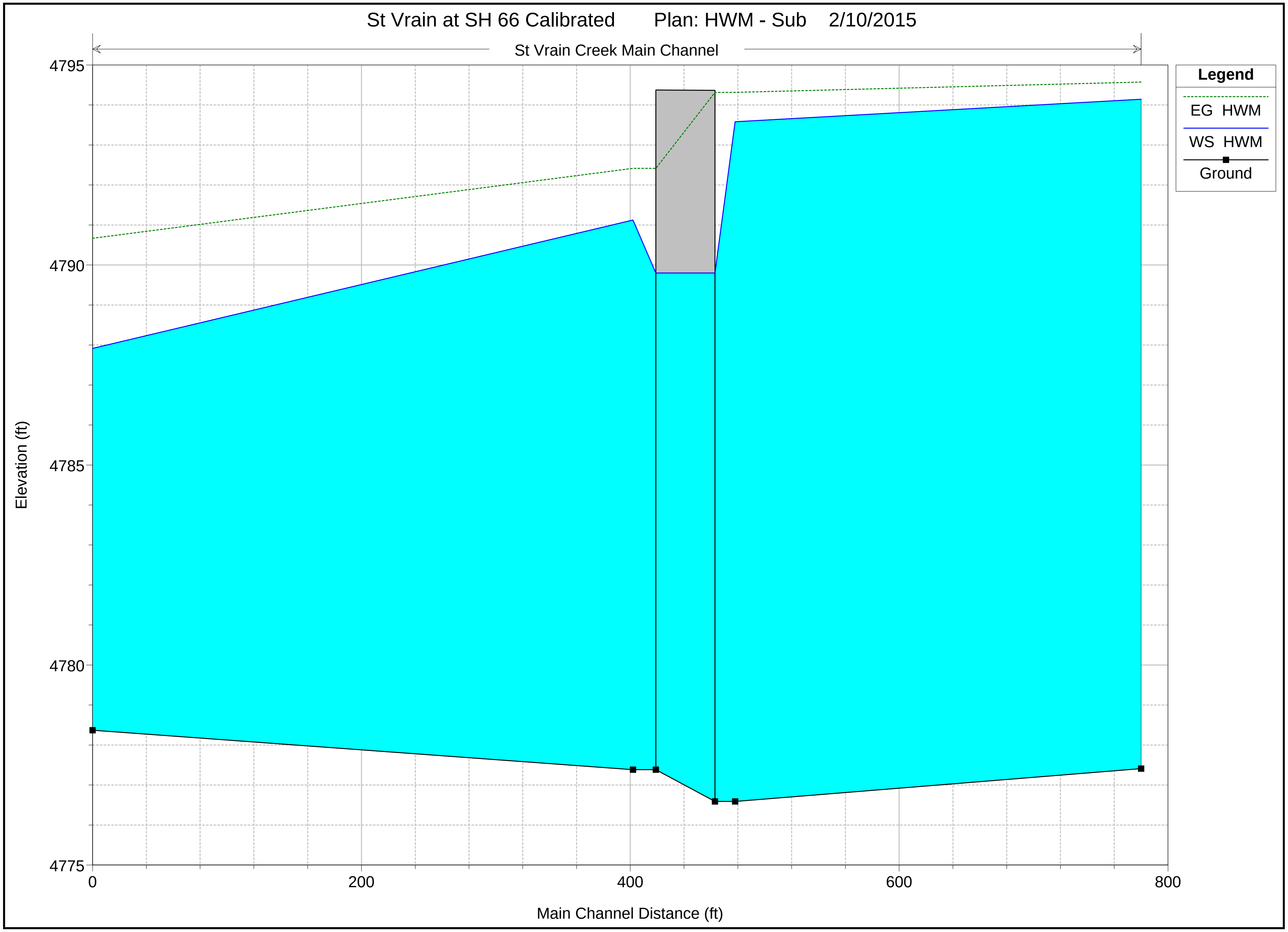


St. Vrain at SH 66

HEC-RAS Plan: HWM - Sub River: St Vrain Creek Reach: Main Channel Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Channel	1780	HWM	23000.00	4777.41	4794.14	4786.83	4794.57	0.000586	5.82	4790.88	400.71	0.26
Main Channel	1478	HWM	23000.00	4776.59	4793.58	4786.94	4794.32	0.001041	6.90	3353.10	299.65	0.34
Main Channel	1463		Bridge									
Main Channel	1402	HWM	23000.00	4777.38	4791.13	4787.60	4792.42	0.002598	9.40	2570.84	276.69	0.52
Main Channel	1000	HWM	23000.00	4778.37	4787.92	4787.92	4790.67	0.008608	13.97	1882.23	333.27	0.89

St. Vrain at SH 66

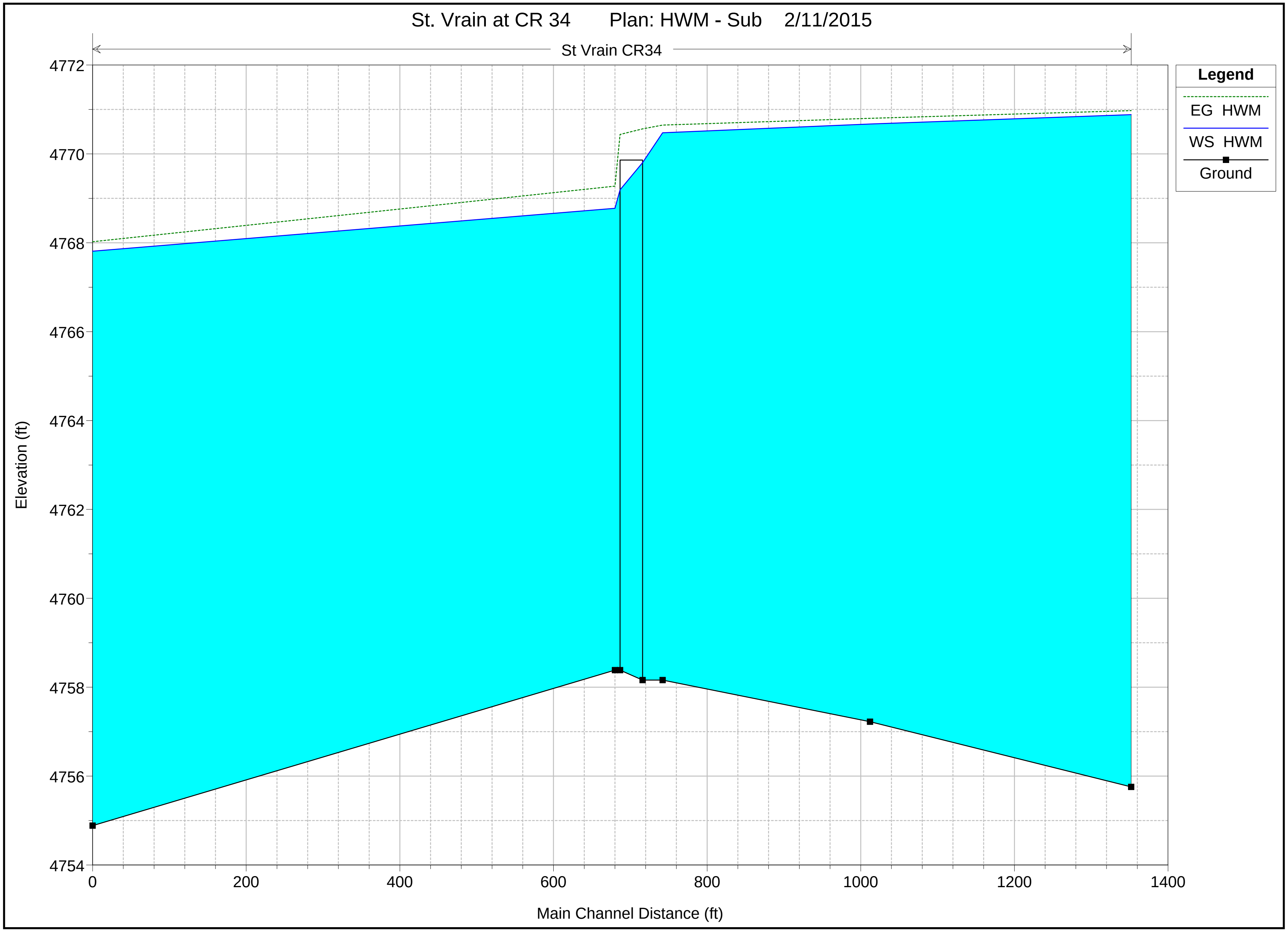


St. Vrain at CR 34

HEC-RAS Plan: HWM - Sub   River: St Vrain   Reach: CR34   Profile: HWM

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
CR34	1852	HWM	27000.00	4755.76	4770.88	4767.78	4770.97	0.000521	3.71	12188.08	3320.76	0.22
CR34	1512	HWM	27000.00	4757.22	4770.67	4767.64	4770.80	0.000477	4.38	12318.03	3639.24	0.22
CR34	1242	HWM	27000.00	4758.16	4770.48	4768.40	4770.65	0.000633	4.66	10680.74	2913.67	0.26
CR34	1206		Bridge									
CR34	1180	HWM	27000.00	4758.38	4768.78	4768.01	4769.28	0.002085	7.36	6862.47	2945.71	0.45
CR34	500	HWM	27000.00	4754.89	4767.81	4766.57	4768.02	0.001435	6.87	9135.60	3445.60	0.38

St. Vrain at CR 34



Appendix B

Flood Frequency Analysis at Stream Flow Gages

Bulletin 17B Frequency Analysis

St. Vrain Creek above Longmont near Hover Street

CDWR Gage SVLONGCO

2002 – 2013   12 years

Bulletin 17B Frequency Analysis
12 Jan 2015 06:01 PM

--- Input Data ---

Analysis Name: DWR ST. VRAIN CREEK LONGMONT
Description:

Data Set Name: ST. VRAIN CREEK LONGMONT DWR
DSS File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\2BT_3St.V_1Lt.H..dss
DSS Pathname: /ST.VRAIN CREEK/LONGMONT, CO/FLOW-PEAK/01jan1900/IR-CENTURY/DWR/

Report File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\DWR_ST._VRAIN_CREEK_LONGMONT\DWR_ST._VRAIN_CREEK_LONGMONT.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\DWR_ST._VRAIN_CREEK_LONGMONT\DWR_ST._VRAIN_CREEK_LONGMONT.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Weibull

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95
Use High Outlier Threshold
High Outlier Threshold: 8146.9

Use Historic Data
Historic Period Start Year: ---
Historic Period End Year: ---

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

<< Plotting Positions >>
ST. VRAIN CREEK LONGMONT DWR

Events Analyzed		Ordered Events			
Day Mon Year	FLOW cfs	Rank	Water Year	FLOW cfs	Weibull Plot Pos
18 Jun 2002	158.0	1	2013	14,000.0*	7.69
30 May 2003	937.0	2	2010	1,090.0	15.38
30 Jun 2004	352.0	3	2005	937.0	23.08
02 Jun 2005	937.0	4	2003	937.0	30.77
08 Jul 2006	408.0	5	2009	781.0	38.46
09 Jun 2007	253.0	6	2011	630.0	46.15
04 Jun 2008	118.0	7	2006	408.0	53.85
26 Jun 2009	781.0	8	2004	352.0	61.54
05 Jun 2010	1,090.0	9	2007	253.0	69.23
09 Jul 2011	630.0	10	2012	227.0	76.92
07 Jul 2012	227.0	11	2002	158.0	84.62
12 Sep 2013	14,000.0	12	2008	118.0	92.31

* Outlier

<< Skew Weighting >>

Based on 12 events, mean-square error of station skew = 0.755
Mean-square error of regional skew = -?

<< Frequency Curve >>
ST. VRAIN CREEK LONGMONT DWR

Computed Curve FLOW, cfs	Expected Probability FLOW, cfs	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, cfs	Confidence Limits 0.95 FLOW, cfs
186,889.8	4,572,144.7	0.2	4,679,627.1	38,018.5
73,228.8	618,491.3	0.5	1,122,917.1	18,704.1
35,722.1	160,513.4	1.0	377,874.8	10,814.8
17,250.0	47,202.8	2.0	125,996.4	6,166.6
6,450.0	10,936.3	5.0	29,066.7	2,836.2
2,992.6	4,002.4	10.0	9,506.9	1,505.0
1,343.2	1,524.9	20.0	3,129.4	735.7
418.9	418.9	50.0	766.7	210.7
199.9	192.2	80.0	368.9	81.0
156.1	147.7	90.0	295.0	57.7
134.9	126.2	95.0	259.4	47.0
114.4	107.8	99.0	224.9	37.2

<< Systematic Statistics >>
ST. VRAIN CREEK LONGMONT DWR

Log Transform: FLOW, cfs		Number of Events	
Mean	2.750	Historic Events	0
Standard Dev	0.544	High Outliers	0
Station Skew	1.473	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	1.473	Systematic Events	12

--- End of Preliminary Results ---

<< High Outlier Test >>

Based on 12 events, 10 percent outlier test deviate K(N) = 2.134
Computed high outlier test value = 8,146.92

1 high outlier(s) identified above input threshold of 8,146.9

```
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
* Note - Collection of historical information and          *
*        comparison with similar data should be explored,  *
*        if not incorporated in this analysis.             *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
```

Statistics and frequency curve adjusted for 1 high outlier(s)

<< Systematic Statistics >>
ST. VRAIN CREEK LONGMONT DWR

Log Transform: FLOW, cfs		Number of Events	
Mean	2.750	Historic Events	0
Standard Dev	0.544	High Outliers	1
Station Skew	1.473	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0

Adopted Skew 1.473	Systematic Events 12

<< Low Outlier Test >>

Based on 12 events, 10 percent outlier test deviate K(N) = 2.134
Computed low outlier test value = 38.9

0 low outlier(s) identified below test value of 38.9

--- Final Results ---

<< Plotting Positions >>
ST. VRAIN CREEK LONGMONT DWR

Events Analyzed			Ordered Events			
Day Mon Year	FLOW cfs	Rank	Water Year	FLOW cfs	Weibull Plot Pos
18 Jun 2002	158.0	1	2013	14,000.0*	7.69
30 May 2003	937.0	2	2010	1,090.0	15.38
30 Jun 2004	352.0	3	2005	937.0	23.08
02 Jun 2005	937.0	4	2003	937.0	30.77
08 Jul 2006	408.0	5	2009	781.0	38.46
09 Jun 2007	253.0	6	2011	630.0	46.15
04 Jun 2008	118.0	7	2006	408.0	53.85
26 Jun 2009	781.0	8	2004	352.0	61.54
05 Jun 2010	1,090.0	9	2007	253.0	69.23
09 Jul 2011	630.0	10	2012	227.0	76.92
07 Jul 2012	227.0	11	2002	158.0	84.62
12 Sep 2013	14,000.0	12	2008	118.0	92.31

* Outlier

<< Skew Weighting >>

Based on 12 events, mean-square error of station skew = 0.755
Mean-square error of regional skew = -?

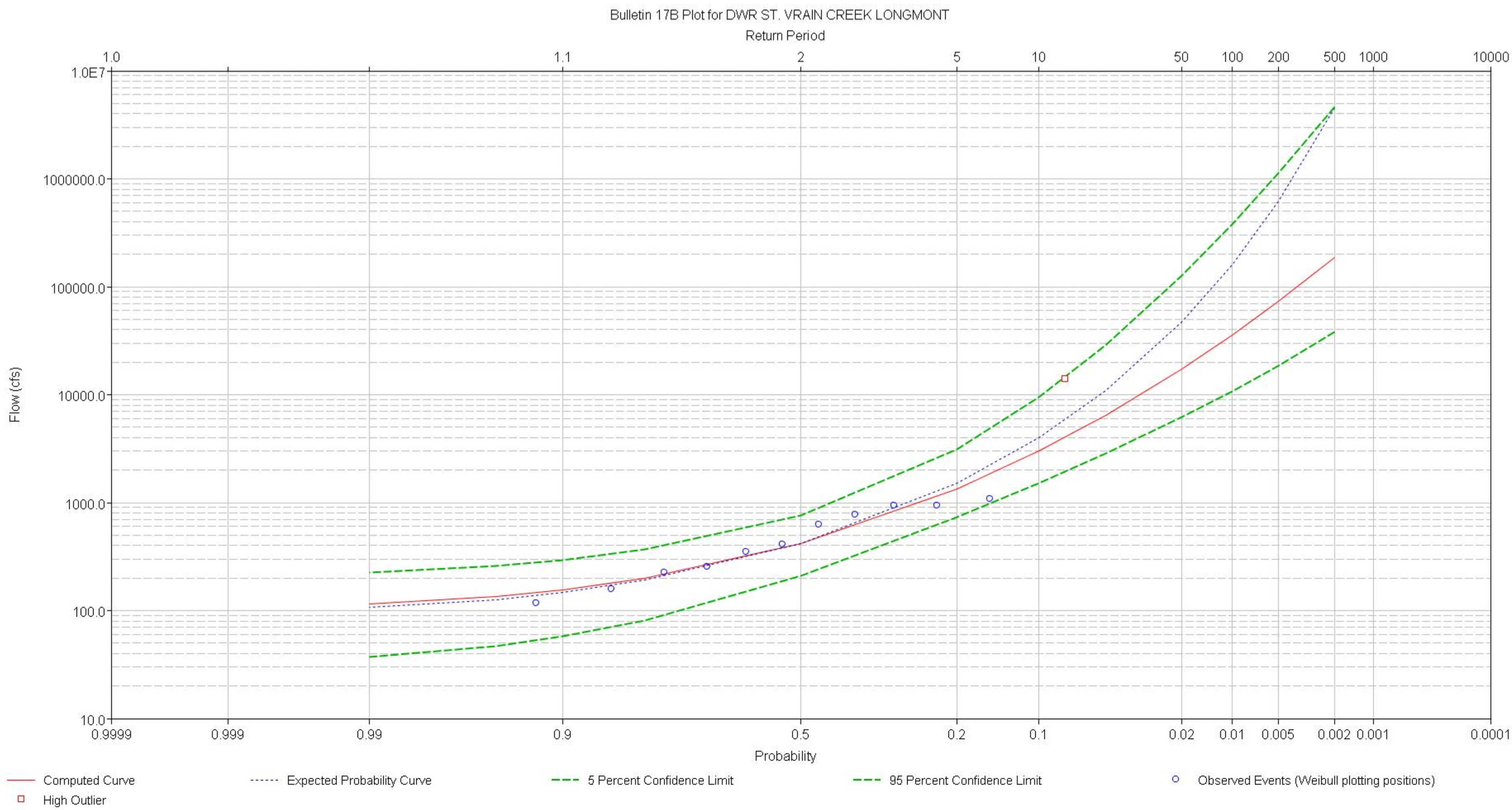
<< Frequency Curve >>
ST. VRAIN CREEK LONGMONT DWR

Computed Curve FLOW, cfs	Expected Probability FLOW, cfs	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, cfs	Confidence Limits 0.95 FLOW, cfs
186,889.8	4,572,144.7	0.2	4,679,627.1	38,018.5
73,228.8	618,491.3	0.5	1,122,917.1	18,704.1
35,722.1	160,513.4	1.0	377,874.8	10,814.8
17,250.0	47,202.8	2.0	125,996.4	6,166.6
6,450.0	10,936.3	5.0	29,066.7	2,836.2
2,992.6	4,002.4	10.0	9,506.9	1,505.0
1,343.2	1,524.9	20.0	3,129.4	735.7
418.9	418.9	50.0	766.7	210.7
199.9	192.2	80.0	368.9	81.0
156.1	147.7	90.0	295.0	57.7
134.9	126.2	95.0	259.4	47.0
114.4	107.8	99.0	224.9	37.2

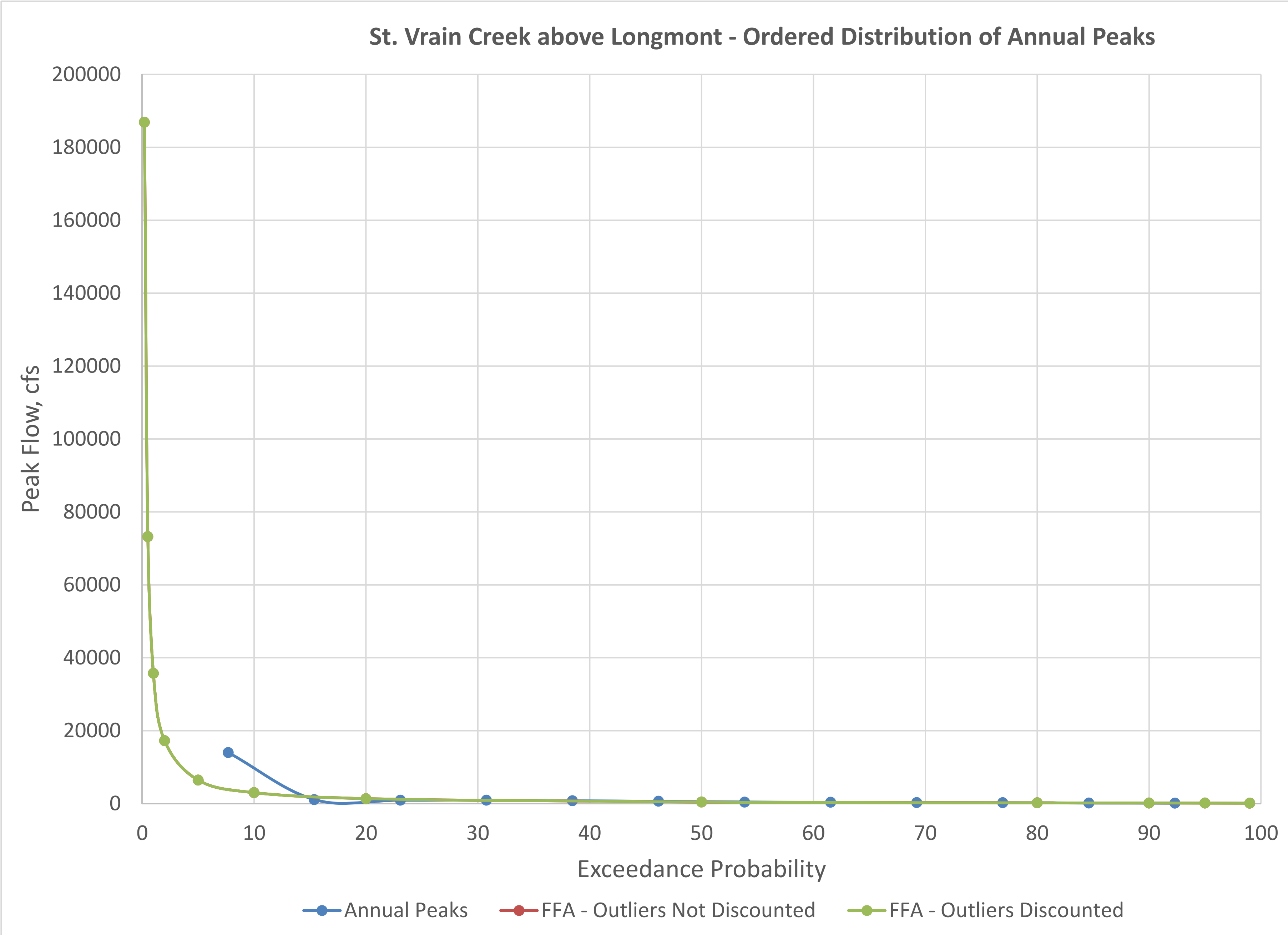
<< Adjusted Statistics >>
ST. VRAIN CREEK LONGMONT DWR

Log Transform: FLOW, cfs		Number of Events	
Mean	2.750	Historic Events	0
Standard Dev	0.544	High Outliers	1
Station Skew	1.473	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	1.473	Systematic Events	12

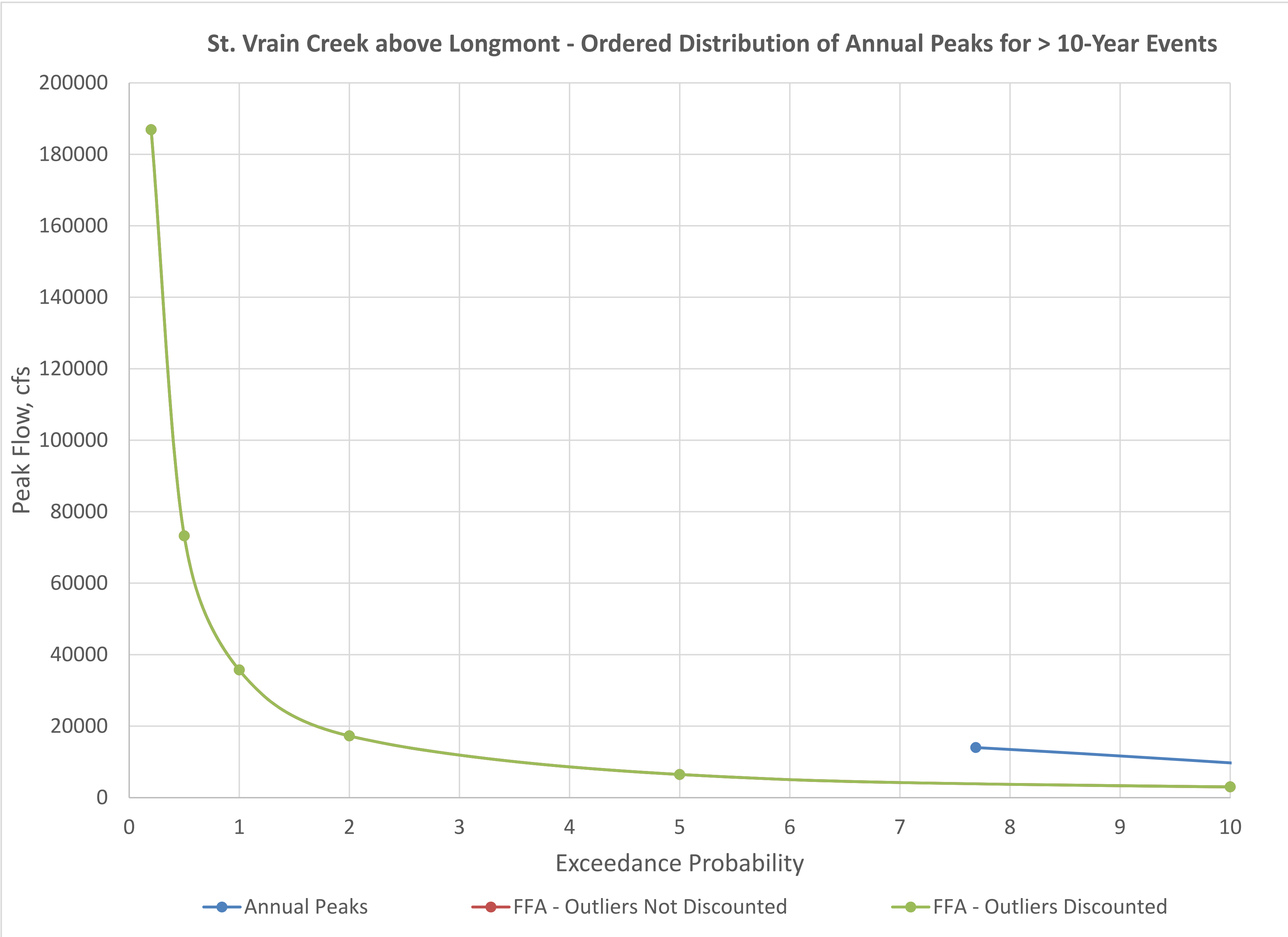
--- End of Analytical Frequency Curve ---

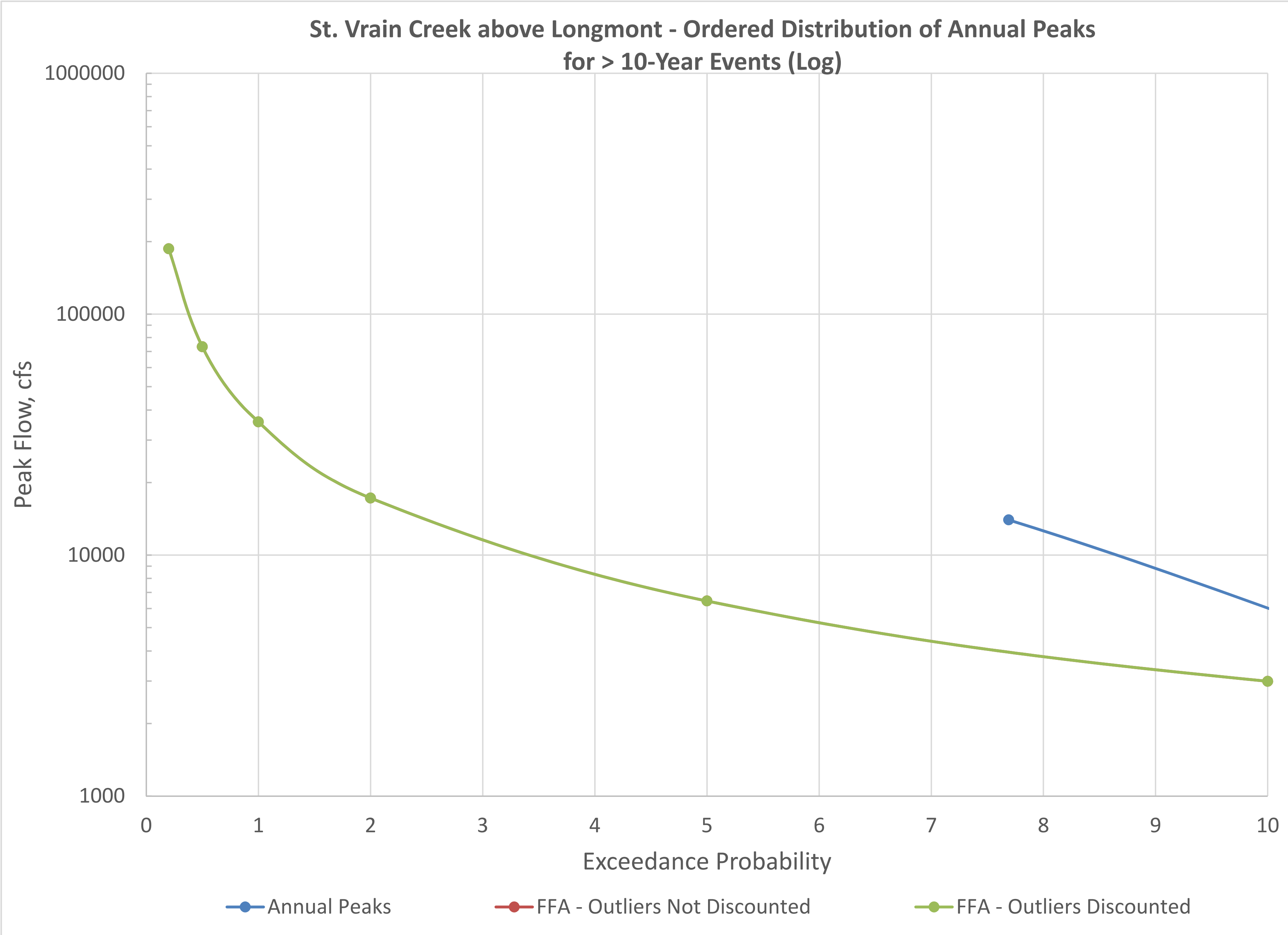


Bulletin 17B Plot for DWR ST. VRAIN CREEK LONGMONT
Return Period
1.0
1.1
2
5
10
50
100
200
500
1000
10000
1.0E7
1000000.0
100000.0
10000.0
1000.0
100.0
10.0
Flow (cfs)
0.9999
0.999
0.99
0.9
0.5
0.2
0.1
0.02
0.01
0.005
0.002
0.001
0.0001
Probability
Computed Curve
Expected Probability Curve
5 Percent Confidence Limit
95 Percent Confidence Limit
Observed Events (Weibull plotting positions)
High Outlier



St. Vrain Creek above Longmont - Ordered Distribution of Annual Peaks
200000
180000
160000
140000
120000
100000
80000
60000
40000
20000
0
Peak Flow, cfs
0
10
20
30
40
50
60
70
80
90
100
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted


St. Vrain Creek above Longmont - Ordered Distribution of Annual Peaks for > 10-Year Events
Peak Flow, cfs
200000
180000
160000
140000
120000
100000
80000
60000
40000
20000
0
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted



St. Vrain Creek above Longmont - Ordered Distribution of Annual Peaks
for > 10-Year Events (Log)
1000000
100000
10000
1000
Peak Flow, cfs
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted

Bulletin 17B Frequency Analysis

St. Vrain Creek below Longmont near Hwy 119

USGS Gage 06725450

CDWR Gage SVCBLOCO

1977 – 2013 (broken) 35 years

Bulletin 17B Frequency Analysis
12 Jan 2015 02:24 PM

--- Input Data ---

Analysis Name: 06725450 St. Vrain CK, Lngmnt
Description: Station 06725450, 2013 Flow 18,500 cfs

Data Set Name: ST. VRAIN CK-LONGMONT, CO 2013
DSS File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\2BT_3St.V_1Lt.H..dss
DSS Pathname: /ST. VRAIN CREEK/LONGMONT, CO/FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/Save Data As: ST. VRAIN CK-LONGMONT, CO 2013/

Report File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\06725450_St._Vrain_CK,_Lngmnt\06725450_St._Vrain_CK,_Lngmnt.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\06725450_St._Vrain_CK,_Lngmnt\06725450_St._Vrain_CK,_Lngmnt.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Weibull

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95
Use High Outlier Threshold
High Outlier Threshold: 8728.2

Use Historic Data
Historic Period Start Year: ---
Historic Period End Year: ---

Display ordinate values using 0 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

<< Plotting Positions >>
ST. VRAIN CK-LONGMONT, CO 2013

Events Analyzed Day Mon Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Weibull Plot Pos
25 Jul 1977	314	1	2013	18,500*	2.78
17 May 1978	2,370	2	1999	3,600	5.56
09 Jun 1979	1,310	3	1995	2,960	8.33
01 May 1980	2,380	4	1980	2,380	11.11
13 Aug 1981	201	5	1978	2,370	13.89
14 Sep 1982	1,090	6	1997	1,700	16.67
05 Feb 1985	850	7	2010	1,520	19.44
10 Jun 1986	1,030	8	1991	1,520	22.22
09 Jun 1987	686	9	2005	1,450	25.00
04 Aug 1988	755	10	2009	1,320	27.78
03 Jun 1989	1,300	11	1979	1,310	30.56
29 May 1990	1,020	12	1989	1,300	33.33
02 Jun 1991	1,520	13	1993	1,250	36.11
12 Jun 1992	589	14	1996	1,210	38.89
18 Jun 1993	1,250	15	1982	1,090	41.67
11 Aug 1994	897	16	2003	1,070	44.44
30 May 1995	2,960	17	2011	1,040	47.22
22 Jun 1996	1,210	18	2006	1,040	50.00

07 Jun 1997	1,700	19	1986	1,030	52.78
26 Apr 1998	870	20	1990	1,020	55.56
30 Apr 1999	3,600	21	2008	901	58.33
31 May 2000	462	22	1994	897	61.11
11 Jul 2001	567	23	1998	870	63.89
23 May 2002	213	24	1985	850	66.67
01 Jun 2003	1,070	25	2004	847	69.44
23 Jul 2004	847	26	1988	755	72.22
04 Jun 2005	1,450	27	2007	710	75.00
09 Jul 2006	1,040	28	1987	686	77.78
24 Sep 2007	710	29	2012	671	80.56
16 Aug 2008	901	30	1992	589	83.33
27 Jun 2009	1,320	31	2001	567	86.11
12 Jun 2010	1,520	32	2000	462	88.89
14 Jul 2011	1,040	33	1977	314	91.67
07 Jul 2012	671	34	2002	213	94.44
13 Sep 2013	18,500	35	1981	201	97.22

* Outlier

<< Skew Weighting >>

```
----------------------------------------------------------------
Based on 35 events, mean-square error of station skew =     0.245
Mean-square error of regional skew =                           -?
----------------------------------------------------------------
```

<< Frequency Curve >>

ST. VRAIN CK-LONGMONT, CO 2013

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
26,959	40,896	0.2	62,704	15,157
16,858	22,688	0.5	35,090	10,187
11,693	14,558	1.0	22,353	7,464
8,014	9,362	2.0	14,059	5,403
4,744	5,173	5.0	7,425	3,432
3,103	3,267	10.0	4,464	2,359
1,953	2,001	20.0	2,600	1,543
934	934	50.0	1,168	739
534	528	80.0	679	397
425	417	90.0	551	304
363	353	95.0	477	253
288	277	99.0	389	192

<< Systematic Statistics >>

ST. VRAIN CK-LONGMONT, CO 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	3.025	Historic Events	0
Standard Dev	0.349	High Outliers	0
Station Skew	0.954	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.954	Systematic Events	35

--- End of Preliminary Results ---

<< High Outlier Test >>

Based on 35 events, 10 percent outlier test deviate K(N) = 2.628

Computed high outlier test value = 8,728.2

1 high outlier(s) identified above input threshold of 8,728.2

* *
* Note - Collection of historical information and *
* comparison with similar data should be explored, *
* if not incorporated in this analysis. *
* *

Statistics and frequency curve adjusted for 1 high outlier(s)

<< Systematic Statistics >>
ST. VRAIN CK-LONGMONT, CO 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	3.023	Historic Events	0
Standard Dev	0.345	High Outliers	1
Station Skew	0.918	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.954	Systematic Events	35
		Historic Period	37

<< Low Outlier Test >>

Based on 37 events, 10 percent outlier test deviate K(N) = 2.65
Computed low outlier test value = 128.4

0 low outlier(s) identified below test value of 128.4

--- Final Results ---

<< Plotting Positions >>
ST. VRAIN CK-LONGMONT, CO 2013

Events Analyzed Day Mon Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Weibull Plot Pos
25 Jul 1977	314	1	2013	18,500*	2.63
17 May 1978	2,370	2	1999	3,600	5.34
09 Jun 1979	1,310	3	1995	2,960	8.13
01 May 1980	2,380	4	1980	2,380	10.91
13 Aug 1981	201	5	1978	2,370	13.70
14 Sep 1982	1,090	6	1997	1,700	16.49
05 Feb 1985	850	7	2010	1,520	19.27
10 Jun 1986	1,030	8	1991	1,520	22.06
09 Jun 1987	686	9	2005	1,450	24.85
04 Aug 1988	755	10	2009	1,320	27.63
03 Jun 1989	1,300	11	1979	1,310	30.42
29 May 1990	1,020	12	1989	1,300	33.20
02 Jun 1991	1,520	13	1993	1,250	35.99
12 Jun 1992	589	14	1996	1,210	38.78
18 Jun 1993	1,250	15	1982	1,090	41.56
11 Aug 1994	897	16	2003	1,070	44.35
30 May 1995	2,960	17	2011	1,040	47.14
22 Jun 1996	1,210	18	2006	1,040	49.92
07 Jun 1997	1,700	19	1986	1,030	52.71
26 Apr 1998	870	20	1990	1,020	55.50
30 Apr 1999	3,600	21	2008	901	58.28
31 May 2000	462	22	1994	897	61.07
11 Jul 2001	567	23	1998	870	63.85
23 May 2002	213	24	1985	850	66.64

01 Jun 2003	1,070	25	2004	847	69.43
23 Jul 2004	847	26	1988	755	72.21
04 Jun 2005	1,450	27	2007	710	75.00
09 Jul 2006	1,040	28	1987	686	77.79
24 Sep 2007	710	29	2012	671	80.57
16 Aug 2008	901	30	1992	589	83.36
27 Jun 2009	1,320	31	2001	567	86.15
12 Jun 2010	1,520	32	2000	462	88.93
14 Jul 2011	1,040	33	1977	314	91.72
07 Jul 2012	671	34	2002	213	94.50
13 Sep 2013	18,500	35	1981	201	97.29

Note: Plotting positions based on historic period (H) = 37
Number of historic events plus high outliers (Z) = 1
Weighting factor for systematic events (W) = 1.0588

* Outlier

<< Skew Weighting >>

Based on 37 events, mean-square error of station skew = 0.228
Mean-square error of regional skew = -?

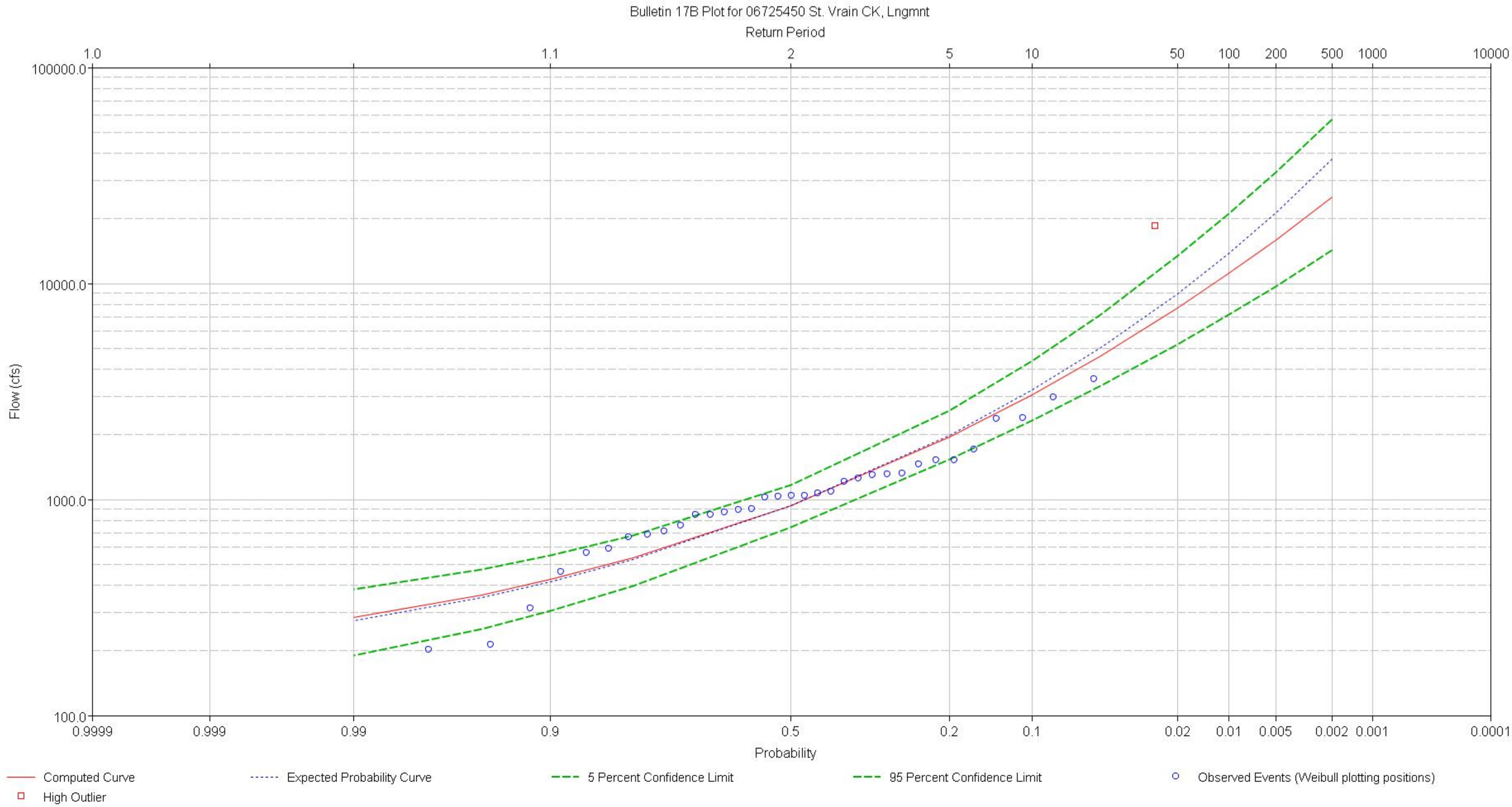
<< Frequency Curve >>
ST. VRAIN CK-LONGMONT, CO 2013

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
25,097	37,594	0.2	57,405	14,269
15,908	21,227	0.5	32,672	9,700
11,146	13,793	1.0	21,076	7,167
7,716	8,976	2.0	13,419	5,232
4,625	5,034	5.0	7,198	3,359
3,053	3,211	10.0	4,375	2,327
1,938	1,984	20.0	2,574	1,535
935	935	50.0	1,167	742
535	529	80.0	679	399
425	417	90.0	549	305
362	352	95.0	475	252
285	273	99.0	384	190

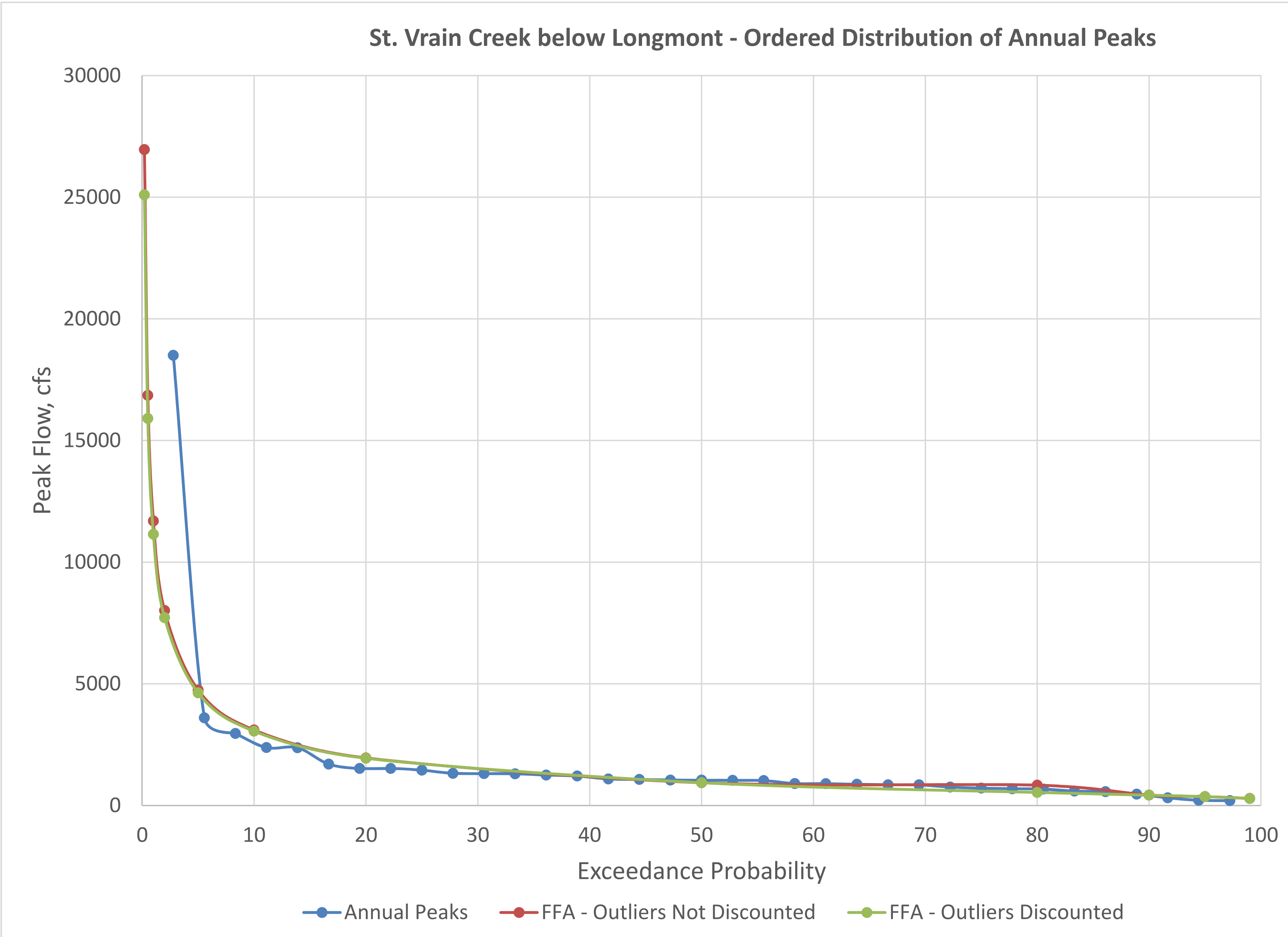
<< Adjusted Statistics >>
ST. VRAIN CK-LONGMONT, CO 2013

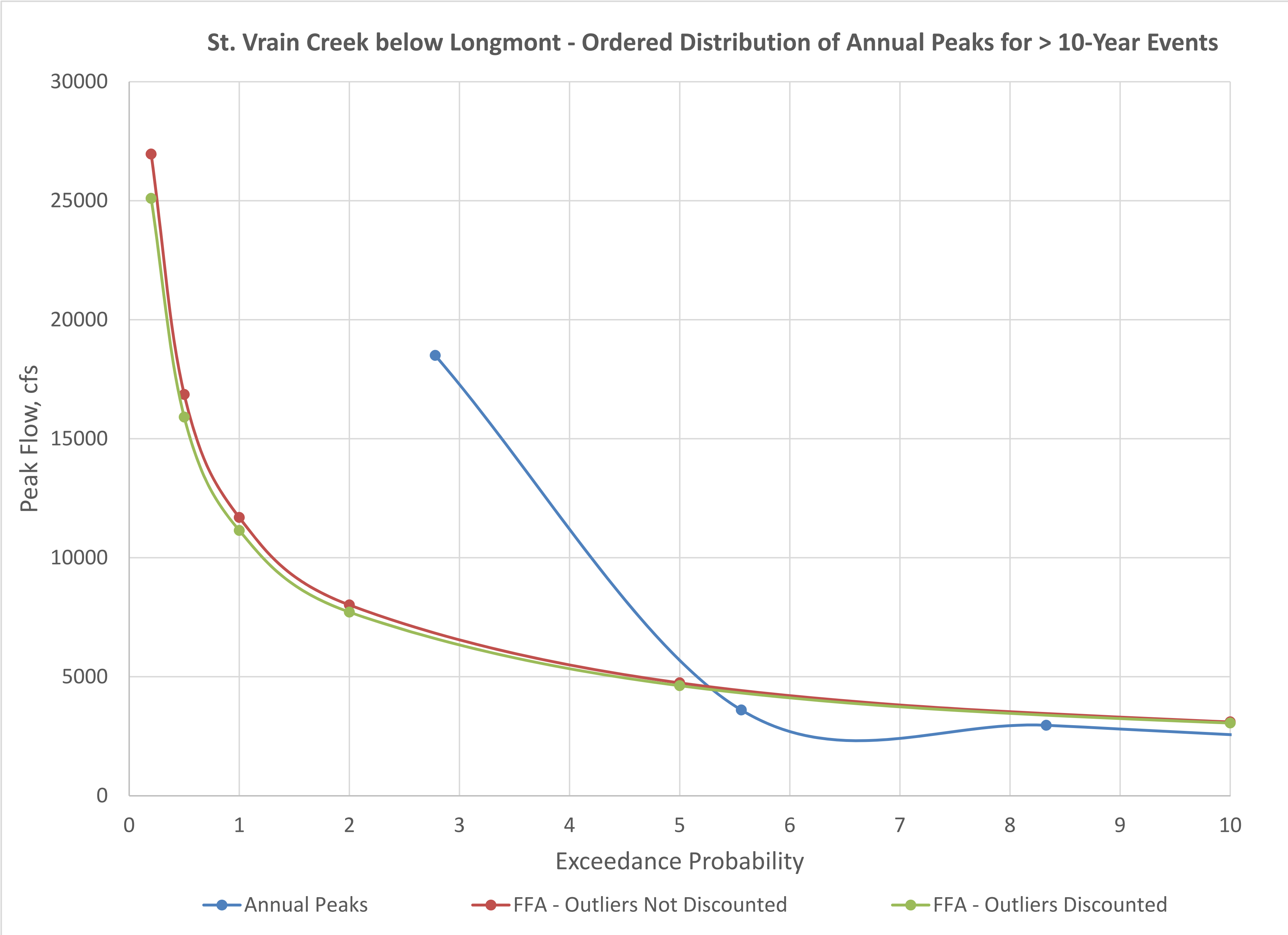
Log Transform: FLOW, CFS		Number of Events	
Mean	3.023	Historic Events	0
Standard Dev	0.345	High Outliers	1
Station Skew	0.918	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.918	Systematic Events	35
		Historic Period	37

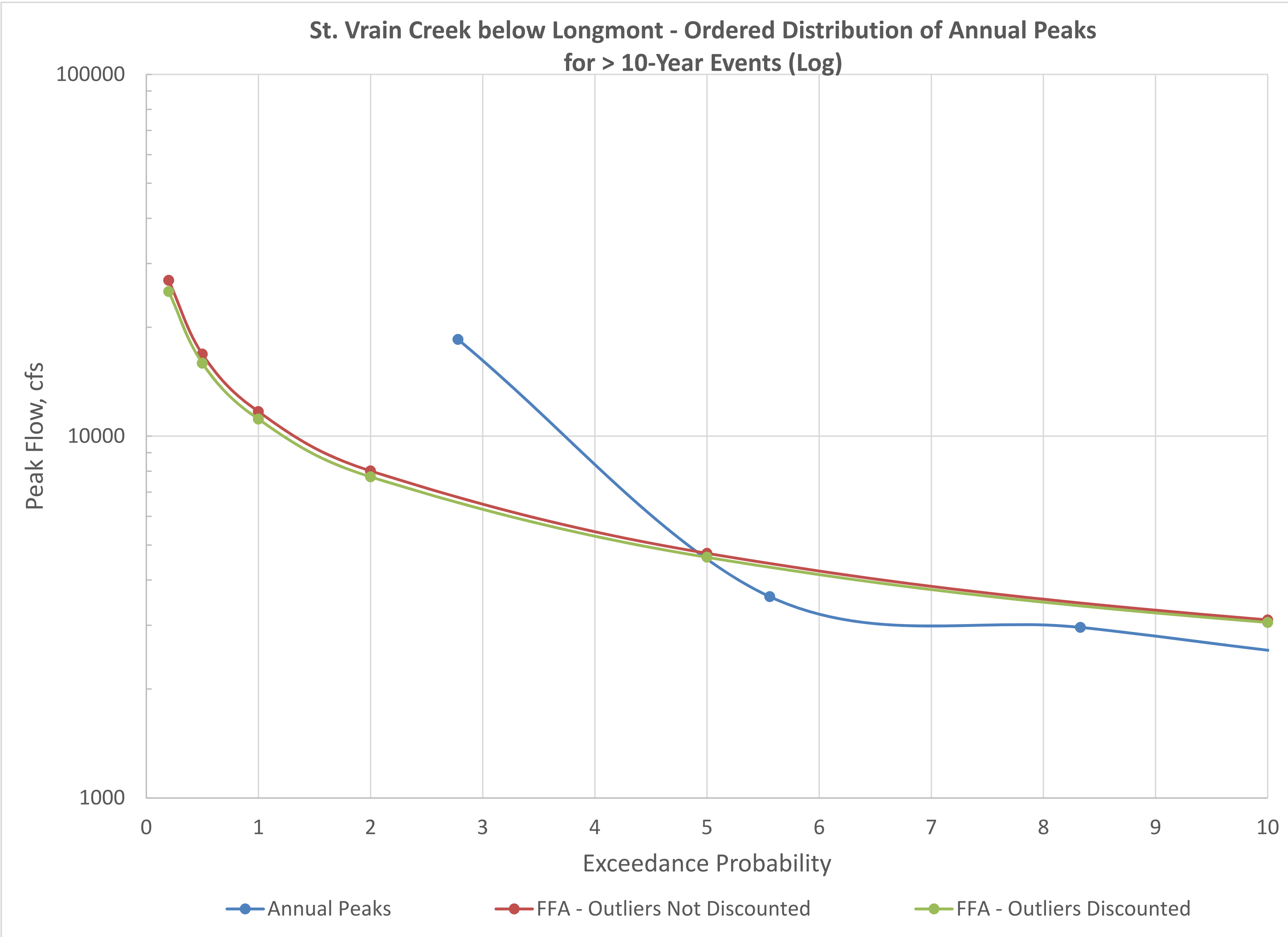
--- End of Analytical Frequency Curve ---



Bulletin 17B Plot for 06725450 St. Vrain CK, Lngmnt
Return Period
1.0
1.1
2
5
10
50
100
200
500
1000
10000
100000.0
10000.0
1000.0
100.0
Flow (cfs)
0.9999
0.999
0.99
0.9
0.5
0.2
0.1
0.02
0.01
0.005
0.002
0.001
0.0001
Probability
Computed Curve
Expected Probability Curve
5 Percent Confidence Limit
95 Percent Confidence Limit
Observed Events (Weibull plotting positions)
High Outlier


St. Vrain Creek below Longmont - Ordered Distribution of Annual Peaks
Peak Flow, cfs
30000
25000
20000
15000
10000
5000
0
0
10
20
30
40
50
60
70
80
90
100
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted


St. Vrain Creek below Longmont - Ordered Distribution of Annual Peaks for > 10-Year Events
Peak Flow, cfs
30000
25000
20000
15000
10000
5000
0
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted


St. Vrain Creek below Longmont - Ordered Distribution of Annual Peaks
for > 10-Year Events (Log)
100000
10000
1000
Peak Flow, cfs
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted

Bulletin 17B Frequency Analysis

St. Vrain Creek at Mouth near Platteville

USGS Gage 06731000

CDWR Gage SVCPLACO

1905 – 2013 (broken) 88 years

Bulletin 17B Frequency Analysis
12 Jan 2015 02:24 PM

--- Input Data ---

Analysis Name: 06731000 St. Vrain Ck, Mouth
Description: USGS website. Station 06731000 +DWR +2013 of 27,900 cfs

Data Set Name: ST. VRAIN CK-MO, 2013
DSS File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\2BT_3St.V_1Lt.H..dss
DSS Pathname: /ST. VRAIN CREEK/MOUTH, NEAR PLATTEVILLE, CO./FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/Save Data As: ST. VRAIN CK-MO, 2013/

Report File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\06731000_St._Vrain_Ck,_Mouth\06731000_St._Vrain_Ck,_Mouth.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\06731000_St._Vrain_Ck,_Mouth\06731000_St._Vrain_Ck,_Mouth.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Weibull

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95
Use High Outlier Threshold
High Outlier Threshold: 20790.1

Use Historic Data
Historic Period Start Year: ---
Historic Period End Year: ---

Display ordinate values using 0 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

<< Plotting Positions >>
ST. VRAIN CK-MO, 2013

Events Analyzed				Ordered Events		
		FLOW		Water	FLOW	Weibull
Day Mon	Year	CFS	Rank	Year	CFS	Plot Pos
11 Jun	1905	2,680	1	2013	27,900*	1.12
06 May	1906	1,620	2	1938	11,300	2.25
29 Jul	1927	1,470	3	1969	10,300	3.37
28 May	1928	1,970	4	1957	9,450	4.49
04 Aug	1929	1,200	5	1949	6,150	5.62
15 Aug	1930	1,310	6	1947	5,920	6.74
06 Jun	1931	662	7	1973	5,620	7.87
13 Jul	1932	320	8	1951	5,390	8.99
20 May	1933	1,870	9	1995	5,190	10.11
14 Jun	1934	2,380	10	1942	4,940	11.24
28 May	1935	2,360	11	1980	4,860	12.36
11 Jun	1936	1,420	12	1958	4,420	13.48
27 Jun	1937	1,990	13	1999	3,720	14.61
03 Sep	1938	11,300	14	1971	3,700	15.73
02 May	1939	595	15	1965	3,700	16.85
03 Jul	1940	1,420	16	1952	3,480	17.98
23 Jun	1941	1,740	17	1970	3,470	19.10
03 May	1942	4,940	18	1979	3,400	20.22

19 May 1943	1,620	19	1997	3,310	21.35
13 May 1944	2,390	20	1961	3,220	22.47
26 Jun 1945	1,230	21	2010	3,100	23.60
18 Jul 1946	1,820	22	1978	3,060	24.72
23 Jun 1947	5,920	23	1983	2,830	25.84
15 Oct 1947	874	24	1905	2,680	26.97
07 Jun 1949	6,150	25	1963	2,630	28.09
26 May 1950	715	26	1967	2,400	29.21
04 Aug 1951	5,390	27	1944	2,390	30.34
24 May 1952	3,480	28	1934	2,380	31.46
14 Jun 1953	593	29	1935	2,360	32.58
15 Jul 1954	178	30	1986	2,260	33.71
15 Jun 1955	360	31	2011	2,180	34.83
28 Jul 1956	589	32	2003	2,070	35.96
09 May 1957	9,450	33	1996	2,010	37.08
09 May 1958	4,420	34	1937	1,990	38.20
22 May 1959	1,890	35	1928	1,970	39.33
06 May 1960	1,210	36	1989	1,900	40.45
04 Jun 1961	3,220	37	1959	1,890	41.57
01 Jul 1962	754	38	1933	1,870	42.70
17 Jun 1963	2,630	39	1946	1,820	43.82
30 May 1964	528	40	1974	1,810	44.94
25 Jul 1965	3,700	41	1991	1,770	46.07
02 Sep 1966	1,300	42	2009	1,760	47.19
21 Jun 1967	2,400	43	1941	1,740	48.31
10 Aug 1968	590	44	1972	1,730	49.44
08 May 1969	10,300	45	2005	1,660	50.56
12 Jun 1970	3,470	46	1943	1,620	51.69
26 Apr 1971	3,700	47	1906	1,620	52.81
06 Jun 1972	1,730	48	1987	1,490	53.93
07 May 1973	5,620	49	2006	1,480	55.06
09 Jun 1974	1,810	50	1927	1,470	56.18
11 Jun 1975	1,420	51	1975	1,420	57.30
03 Aug 1976	795	52	1940	1,420	58.43
26 Jul 1977	868	53	1936	1,420	59.55
18 May 1978	3,060	54	1984	1,390	60.67
10 Jun 1979	3,400	55	2004	1,370	61.80
01 May 1980	4,860	56	1993	1,360	62.92
29 May 1981	491	57	1982	1,350	64.04
13 May 1982	1,350	58	1930	1,310	65.17
19 May 1983	2,830	59	1966	1,300	66.29
26 May 1984	1,390	60	1998	1,290	67.42
10 Jun 1985	1,050	61	2008	1,240	68.54
10 Jun 1986	2,260	62	1945	1,230	69.66
09 Jun 1987	1,490	63	1960	1,210	70.79
20 May 1988	848	64	1929	1,200	71.91
04 Jun 1989	1,900	65	1990	1,120	73.03
12 Jun 1990	1,120	66	1985	1,050	74.16
02 Jun 1991	1,770	67	2012	1,020	75.28
25 Aug 1992	1,020	68	1992	1,020	76.40
18 Jun 1993	1,360	69	2000	960	77.53
03 Jun 1994	799	70	1948	874	78.65
30 May 1995	5,190	71	1977	868	79.78
27 May 1996	2,010	72	1988	848	80.90
07 Jun 1997	3,310	73	1994	799	82.02
26 Apr 1998	1,290	74	1976	795	83.15
01 May 1999	3,720	75	1962	754	84.27
16 Jul 2000	960	76	2001	744	85.39
06 May 2001	744	77	1950	715	86.52
23 May 2002	670	78	2002	670	87.64
31 May 2003	2,070	79	1931	662	88.76
24 Jul 2004	1,370	80	1939	595	89.89
04 Jun 2005	1,660	81	1953	593	91.01
09 Jul 2006	1,480	82	1968	590	92.13
16 Aug 2008	1,240	83	1956	589	93.26
28 Jun 2009	1,760	84	1964	528	94.38
13 Jun 2010	3,100	85	1981	491	95.51
15 Jul 2011	2,180	86	1955	360	96.63
07 Jul 2012	1,020	87	1932	320	97.75
13 Sep 2013	27,900	88	1954	178	98.88

* Outlier

<< Skew Weighting >>

Based on 88 events, mean-square error of station skew =	0.078
Mean-square error of regional skew =	-?

<< Frequency Curve >>
ST. VRAIN CK-MO, 2013

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
27,212	30,312	0.2	41,477	19,472
19,495	21,121	0.5	28,443	14,422
14,912	15,855	1.0	21,019	11,322
11,210	11,727	2.0	15,245	8,741
7,411	7,614	5.0	9,594	5,993
5,205	5,292	10.0	6,483	4,325
3,453	3,482	20.0	4,142	2,942
1,661	1,661	50.0	1,924	1,432
855	849	80.0	1,004	711
620	612	90.0	742	501
481	473	95.0	588	379
309	299	99.0	392	231

<< Systematic Statistics >>
ST. VRAIN CK-MO, 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	3.241	Historic Events	0
Standard Dev	0.362	High Outliers	0
Station Skew	0.343	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.343	Systematic Events	88

--- End of Preliminary Results ---

<< Low Outlier Test >>

Based on 88 events, 10 percent outlier test deviate K(N) = 2.973
Computed low outlier test value = 145.9

0 low outlier(s) identified below test value of 145.9

<< High Outlier Test >>

Based on 88 events, 10 percent outlier test deviate K(N) = 2.973
Computed high outlier test value = 20,790.1

1 high outlier(s) identified above input threshold of 20,790.1

* Note - Collection of historical information and comparison with similar data should be explored, if not incorporated in this analysis.

Statistics and frequency curve adjusted for 1 high outlier(s)

<< Systematic Statistics >>
ST. VRAIN CK-MO, 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	3.238	Historic Events	0
Standard Dev	0.358	High Outliers	1
Station Skew	0.290	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.343	Systematic Events	88
		Historic Period	109

--- Final Results ---

<< Plotting Positions >>
ST. VRAIN CK-MO, 2013

Events Analyzed Day Mon Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Weibull Plot Pos
11 Jun 1905	2,680	1	2013	27,900*	0.91
06 May 1906	1,620	2	1938	11,300	1.93
29 Jul 1927	1,470	3	1969	10,300	3.06
28 May 1928	1,970	4	1957	9,450	4.18
04 Aug 1929	1,200	5	1949	6,150	5.31
15 Aug 1930	1,310	6	1947	5,920	6.44
06 Jun 1931	662	7	1973	5,620	7.57
13 Jul 1932	320	8	1951	5,390	8.70
20 May 1933	1,870	9	1995	5,190	9.83
14 Jun 1934	2,380	10	1942	4,940	10.96
28 May 1935	2,360	11	1980	4,860	12.08
11 Jun 1936	1,420	12	1958	4,420	13.21
27 Jun 1937	1,990	13	1999	3,720	14.34
03 Sep 1938	11,300	14	1971	3,700	15.47
02 May 1939	595	15	1965	3,700	16.60
03 Jul 1940	1,420	16	1952	3,480	17.73
23 Jun 1941	1,740	17	1970	3,470	18.86
03 May 1942	4,940	18	1979	3,400	19.98
19 May 1943	1,620	19	1997	3,310	21.11
13 May 1944	2,390	20	1961	3,220	22.24
26 Jun 1945	1,230	21	2010	3,100	23.37
18 Jul 1946	1,820	22	1978	3,060	24.50
23 Jun 1947	5,920	23	1983	2,830	25.63
15 Oct 1947	874	24	1905	2,680	26.76
07 Jun 1949	6,150	25	1963	2,630	27.88
26 May 1950	715	26	1967	2,400	29.01
04 Aug 1951	5,390	27	1944	2,390	30.14
24 May 1952	3,480	28	1934	2,380	31.27
14 Jun 1953	593	29	1935	2,360	32.40
15 Jul 1954	178	30	1986	2,260	33.53
15 Jun 1955	360	31	2011	2,180	34.66
28 Jul 1956	589	32	2003	2,070	35.78
09 May 1957	9,450	33	1996	2,010	36.91
09 May 1958	4,420	34	1937	1,990	38.04
22 May 1959	1,890	35	1928	1,970	39.17
06 May 1960	1,210	36	1989	1,900	40.30
04 Jun 1961	3,220	37	1959	1,890	41.43
01 Jul 1962	754	38	1933	1,870	42.55
17 Jun 1963	2,630	39	1946	1,820	43.68
30 May 1964	528	40	1974	1,810	44.81
25 Jul 1965	3,700	41	1991	1,770	45.94
02 Sep 1966	1,300	42	2009	1,760	47.07
21 Jun 1967	2,400	43	1941	1,740	48.20
10 Aug 1968	590	44	1972	1,730	49.33
08 May 1969	10,300	45	2005	1,660	50.45

12 Jun 1970	3,470	46	1943	1,620	51.58
26 Apr 1971	3,700	47	1906	1,620	52.71
06 Jun 1972	1,730	48	1987	1,490	53.84
07 May 1973	5,620	49	2006	1,480	54.97
09 Jun 1974	1,810	50	1927	1,470	56.10
11 Jun 1975	1,420	51	1975	1,420	57.23
03 Aug 1976	795	52	1940	1,420	58.35
26 Jul 1977	868	53	1936	1,420	59.48
18 May 1978	3,060	54	1984	1,390	60.61
10 Jun 1979	3,400	55	2004	1,370	61.74
01 May 1980	4,860	56	1993	1,360	62.87
29 May 1981	491	57	1982	1,350	64.00
13 May 1982	1,350	58	1930	1,310	65.13
19 May 1983	2,830	59	1966	1,300	66.25
26 May 1984	1,390	60	1998	1,290	67.38
10 Jun 1985	1,050	61	2008	1,240	68.51
10 Jun 1986	2,260	62	1945	1,230	69.64
09 Jun 1987	1,490	63	1960	1,210	70.77
20 May 1988	848	64	1929	1,200	71.90
04 Jun 1989	1,900	65	1990	1,120	73.03
12 Jun 1990	1,120	66	1985	1,050	74.15
02 Jun 1991	1,770	67	2012	1,020	75.28
25 Aug 1992	1,020	68	1992	1,020	76.41
18 Jun 1993	1,360	69	2000	960	77.54
03 Jun 1994	799	70	1948	874	78.67
30 May 1995	5,190	71	1977	868	79.80
27 May 1996	2,010	72	1988	848	80.92
07 Jun 1997	3,310	73	1994	799	82.05
26 Apr 1998	1,290	74	1976	795	83.18
01 May 1999	3,720	75	1962	754	84.31
16 Jul 2000	960	76	2001	744	85.44
06 May 2001	744	77	1950	715	86.57
23 May 2002	670	78	2002	670	87.70
31 May 2003	2,070	79	1931	662	88.82
24 Jul 2004	1,370	80	1939	595	89.95
04 Jun 2005	1,660	81	1953	593	91.08
09 Jul 2006	1,480	82	1968	590	92.21
16 Aug 2008	1,240	83	1956	589	93.34
28 Jun 2009	1,760	84	1964	528	94.47
13 Jun 2010	3,100	85	1981	491	95.60
15 Jul 2011	2,180	86	1955	360	96.72
07 Jul 2012	1,020	87	1932	320	97.85
13 Sep 2013	27,900	88	1954	178	98.98

Note: Plotting positions based on historic period (H) = 109
Number of historic events plus high outliers (Z) = 1
Weighting factor for systematic events (W) = 1.2414

* Outlier

<< Skew Weighting >>

Based on 109 events, mean-square error of station skew = 0.063
Mean-square error of regional skew = -?

<< Frequency Curve >>
ST. VRAIN CK-MO, 2013

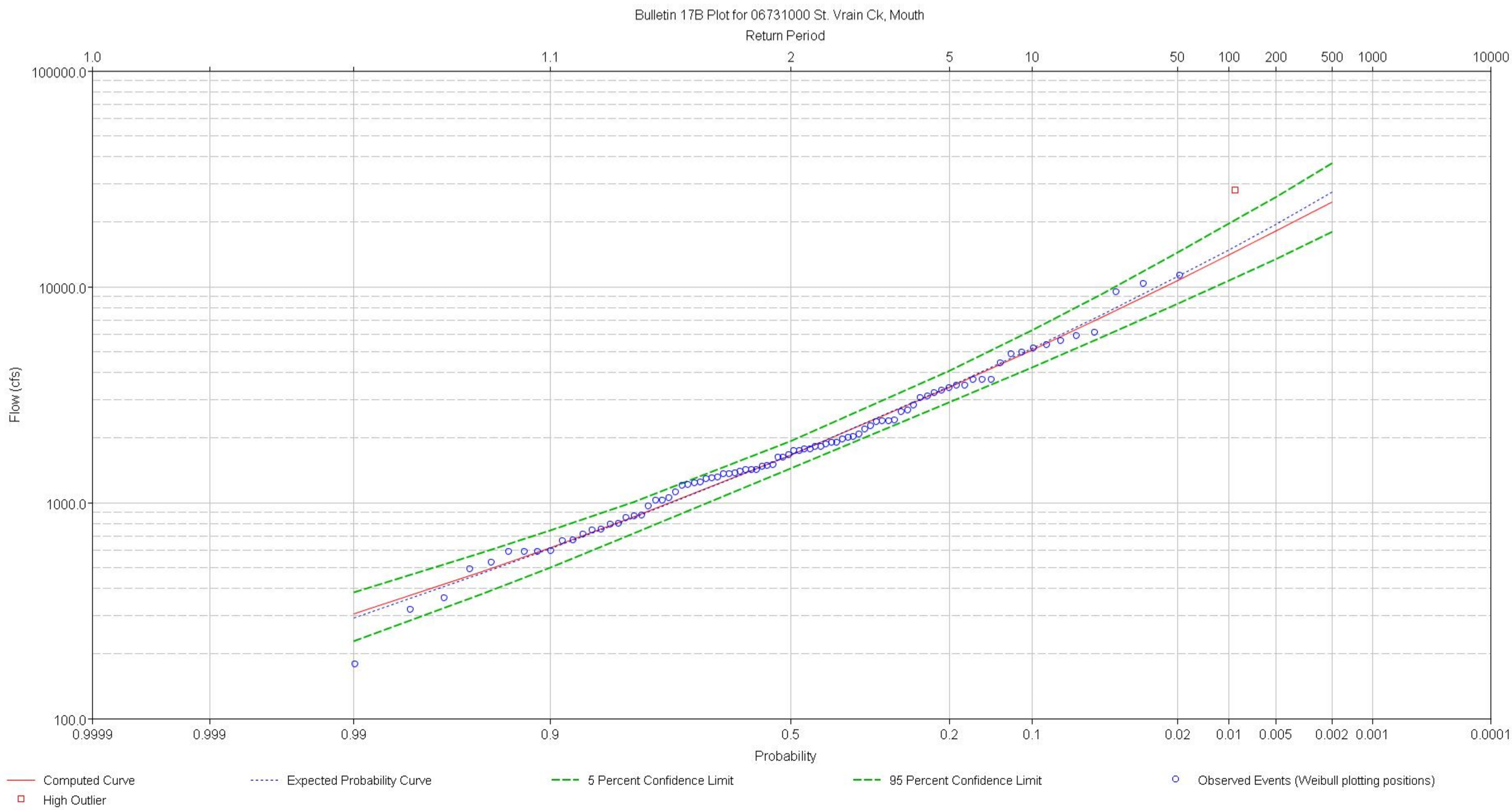
Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
24,789	27,454	0.2	37,324	17,908
18,060	19,488	0.5	26,089	13,464
13,990	14,832	1.0	19,559	10,690
10,649	11,118	2.0	14,388	8,347
7,156	7,345	5.0	9,222	5,807
5,086	5,169	10.0	6,316	4,238
3,414	3,442	20.0	4,086	2,914
1,664	1,664	50.0	1,923	1,437

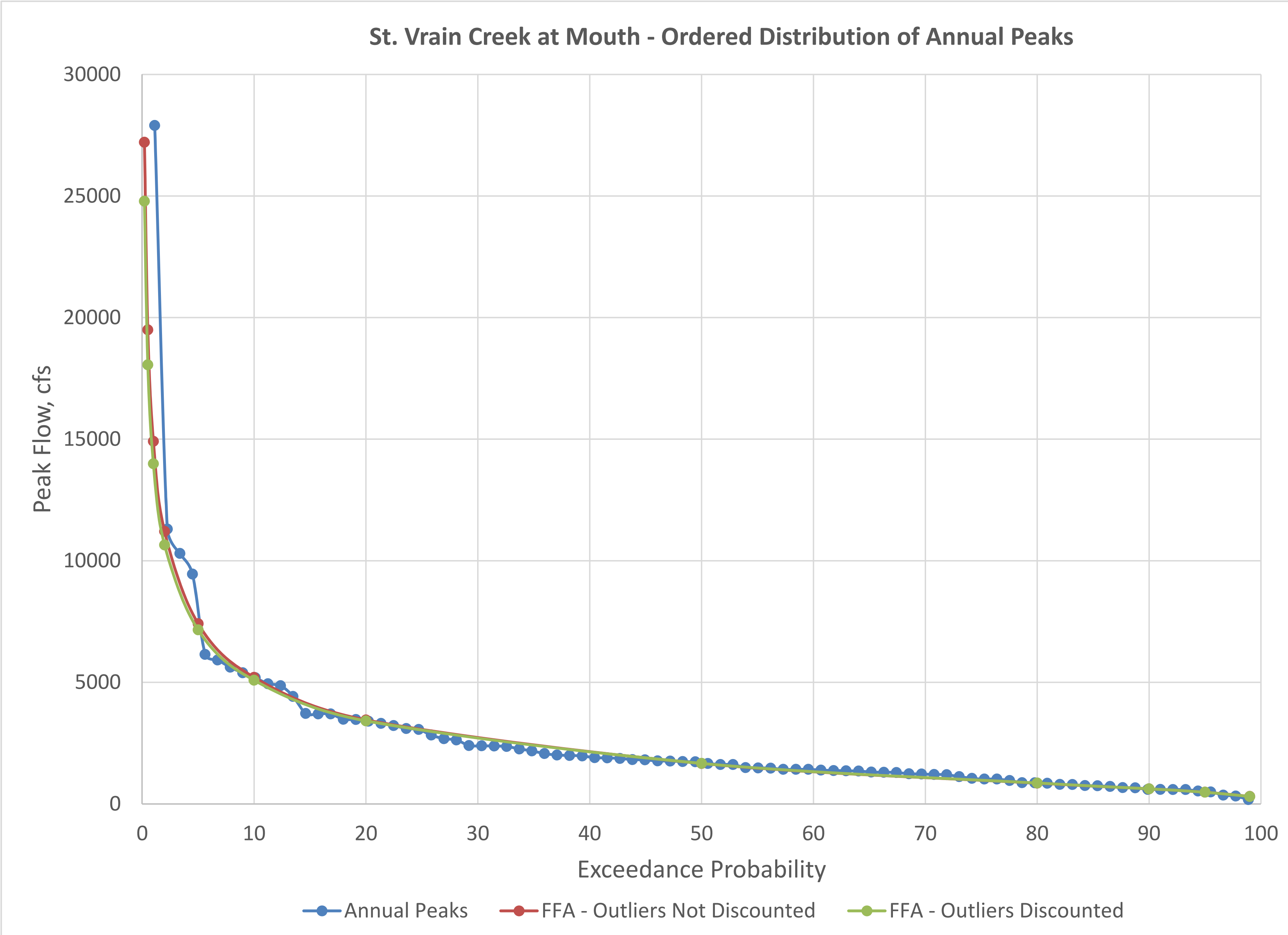
858	852	80.0	1,006	715
620	613	90.0	742	502
480	471	95.0	585	378
304	293	99.0	385	227

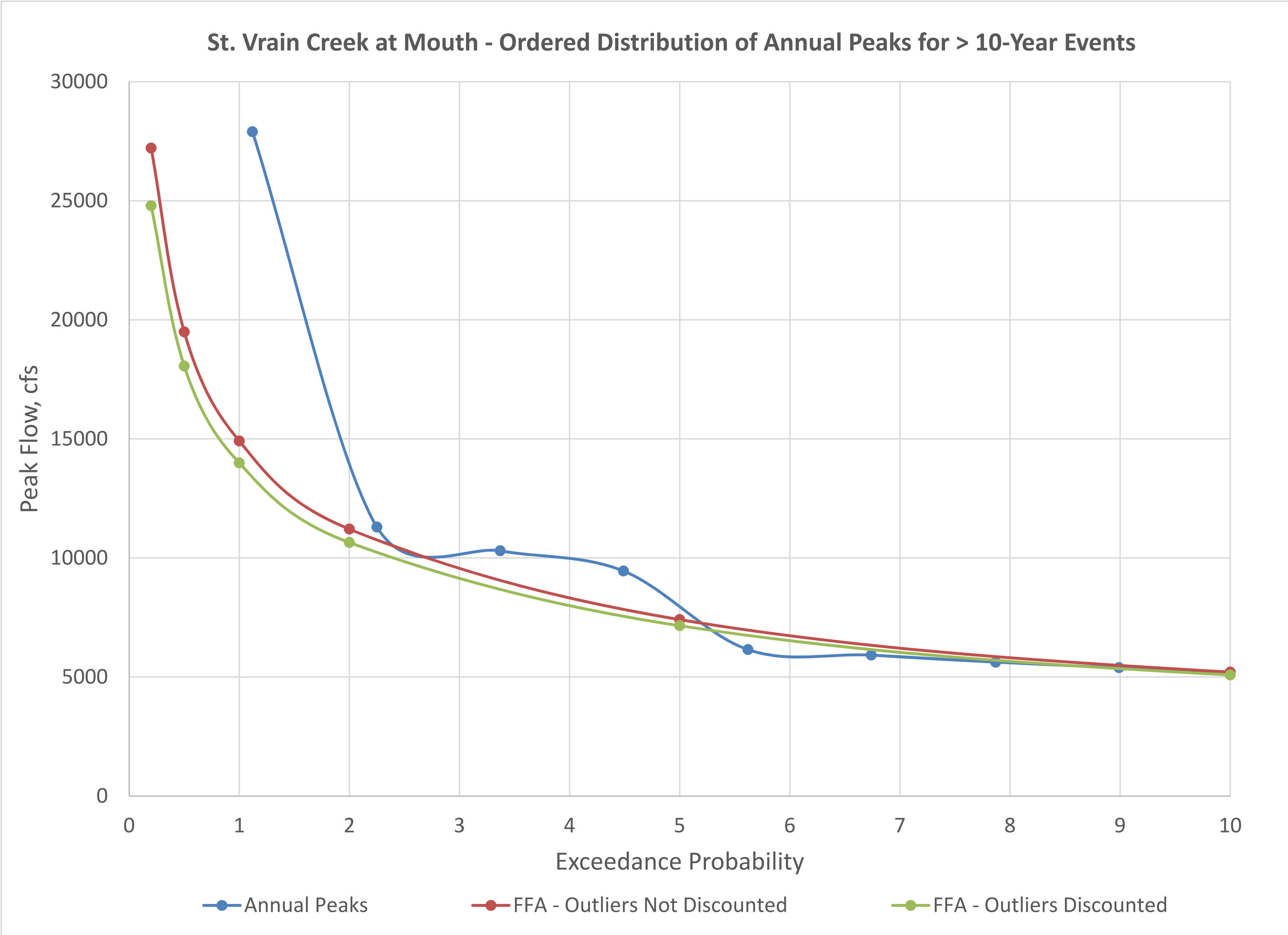
<< Adjusted Statistics >>
ST. VRAIN CK-MO, 2013

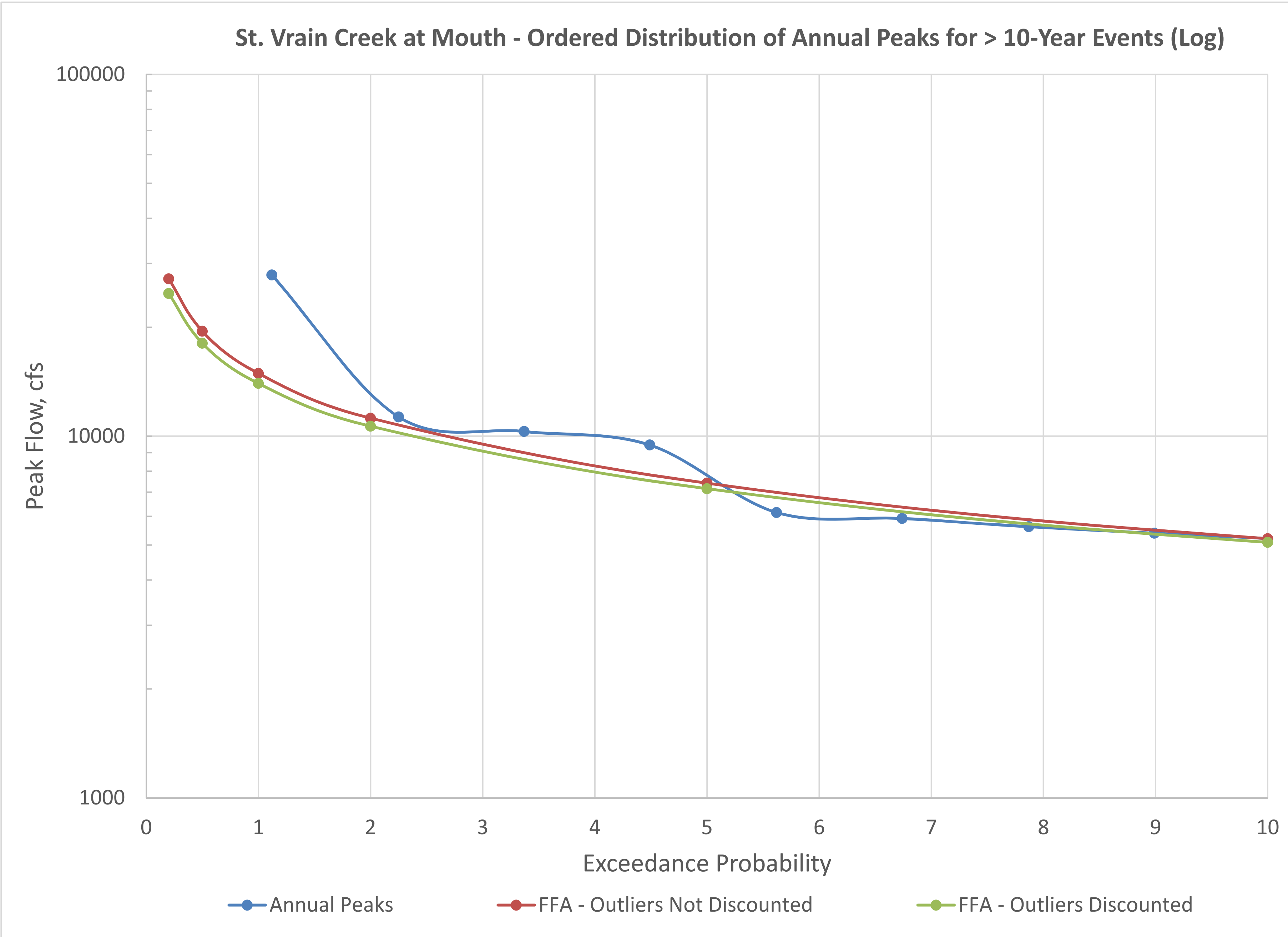
Log Transform: FLOW, CFS		Number of Events	
Mean	3.238	Historic Events	0
Standard Dev	0.358	High Outliers	1
Station Skew	0.290	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.290	Systematic Events	88
		Historic Period	109

--- End of Analytical Frequency Curve ---


Bulletin 17B Plot for 06731000 St. Vrain Ck, Mouth
Return Period
1.0
1.1
2
5
10
50
100
200
500
1000
10000
100000.0
10000.0
1000.0
100.0
Flow (cfs)
0.9999
0.999
0.99
0.9
0.5
0.2
0.1
0.02
0.01
0.005
0.002
0.001
0.0001
Probability
Computed Curve
Expected Probability Curve
5 Percent Confidence Limit
95 Percent Confidence Limit
Observed Events (Weibull plotting positions)
High Outlier


St. Vrain Creek at Mouth - Ordered Distribution of Annual Peaks
Peak Flow, cfs
30000
25000
20000
15000
10000
5000
0
0
10
20
30
40
50
60
70
80
90
100
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted


St. Vrain Creek at Mouth - Ordered Distribution of Annual Peaks for > 10-Year Events
Peak Flow, cfs
30000
25000
20000
15000
10000
5000
0
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted



St. Vrain Creek at Mouth - Ordered Distribution of Annual Peaks for > 10-Year Events (Log)
100000
10000
1000
Peak Flow, cfs
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted

Bulletin 17B Frequency Analysis

Lefthand Creek at Mouth near Longmont

USGS Gage 06725000

CDWR Gage LEFTLOCO

1927 – 1955 (broken) 19 years

Bulletin 17B Frequency Analysis
12 Jan 2015 02:24 PM

--- Input Data ---

Analysis Name: 06725000 Left Hand Ck, Mouth
Description: Copy of Downloaded from USGS website. Station 06725000 + 2013

Data Set Name: LEFT HAND CREEK-MOUTH, 2013
DSS File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\2BT_3St.V_1Lt.H..dss
DSS Pathname: /LEFT HAND CREEK/MOUTH, AT LONGMONT, CO./FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/Save Data As: LEFT HAND CREEK-MOUTH, 2013/

Report File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\06725000_Left_Hand_Ck,_Mouth\06725000_Left_Hand_Ck,_Mouth.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\06725000_Left_Hand_Ck,_Mouth\06725000_Left_Hand_Ck,_Mouth.xml

Start Date:
End Date:

Skew Option: Use Station Skew
Regional Skew: -Infinity
Regional Skew MSE: -Infinity

Plotting Position Type: Weibull

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95
Use High Outlier Threshold
High Outlier Threshold: 3249.4

Use Historic Data
Historic Period Start Year: ---
Historic Period End Year: ---

Display ordinate values using 0 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

<< Plotting Positions >>
LEFT HAND CREEK-MOUTH, 2013

Events Analyzed				Ordered Events			
			FLOW		Water	FLOW	Weibull
Day	Mon	Year	CFS	Rank	Year	CFS	Plot Pos
28	Jul	1927	113	1	2013	5,000*	5.00
10	May	1928	252	2	1938	812	10.00
06	Aug	1929	257	3	1942	369	15.00
13	Aug	1930	252	4	1933	355	20.00
06	Jun	1931	69	5	1941	321	25.00
18	Jun	1932	69	6	1929	257	30.00
19	May	1933	355	7	1930	252	35.00
05	May	1934	106	8	1928	252	40.00
18	May	1935	228	9	1935	228	45.00
10	Jun	1936	161	10	1937	192	50.00
03	Jun	1937	192	11	1936	161	55.00
02	Sep	1938	812	12	1955	150	60.00
01	May	1939	66	13	1927	113	65.00
28	Sep	1940	36	14	1934	106	70.00
22	Jun	1941	321	15	1932	69	75.00
19	Apr	1942	369	16	1931	69	80.00
14	Jul	1954	15	17	1939	66	85.00
19	Aug	1955	150	18	1940	36	90.00

13 Sep 2013	5,000	19	1954	15	95.00

* Outlier

<< Skew Weighting >>

Based on 19 events, mean-square error of station skew = 0.321
Mean-square error of regional skew = -?

<< Frequency Curve >>
LEFT HAND CREEK-MOUTH, 2013

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
16,145	47,411	0.2	96,312	5,628
8,834	18,866	0.5	42,474	3,464
5,486	9,622	1.0	22,307	2,355
3,332	4,960	2.0	11,407	1,566
1,645	2,064	5.0	4,457	871
916	1,047	10.0	2,072	528
474	505	20.0	901	292
156	156	50.0	251	95
62	60	80.0	101	32
41	38	90.0	70	19
30	27	95.0	54	13
18	15	99.0	35	7

<< Systematic Statistics >>
LEFT HAND CREEK-MOUTH, 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.251	Historic Events	0
Standard Dev	0.534	High Outliers	0
Station Skew	0.647	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.647	Systematic Events	19

--- End of Preliminary Results ---

<< High Outlier Test >>

Based on 19 events, 10 percent outlier test deviate K(N) = 2.361
Computed high outlier test value = 3,249.4

1 high outlier(s) identified above input threshold of 3,249.4

* *
* Note - Collection of historical information and *
* comparison with similar data should be explored, *
* if not incorporated in this analysis. *
* *

Statistics and frequency curve adjusted for 1 high outlier(s)

<< Systematic Statistics >>
LEFT HAND CREEK-MOUTH, 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.189	Historic Events	0
Standard Dev	0.435	High Outliers	1
Station Skew	-0.116	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.647	Systematic Events	19
		Historic Period	87

<< Low Outlier Test >>

Based on 87 events, 10 percent outlier test deviate K(N) = 2.97
Computed low outlier test value = 7.9

0 low outlier(s) identified below test value of 7.9

--- Final Results ---

<< Plotting Positions >>
LEFT HAND CREEK-MOUTH, 2013

Events Analyzed Day Mon Year	FLOW CFS	Ordered Events Rank	Water Year	FLOW CFS	Weibull Plot Pos
28 Jul 1927	113	1	2013	5,000*	1.14
10 May 1928	252	2	1938	812	4.42
06 Aug 1929	257	3	1942	369	9.85
13 Aug 1930	252	4	1933	355	15.28
06 Jun 1931	69	5	1941	321	20.71
18 Jun 1932	69	6	1929	257	26.14
19 May 1933	355	7	1930	252	31.57
05 May 1934	106	8	1928	252	36.99
18 May 1935	228	9	1935	228	42.42
10 Jun 1936	161	10	1937	192	47.85
03 Jun 1937	192	11	1936	161	53.28
02 Sep 1938	812	12	1955	150	58.71
01 May 1939	66	13	1927	113	64.14
28 Sep 1940	36	14	1934	106	69.57
22 Jun 1941	321	15	1932	69	75.00
19 Apr 1942	369	16	1931	69	80.43
14 Jul 1954	15	17	1939	66	85.86
19 Aug 1955	150	18	1940	36	91.29
13 Sep 2013	5,000	19	1954	15	96.72

Note: Plotting positions based on historic period (H) = 87
Number of historic events plus high outliers (Z) = 1
Weighting factor for systematic events (W) = 4.7778

* Outlier

<< Skew Weighting >>

Based on 87 events, mean-square error of station skew = 0.067
Mean-square error of regional skew = -?

<< Frequency Curve >>
LEFT HAND CREEK-MOUTH, 2013

Computed Curve	Expected Probability	Percent Chance	Confidence Limits 0.05	0.95

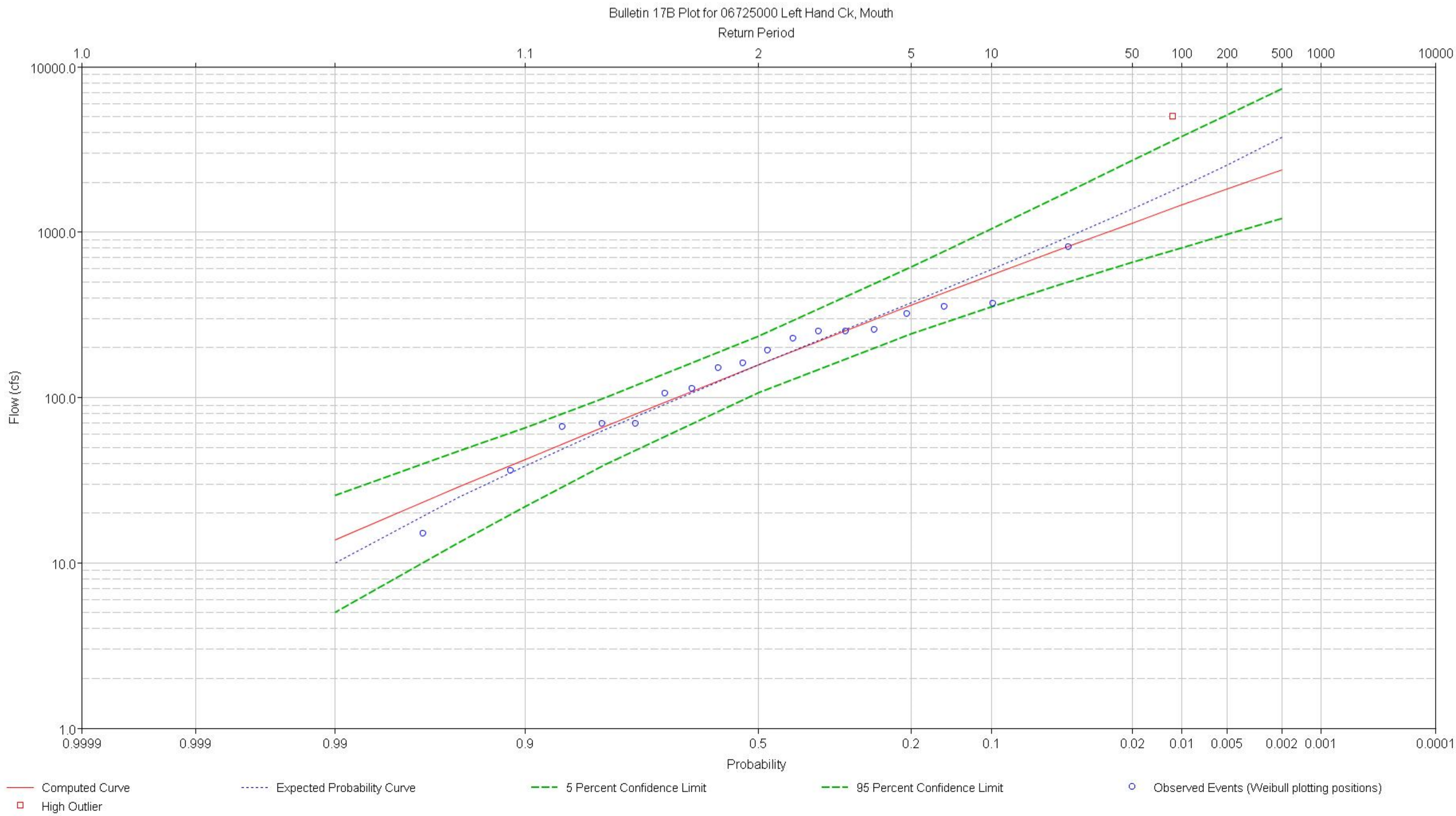
FLOW, CFS		Exceedance	FLOW, CFS	
2,390	3,731	0.2	7,361	1,212
1,823	2,555	0.5	5,111	971
1,454	1,894	1.0	3,775	806
1,132	1,382	2.0	2,706	655
774	878	5.0	1,638	475
549	594	10.0	1,049	354
360	375	20.0	615	242
157	157	50.0	233	106
67	64	80.0	99	39
42	39	90.0	66	22
29	25	95.0	47	13
14	10	99.0	26	5

<< Adjusted Statistics >>

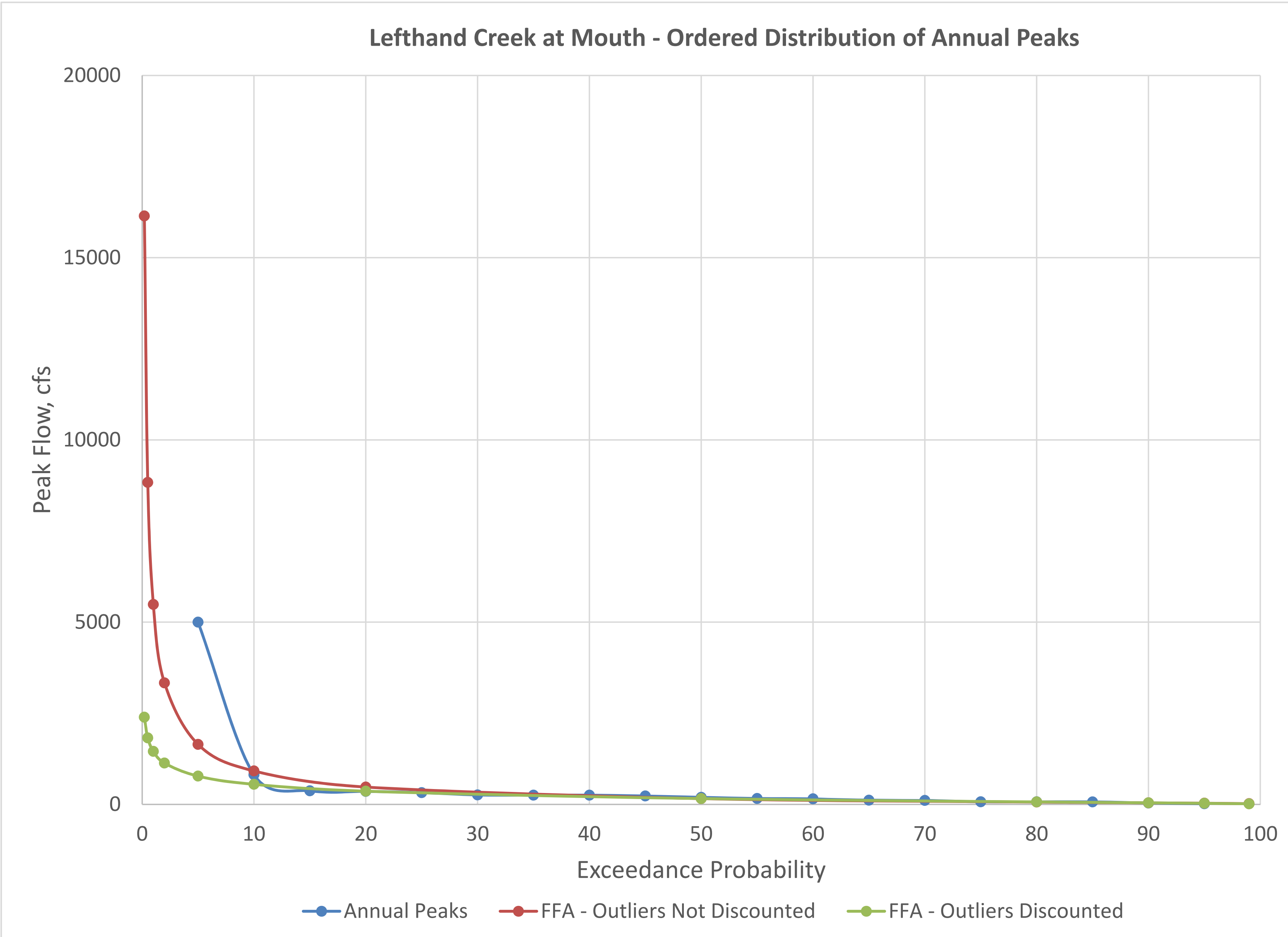
LEFT HAND CREEK-MOUTH, 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.189	Historic Events	0
Standard Dev	0.435	High Outliers	1
Station Skew	-0.116	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	-0.116	Systematic Events	19
		Historic Period	87

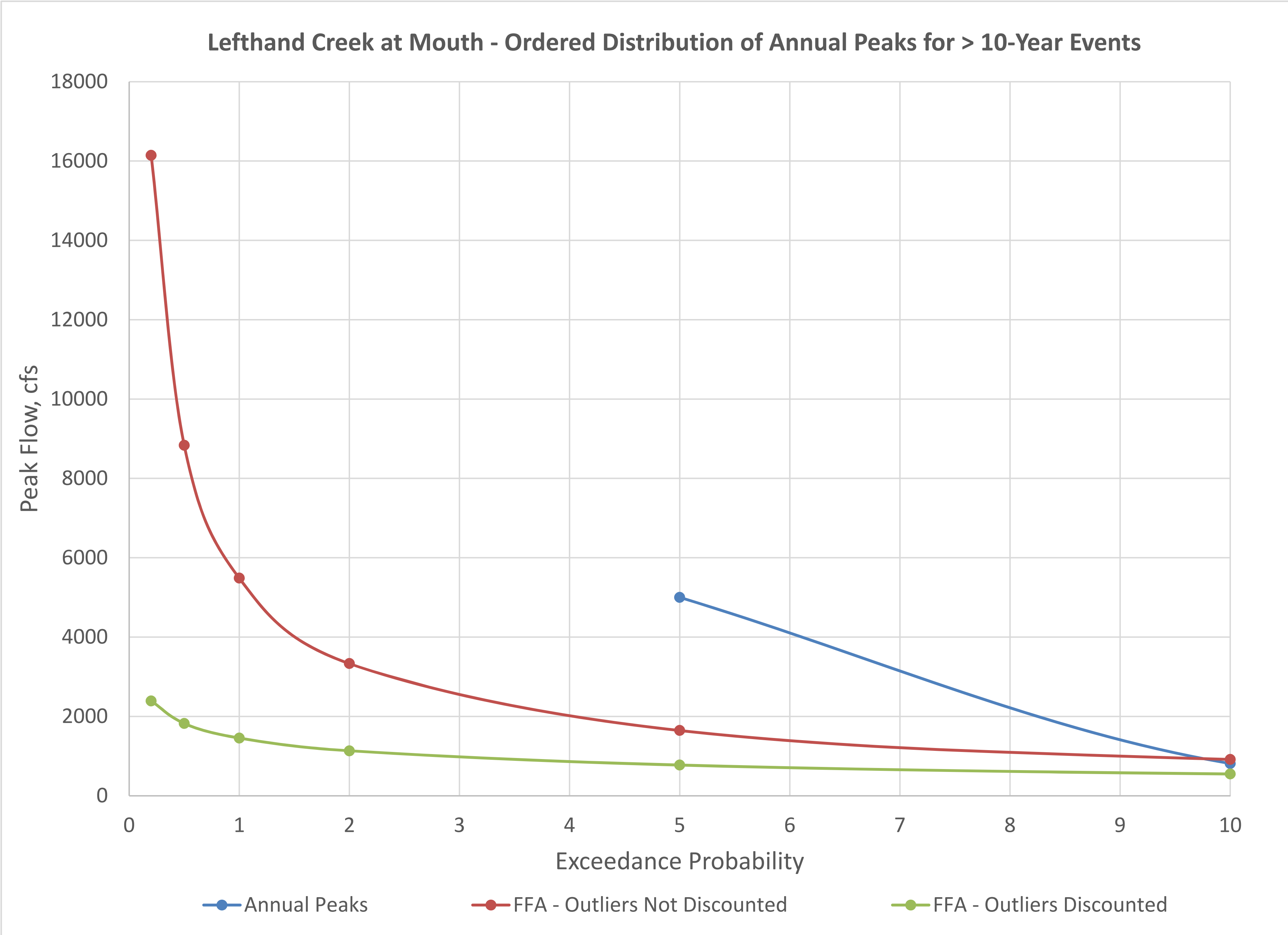
--- End of Analytical Frequency Curve ---



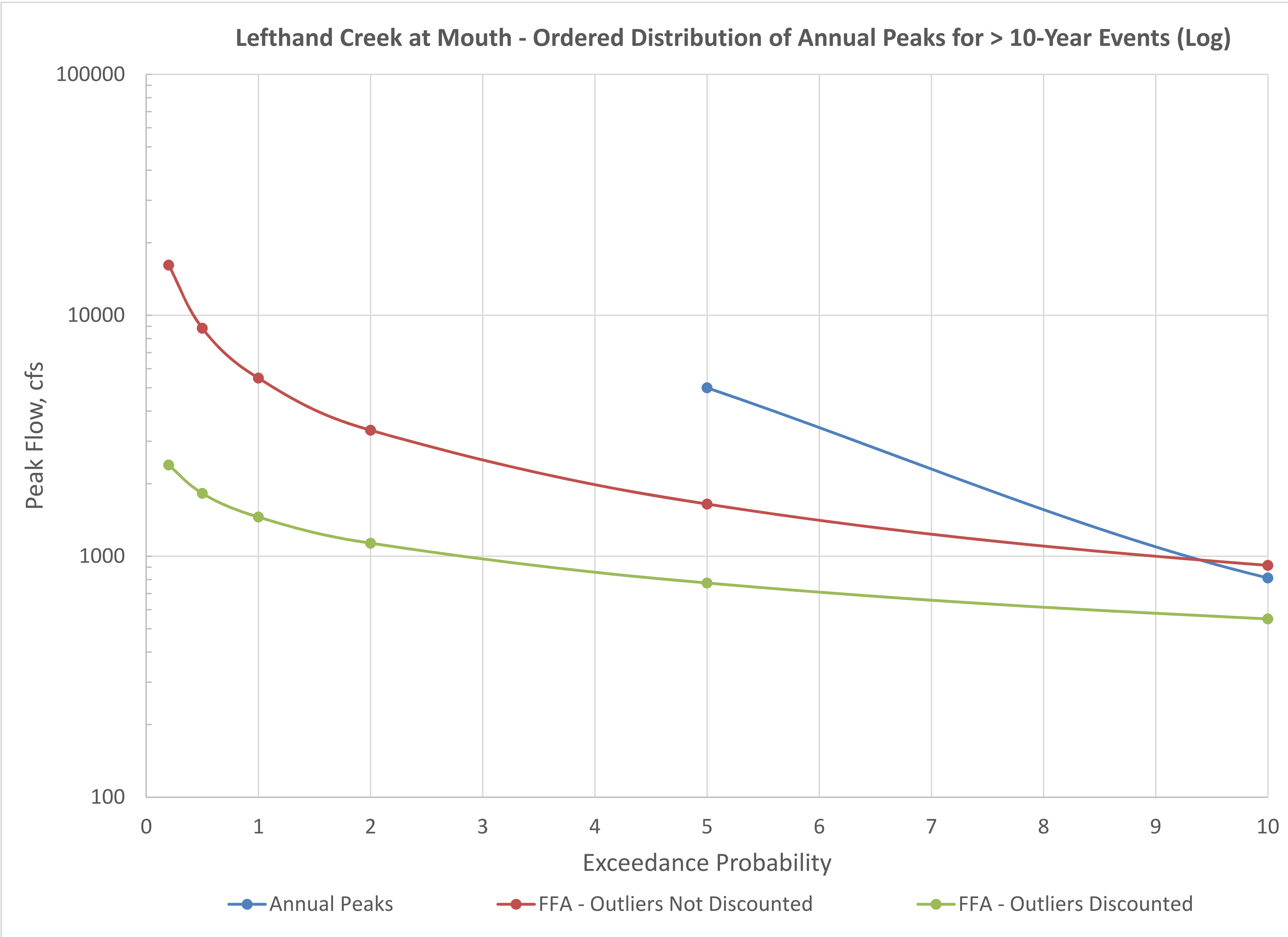
Bulletin 17B Plot for 06725000 Left Hand Ck, Mouth
Return Period
1.0
1.1
2
5
10
50
100
200
500
1000
10000
10000.0
1000.0
100.0
10.0
1.0
Flow (cfs)
0.9999
0.999
0.99
0.9
0.5
0.2
0.1
0.02
0.01
0.005
0.002
0.001
0.0001
Probability
Computed Curve
Expected Probability Curve
5 Percent Confidence Limit
95 Percent Confidence Limit
Observed Events (Weibull plotting positions)
High Outlier



Lefthand Creek at Mouth - Ordered Distribution of Annual Peaks
Peak Flow, cfs
20000
15000
10000
5000
0
0
10
20
30
40
50
60
70
80
90
100
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted



Lefthand Creek at Mouth - Ordered Distribution of Annual Peaks for > 10-Year Events
Peak Flow, cfs
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted


Lefthand Creek at Mouth - Ordered Distribution of Annual Peaks for > 10-Year Events (Log)
100000
10000
1000
100
Peak Flow, cfs
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted

Appendix C

Rainfall Depth-Area Reduction Factors

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February 20, 2015

Memo for Record

To: CDOT Flood Hydrology Committee

Subject: Colorado Front Range 24-hr Rainfall Areal Reduction Factors

1. Overview

The Colorado Department of Transportation (CDOT) Flood Hydrology Committee tasked Applied Weather Associates (AWA) to derive 24-hour areal reduction factors (ARFs) for the Front Range of Colorado for area sizes of 1- to 1000-sqmi. In addition, basin specific ARFs for the September 2013 rainfall event were calculated for four basins (Boulder Creek, St Vrain, Big Thompson, and Thompson).

2. Introduction

Information about extreme precipitation is of interest for a variety of purposes, which include meteorological and hydrologic engineering applications such as dam design, river management, and rainfall-runoff-relations. These entail knowledge on the spatial and temporal variability of precipitation over an area. In order to obtain areal average values for an area, point rainfall amounts are transformed to average rainfall amounts over a specified area. These issues are addressed using depth-area curves which require the use of ARFs. The derivation of ARFs is an important topic that has been dealt with using several methodologies.

The National Ocean and Atmospheric Administration (NOAA) defines an ARF as the ratio between area-averaged rainfall to the maximum depth at the storm center (NOAA Atlas 2, 1973). The most common sources for generalized ARFs and depth-area curves in the United States are from the NOAA Atlas 2 (NOAA Atlas 2, 1973) (Figure 1), and the U.S. Weather Bureau's Technical Paper 29 (U.S. Weather Bureau, 1957-60). Examples of site specific ARFs and depth-area curves are referenced in the NOAA Technical Report 24 (Meyers and Zehr, 1980) for the semi-arid southwest, the NOAA Technical Memorandum Hydro- 40 (NOAA Hydro-40, 1980) for the semi-arid southwest, and the city of Las Vegas, Nevada (Gou, 2011).

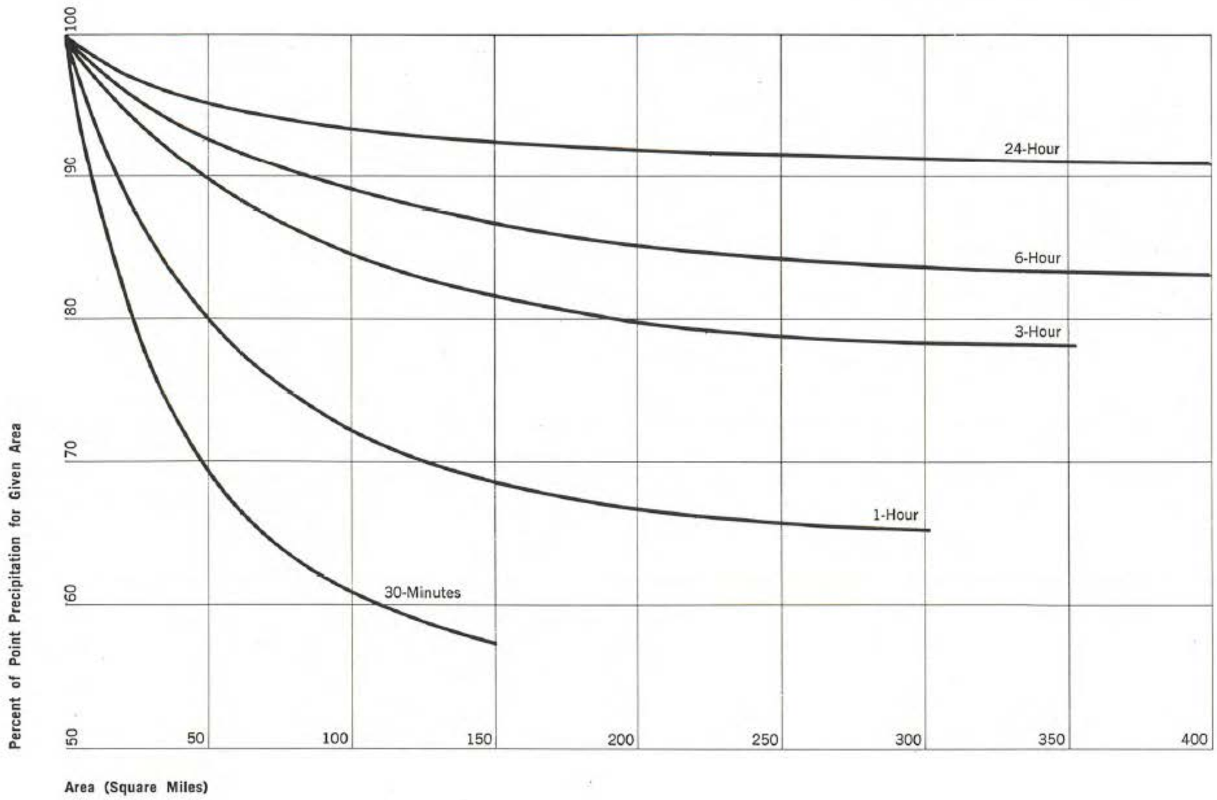

Figure 1: NOAA Atlas 2 Volume 3 ARF curves

There are two common methods for deriving ARFs: geographically fixed and storm centered. Geographically fixed ARFs originate from rainfall statistics, whereas storm centered ARF values are based on discrete rainfall events. Geographically fixed ARFs relate the precipitation depth at a point to a fixed area. The representative point is the mean of annual maximum point rainfall values at gauged points located within the network (U.S. Weather Bureau, 1957-60; NOAA Atlas 2, 1973; Osborn et al., 1980). This is a hypothetical point rather than a point for a particular location. The areas within the network are known beforehand and are both fixed in time and space (U.S. Weather Bureau, 1957-60; Osborn et al., 1980). With geographically fixed ARFs, the storm center does not correspond with the center of the location and does not need to fall within the area at all (Omolayo, 1993). Geographically fixed ARFs are based on different parts of different storms instead of the maximum point values located at the representative storm centers. A geographically fixed ARF is calculated as:

$$ARF_{Fixed} = \frac{\frac{1}{n}\sum_{j=1}^{n}\hat{R}_j}{\frac{1}{k}\sum_{i=1}^{k}\left(\frac{1}{n}\sum_{j=1}^{n}R_{ij}\right)},$$

where $\hat{R}_j$ is the annual maximum areal rainfall for year *j*, R_{ij} is the annual maximum point rainfall for year *j* at station *i*, *k* is the number of stations in the area, and *n* is the number of years.

The storm centered ARF does not have a fixed area in which rain falls but changes dynamically with each storm event (NOAA Atlas 2, 1973; Gou, 2011). Instead of the representative point being an average, the representative point is the center of the storm, defined as the point of maximum rainfall. Storm centered ARFs are calculated as the ratio of areal storm rainfall enclosed between isohyets equal to or greater than the isohyet value to the maximum point rainfall at the storm center. A storm centered ARF is calculated as:

$$ARF_{center} = \frac{\overline{R}_i}{R_{center}},$$

where $\overline{R}_i$ is the areal storm rainfall enclosed between isohyets equal to or greater than the isohyets, and R_{center} is the maximum point rainfall at the storm center.

3. Methods

AWA calculated ARFs use a storm centered depth-area approach based on gridded hourly rainfall data from the Storm Precipitation Analysis System (SPAS). SPAS has demonstrated reliability in producing highly accurate, high resolution rainfall analyses during hundreds of post-storm precipitation analyses (Tomlinson and Parzybok, 2004; Parzybok and Tomlinson, 2006). SPAS has evolved into a hydrometeorological tool that provides accurate precipitation data at a high spatial and temporal resolution for use in a variety of sensitive hydrologic applications. AWA and METSTAT, Inc. initially developed SPAS in 2002 for use in producing storm centered Depth-Area-Duration (DAD) values for Probable Maximum Precipitation (PMP) analyses. SPAS utilizes precipitation gauge data, “basemaps” and radar data (when available) to produce gridded precipitation at time intervals as short as 5-minutes, at spatial scales as fine as 1-km^2 and in a variety of customizable formats. To date, (December 2014) SPAS has analyzed over four-hundred storm centers across all types of terrain, among highly varied meteorological settings and with some events occurring over 100-years ago. For more detailed discussions on SPAS and DAD calculations refer to (Tomlinson et al., 2003-2012, Kappel et al., 2012-2014).

4. September 2013 Basin ARFs

The Colorado September 8-17, 2013 rainfall event was analyzed using the SPAS (SPAS number 1302) for use in several PMP and hydrologic model calibration studies (Figure 2). The hourly gridded rainfall data, based on gauge adjusted radar data, were used to derive basin specific ARFs. Four basins (Table 1) located along the Colorado Front Range were used to derive the 24-hour basin specific ARFs. The SPAS DAD program was used to derive basin specific 24-hour depth-area values. The point maximum (1-mi^2) 24-hour rainfall (within each basin) was selected as the storm center. The maximum average basin 24-hour rainfall depth for standard area sizes (1-, 10-, 25-, 50-, 100-, 200-, 300-, 400-, and 500-mi^2) up to the basin total area were calculated. The point maximum and maximum areal averages depths were used to calculate the basin specific ARFs.

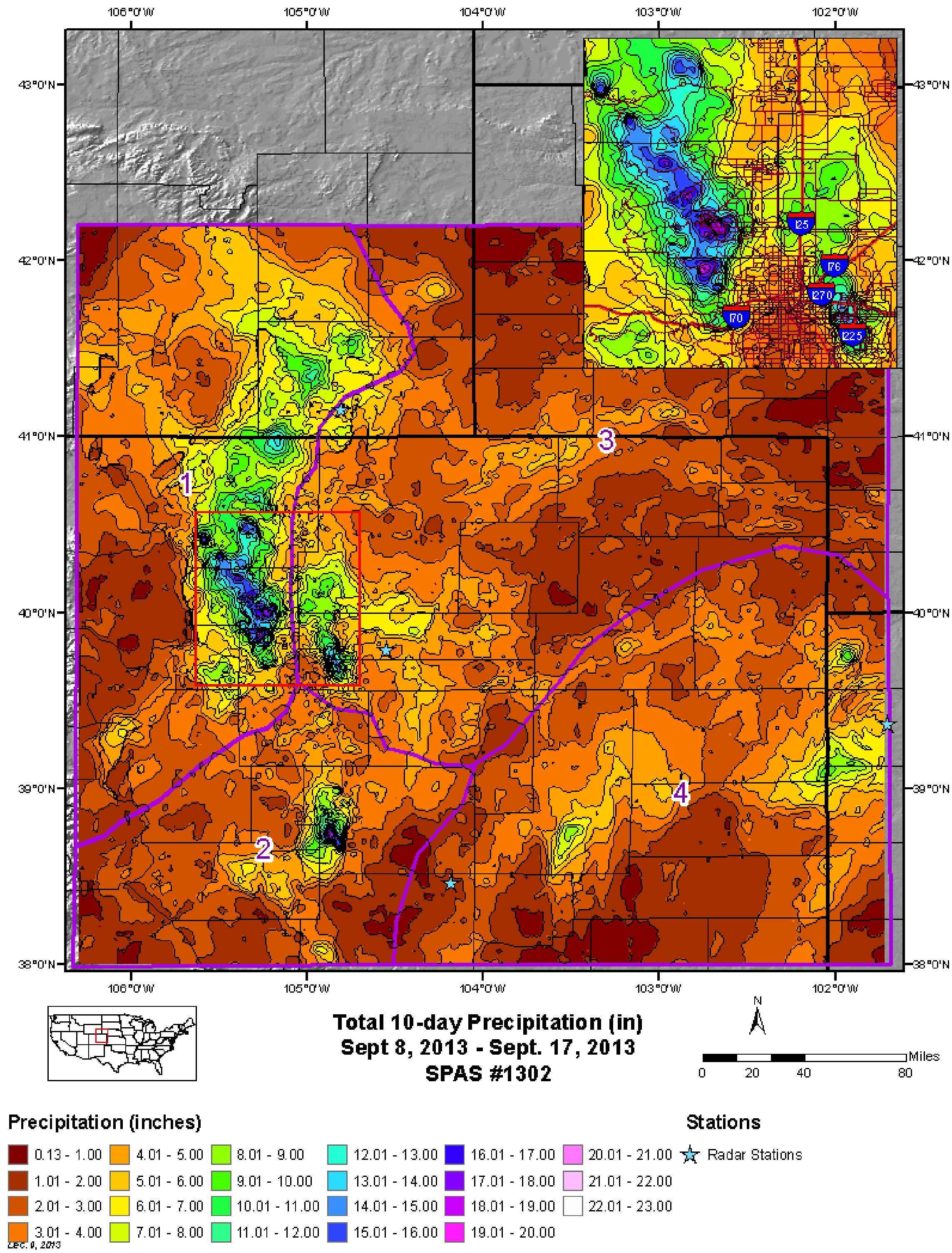

Figure 2: Basin specific ARFs for the September 2013 event compared to NOAA Atlas 2 ARF curve and to the HMR 55a Orographic C ARF

Table 1: Basin specific 24-hour ARFs for the September 2013 storm event

Basin	Area	ARF
Boulder Creek	446	0.352
St Vrain	982	0.384
Big Thompson	630	0.357
Thompson	827	0.355

The four calculated basin specific 24-hour ARFs for the September 2013 event were compared to NOAA Atlas 2 24-hour ARF curve and to the HMR 55A Orographic C 24-hour ARF curve (Hansen et al., 1988) (Figure 3). Table 1 shows the basin specific 24-hour ARF values. As expected, the four September 2013 basin ARF values have a significantly larger reduction in rainfall than published NOAA Atlas 2 and HMR 55A ARFs.

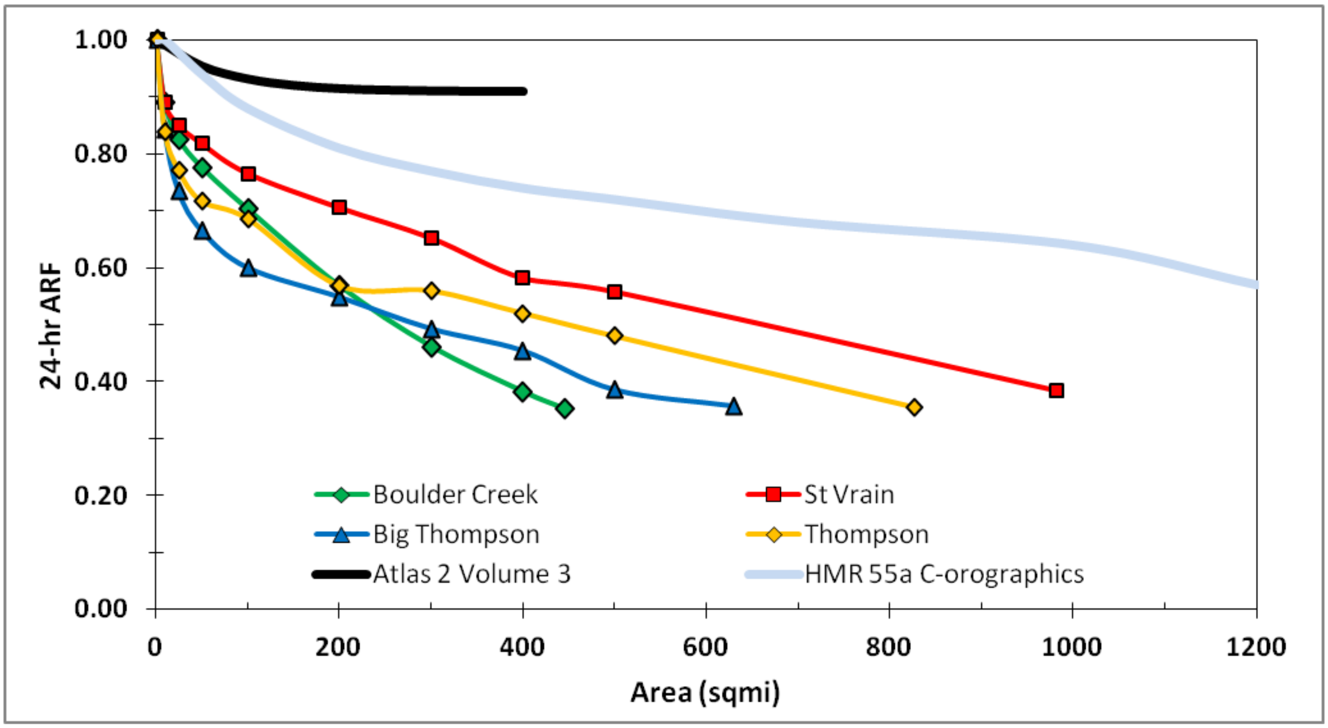

Figure 3: Basin specific 24-hour ARFs for the September 2013 event compared to NOAA Atlas 2 24-hour ARF curve and to the HMR 55A Orographic C 24-hour ARF curve

5. Colorado Front Range ARFs

Initially, twenty-eight SPAS storm center DAD zones were identified to have occurred over similar meteorological and topographic regions as the September 2013 storm event that occurred along the Colorado Front Range (Figure 4). The initial list was refined to nine storm centers that had storm characteristics representative of an upslope synoptic event similar to the four basins analyzed in this study. Storm events removed from the initial list were representative of shorter duration localized storm events or different topographic settings. The final set of nine storm centers (Table 2 and Figure 5) were used to derive 24-hour storm center ARFs.

The point maximum (1-mi^2) 24-hour rainfall (within each SPAS DAD zone) was selected as the storm center. The maximum average 24-hour rainfall depth for standard area sizes (1-, 10-, 25-, 50-, 100-, 150-, 200-, 250-, 300-, 350-, 400-, 450-, 500-, 700-, and 1000-mi^2) were calculated. The point maximum and maximum areal averages depths were used to calculate each events specific ARFs. Based on the nine events, an average ARF for each area size was calculated. Several other ARF curves were created for comparison purposes: maximum, minimum, +1-sigma, 85% confidence, 90% confidence, and 95% confidence. Based on discussions with the CDOT flood review committee and Nolan Doesken (Colorado State Climatologist), the 85% confidence ARF ($ARF_{85\%}$) was selected as the best representation of ARFs along the Colorado Front Range.

The final equation used to represent Colorado Front Range 24-hour ARFs is:

$$ARF_{85\%} = 0.646 + 0.354 * \exp(-kA)$$

where $ARF_{85\%}$ is the 85% confidence ARF, k is a decay coefficient, and A is storm area in square miles. The average ARF curve and final 85% confidence ARF curve are shown in Figure 6. The NOAA Atlas 2 ARF curve and HMR 55A Orographic C curve are also shown for comparison (Figure 6 and Table 3).

Table 2: Final SPAS storm centered locations with similar meteorology and topography as the September 2013 storm event used to derive 24-hr ARFs

ID	SPAS ID	Storm Location	Dates	Latitude	Longitude	Max Precipitation	HMR 55A CLASS	HMR 55A SUBUNIT
1	1211	Gibson Dam, WY	Jun. 6-8, 1964	48.3541	-113.3708	19.16	Orographic	Orographic "A"
2	1251	Lake Maloya, NM	May 17-21, 1955	37.0090	-104.3410	14.82	Orographic	Orographic "E"
3	1252	Waterton Red Rock, AB	June 14-21, 1975	49.0875	-114.0458	14.46	Orographic	Orographic "A"
4	1253	Big Elk Meadow, CO	May 3-8, 1969	40.2700	-105.4200	20.01	Orographic	Orographic "C"
5	1302	Northeast Colorado	Sep. 8-17, 2013	40.0150	-105.2650	20.41	Orographic	Orographic "C"
6	1320	Calgary, AB	Jun.19-22, 2013	50.6350	-114.8550	13.78	Orographic	Orographic "A"
7	1325	Savageton, WY	Sep. 27-Oct. 1, 1923	43.8458	-105.8042	17.56	Nonorographic	Min. Nonorographic "A"
8	1335	Warrick, MT	Jun. 5-10, 1906	48.0791	-109.7041	13.69	Orographic	Orographic "A"
9	1338	Spionkop Creek, AB	Jun. 4-7, 1995	49.1708	-114.1625	14.48	Orographic	Orographic "A"

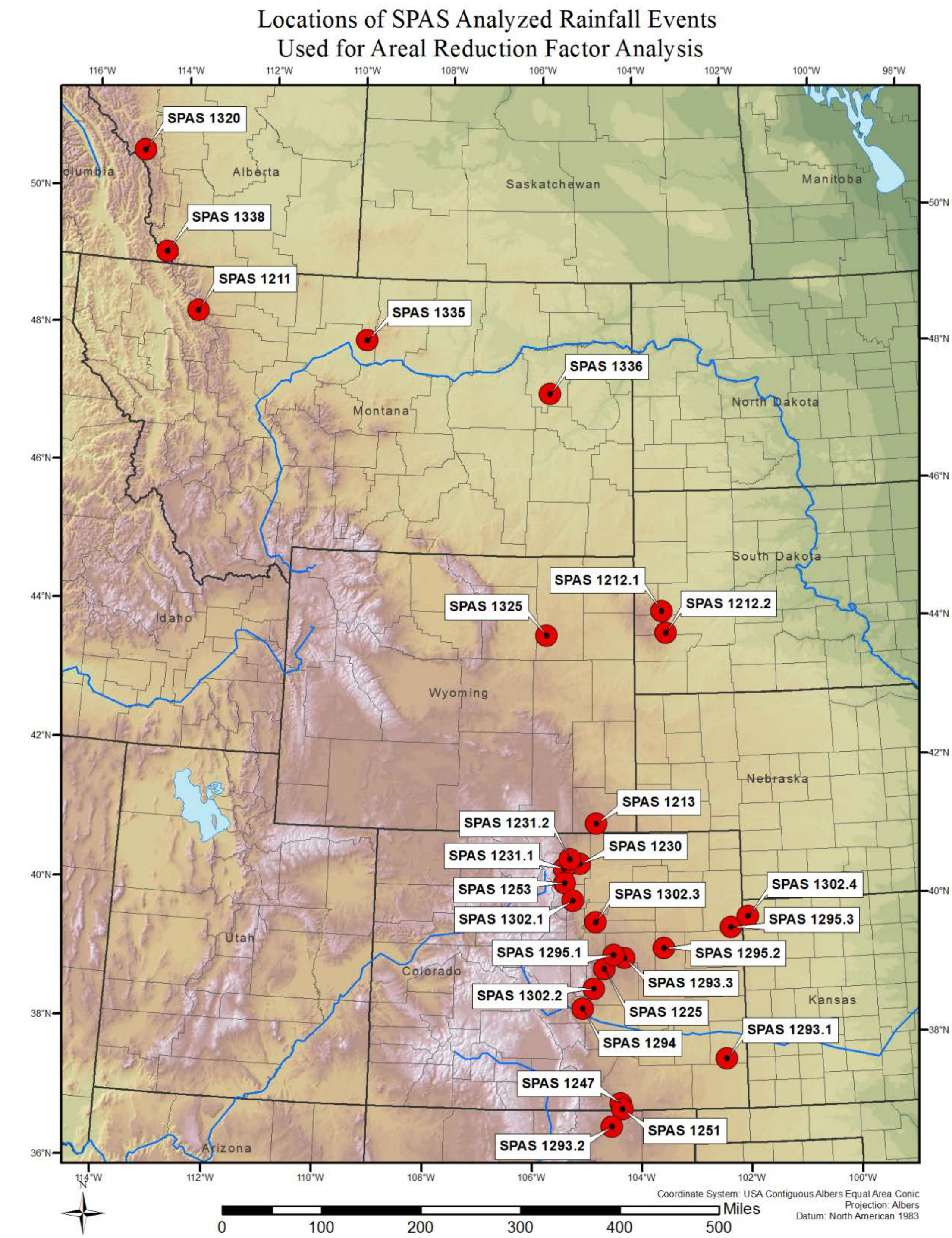

Figure 4: Initial twenty-eight SPAS storm center locations with similar meteorology and topography as the September 2013 storm event

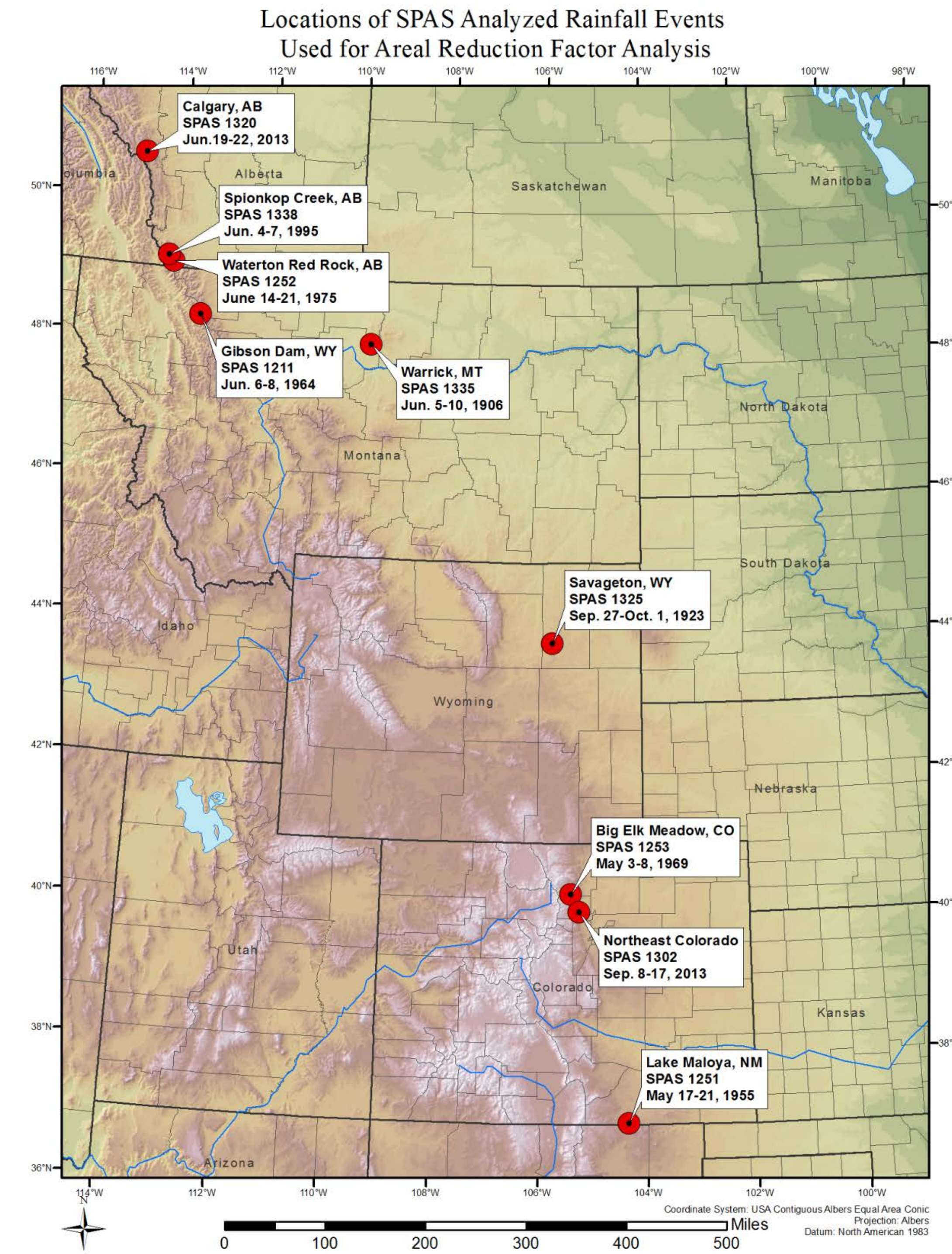

Figure 5: Final SPAS storm center locations used to derive 24-hr ARFs

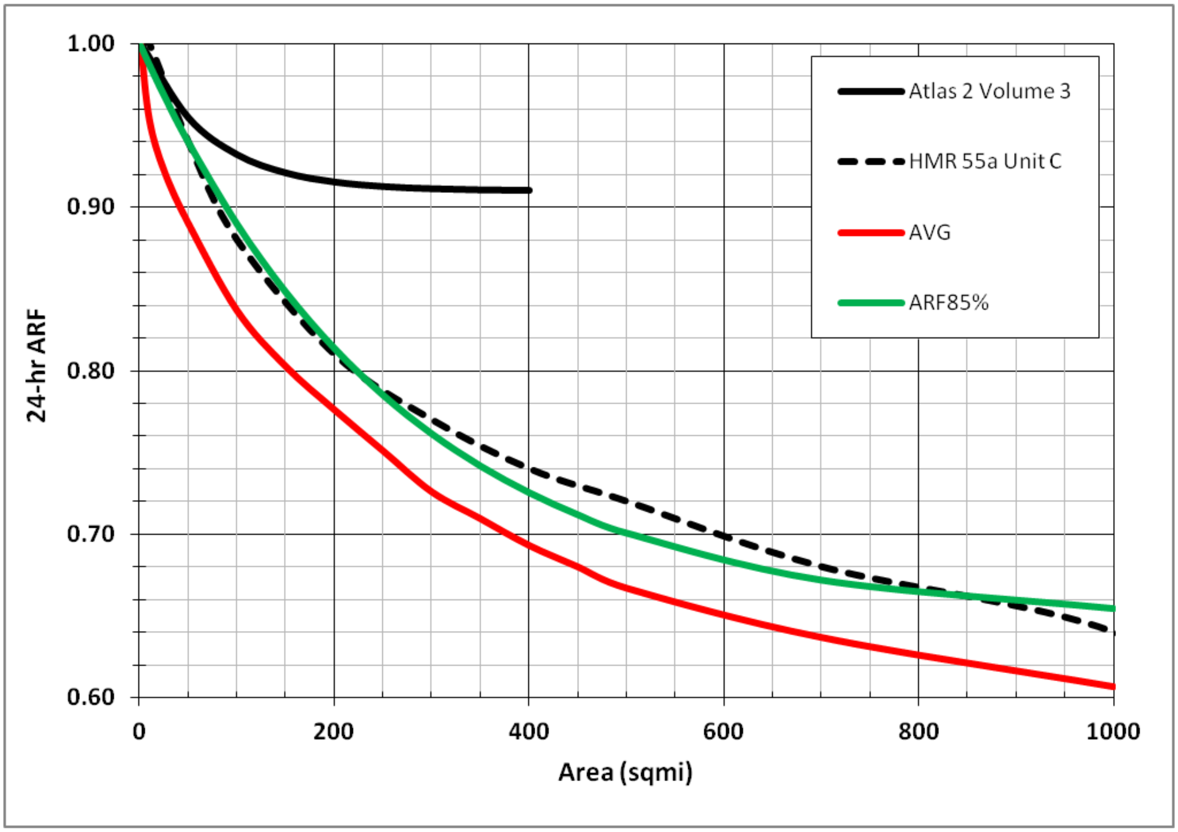

Figure 6: The average 24-hour ARF curve and final 85% confidence 24-hour ARF curve. The NOAA Atlas 2 24-hour ARF curve and HMR 55A Orographic C 24-hour ARF curve are shown for comparison.

Table 3: Comparison of 24-hour ARF values. AVG is the average ARF, ARF85% is the 85% confidence ARF, HMR 55A is HMR 55A Orographic C ARF, and Atlas 2 is NOAA Atlas 2 ARF.

*** General Storms 24-hr ARF				
Area (sqmi)	AVG	ARF85%	HMR 55a	Atlas 2
1	1.00	1.00	1.00	1.00
10	0.95	0.99	1.00	-
25	0.92	0.97	0.97	-
50	0.89	0.94	0.94	0.95
100	0.84	0.89	0.88	0.93
150	0.80	0.85	0.85	0.92
200	0.78	0.81	0.81	0.92
250	0.75	0.78	0.79	0.91
300	0.73	0.76	0.77	0.91
350	0.71	0.74	0.76	0.91
400	0.69	0.73	0.74	0.91
450	0.68	0.71	0.73	-
500	0.67	0.70	0.72	-
700	0.64	0.67	0.68	-
1000	0.61	0.65	0.64	-

6. Results

The final derived $ARF_{85\%}$ values created significantly larger reductions in point rainfall as compared to NOAA Atlas 2. In order to apply the new $ARF_{85\%}$ data, a transition between NOAA Atlas 2 and the final $ARF_{85\%}$ curve was created (CDOT flood review committee). A linear transition was applied between NOAA Atlas 2 315-mi^2 ARF value and $ARF_{85\%}$ 500-mi^2 (Figure 7 and Table 4). The final 24-hour $ARF_{85\%}$ curve is compared to the four basin specific 24-hour ARF curves for the September 2013 event (Figure 8).

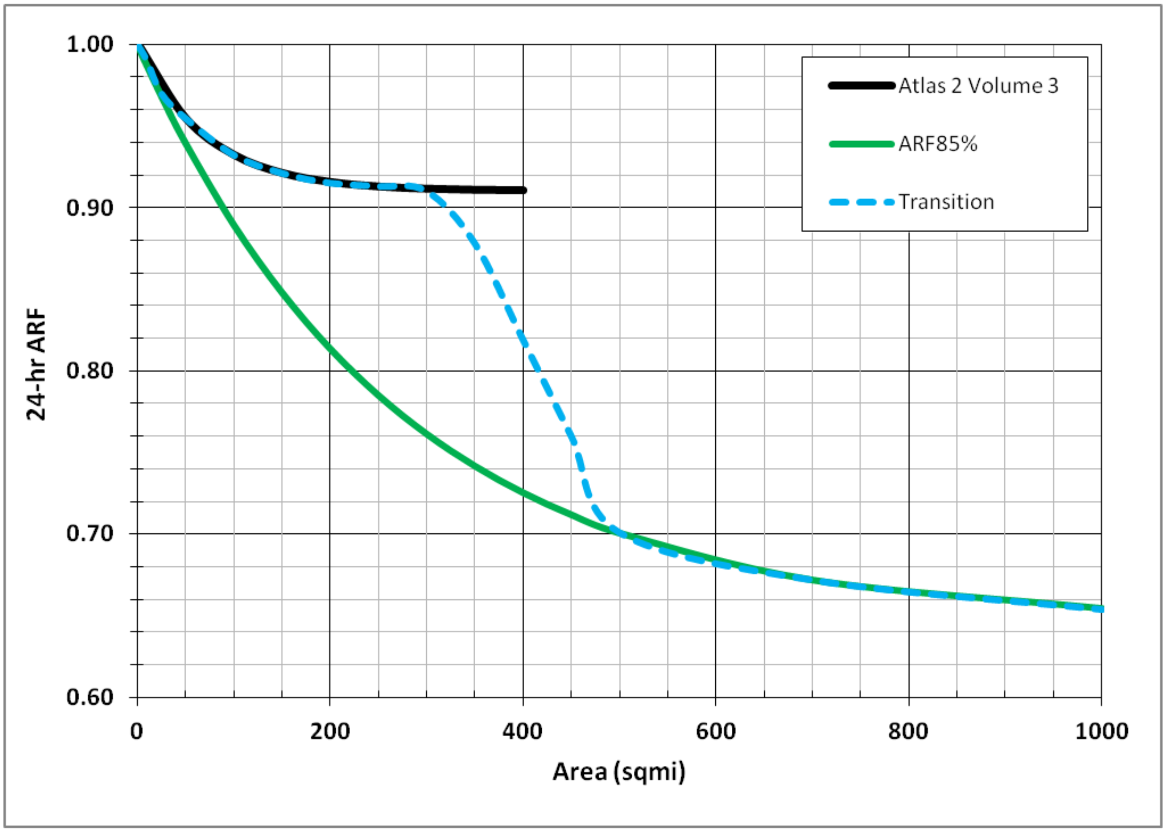

Figure 7: Final 24-hr ARF curve with transition between NOAA Atlas 2 and AWA $ARF_{85\%}$

Table 4: Comparison of final 24-hour ARF values. ARF85% is the 85% confidence ARF. Transition is the transition between NOAA Atlas 2 and ARF85%, and Atlas 2 is NOAA Atlas 2 ARF.

*** General Storms 24-hr ARF			
Area (sqmi)	**ARF85%**	**Transition**	**Atlas 2**
1	1.00	1.00	1.00
10	0.99	0.99	-
25	0.97	0.97	-
50	0.94	0.95	0.95
100	0.89	0.93	0.93
150	0.85	0.92	0.92
200	0.81	0.92	0.92
250	0.78	0.91	0.91
300	0.76	0.91	0.91
350	0.74	0.88	0.91
400	0.73	0.82	0.91
450	0.71	0.76	-
500	0.70	0.70	-
700	0.67	0.67	-
1000	0.65	0.65	-

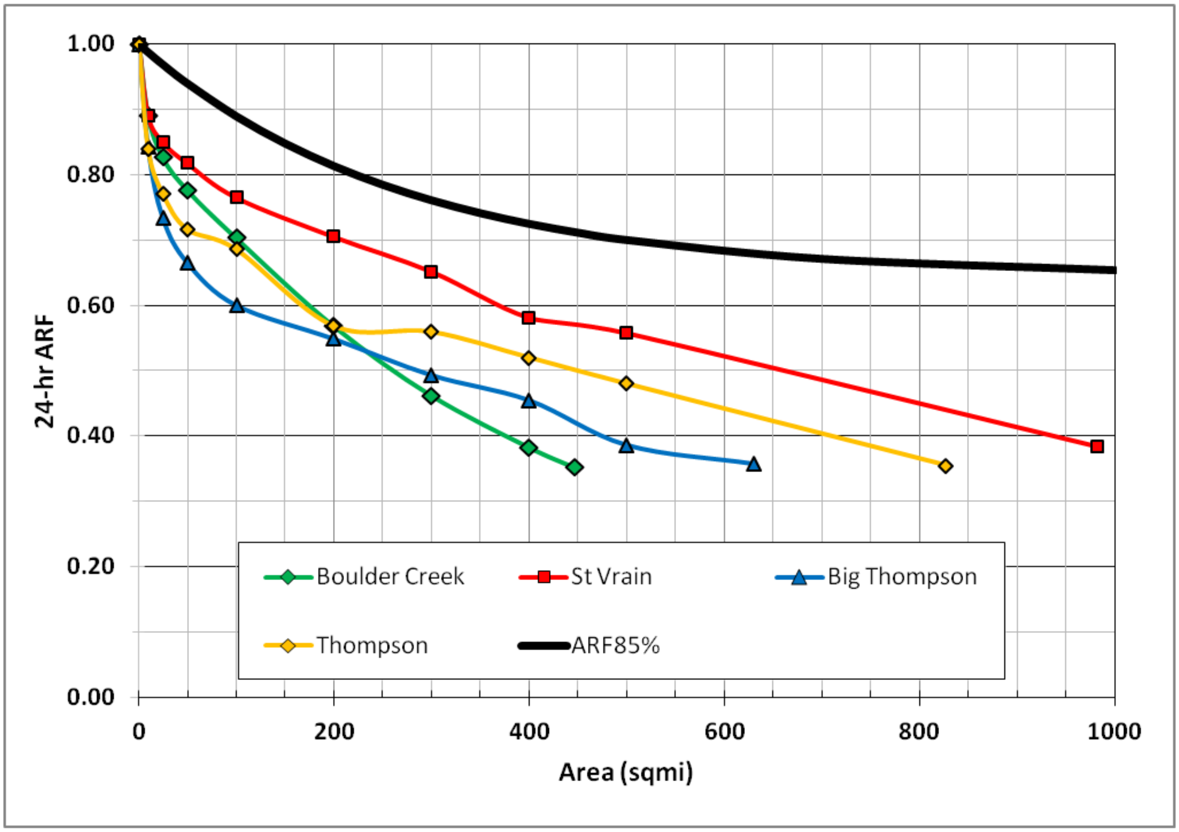

Figure 8: 24-hour ARF curve compared to basin specific ARFs for the September 2013 event

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Appendix D

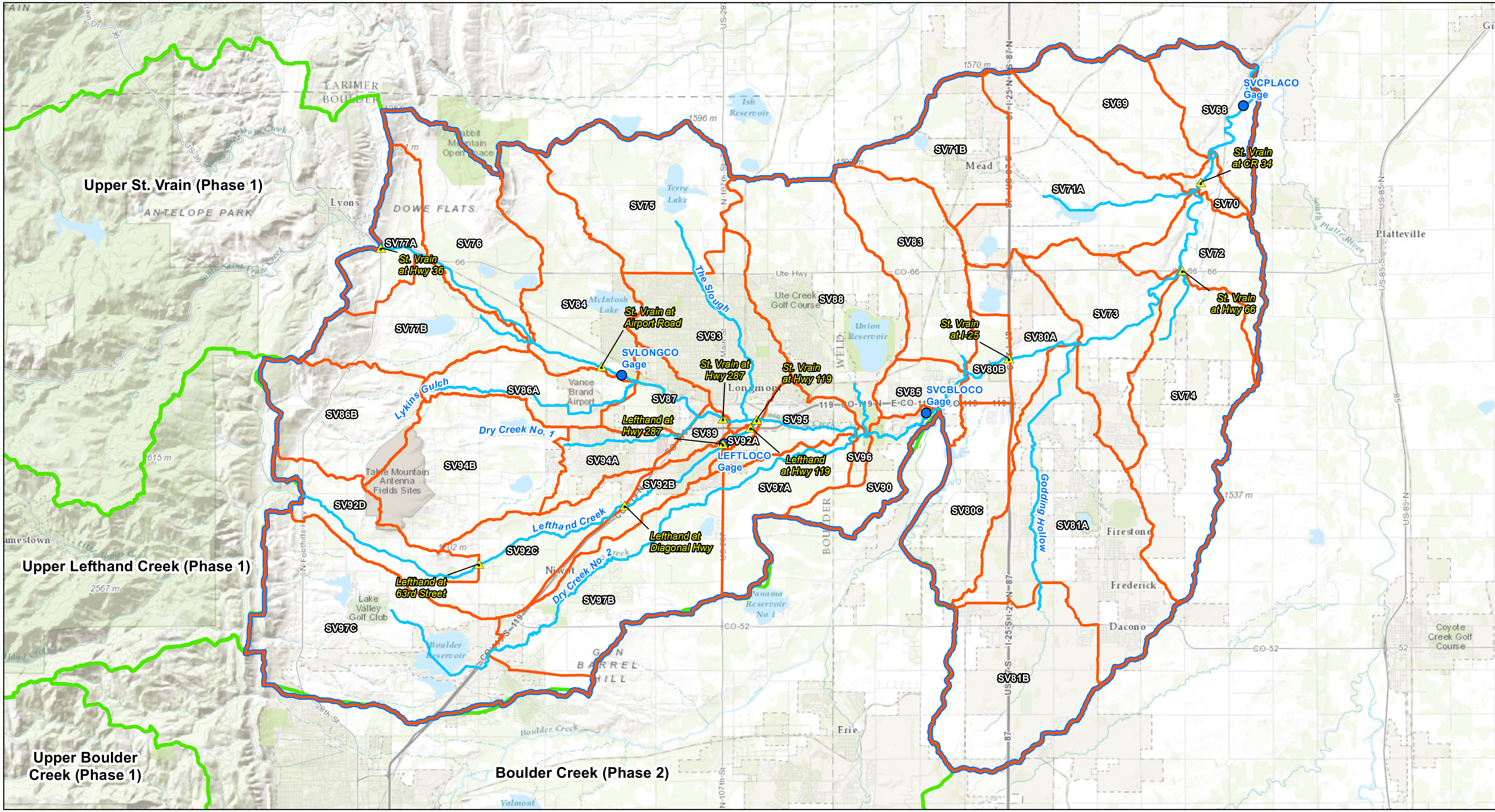
Rainfall/Runoff Modeling

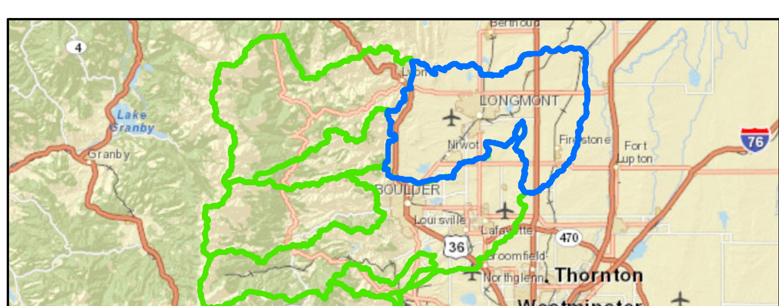
Appendix D.1

HEC-HMS Model Input

Figure D.1 - Lower St. Vrain Watershed (Phase 2)

HEC-HMS Model Input Parameters





Lower St. Vrain Watershed - Phase 2

Figure D.1

CDOT Flood Recovery Hydrologic Evaluation

Service Layer Credits: Sources: Esri, HERE, DeLorme, TomTom, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community
Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri

0 1 2
Miles

- Investigation Sites
- Stream Gage Station
- Tributaries
- Lower St. Vrain SubBasins
- Lower St. Vrain Watershed Boundary
- St. Vrain Watershed Boundary

Location	Estimated 2013 Peak Discharge	Tributary Area
St. Vrain at Hwy 36	23,000	218
St. Vrain at Airport Road	14,000	237
St. Vrain at Hwy 287	14,500	276
St. Vrain at Hwy 119	18,500	368
St. Vrain at I-25	23,500	889
St. Vrain at Hwy 66	23,000	942
St. Vrain at CR 34	27,000	965
Lefthand at 63rd Street	7,000	63
Lefthand at Diagonal Hwy	8,700	69
Lefthand at Hwy 287	5,000	72
Lefthand at Hwy 119	4,800	72

Appendix D.1

Lower St. Vrain Watershed - HEC-HMS Model Inputs

Model ID	Type	Description	Area (sq.mi.)	CN (10-day)	CN (24-hr)	Cp	Kn	L (mi)	Lc (mi)	S (ft/mi)	Lag Time (hr)	L (ft)	S (ft/ft)	n Channel	n Left OB	n Right OB	Loss (10-day)
Upper St. Vrain	Source	Upper SVC Model Inflow Hydrographs	218.39														
J78	Junction	SVC at Hwy 36	218.39														
R78	Reach	SVC below Hwy 36	218.39									5900	0.009	0.100	0.100	0.100	0.10
SV77A	Subbasin	SVC Area below Hwy 36	3.88	63.1	80.3	0.4	0.1	3.5	1.0	264.5	1.3						
J77	Junction	SVC below Union Road	222.27														
R70	Reach	SVC above 75th Street	222.27									18746	0.007	0.100	0.100	0.100	0.20
SV76	Subbasin	SVC Area north of Hygiene Road	8.37	64.0	83.0	0.4	0.1	7.6	3.8	174.4	2.8						
SV77B	Subbasin	Foothills Reservoir Area near Hygiene Road	6.75	62.5	79.9	0.4	0.1	6.4	3.3	231.7	2.5						
J49	Junction	SVC upstream of 75th Street	237.39														
R140	Reach	SVC above Golden Ponds (near Airport Rd)	237.39									14670	0.005	0.100	0.100	0.100	0.20
SV84	Subbasin	SVC Area above McIntosh Lake (Airport Road)	9.29	64.1	84.0	0.4	0.1	7.5	3.1	55.2	3.2						
SV86B	Subbasin	Lykins Gulch Headwater Area	7.50	64.6	80.2	0.4	0.1	5.5	2.7	276.8	2.1						
J86	Junction	Lykins Gulch at Boulder Feeder Canal	7.50														
R160	Reach	Lykins Gulch	7.50									33437	0.007	0.050	0.050	0.050	
SV86A	Subbasin	Lykins Gulch Area above SVC	4.65	76.3	87.8	0.4	0.1	5.3	2.4	40.7	2.8						
J28	Junction	Lykins Gulch at SVC Confluence	12.14														
J29	Junction	SVC at Golden Ponds (Lykins Ditch Confluence)	258.82														
R200	Reach	SVC above Dry Creek #1 Confluence	258.82									10492	0.004	0.100	0.100	0.100	0.10
SV87	Subbasin	SVC Area above Dry Creek #1 Confluence	2.75	78.2	91.3	0.4	0.1	3.1	1.6	32.5	2.1						
J22	Junction	SVC above Dry Creek #1 Confluence	261.57														
SV94B	Subbasin	Dry Creek #1 Headwaters	10.73	62.9	79.5	0.4	0.1	5.9	3.0	93.7	2.7						
J94B	Junction	Dry Creek #1 at Clover Basin Ditch	10.73														
R260	Reach	Dry Creek #1 above SVC	10.73									18837	0.004	0.050	0.050	0.050	
SV94A	Subbasin	Dry Creek #1 Area above SVC	2.98	80.6	90.5	0.4	0.1	4.5	3.0	38.4	2.8						
J23	Junction	Dry Creek #1 at SVC Confluence above BNSF Railroad	13.71														
J215	Junction	SVC at BNSF Railroad	275.28														
R205	Reach	SVC between BNSF Railroad and Hwy 287	275.28									4948	0.004	0.100	0.100	0.100	
SV89	Subbasin	SVC Area between BNSF Railroad and Hwy 287	0.95	79.4	91.8	0.4	0.1	2.1	1.2	36.2	1.7						
J21	Junction	SVC below Hwy 287 (Main St.) at Dry Creek #1 Old Channel	276.24														
R210	Reach	SVC below Hwy 287 (Main St.)	276.24									2635	0.004	0.100	0.100	0.100	0.05
SV75	Subbasin	Terry Lake Area (The Slough)	10.72	63.9	85.0	0.4	0.1	5.9	2.5	69.3	2.7						
J75	Junction	Spring Gulch (The Slough) at Ute Hwy 66	10.72														
R190	Reach	Spring Gulch (The Slough)	10.72									23381	0.005	0.050	0.050	0.050	
SV93	Subbasin	Spring Gulch Area (The Slough)	8.85	77.9	90.4	0.4	0.1	6.1	3.3	33.1	3.4						
J36	Junction	The Slough (Spring Gulch) at SVC Confluence	19.57														
J35	Junction	SVC at Martin Street (The Slough)	295.81														
R2120	Reach	SVC above LHC Confluence	295.81									1611	0.004	0.100	0.100	0.100	0.05
Upper Lefthand	Source	Upper LHC Model Inflow Hydrographs	58.12														
J92E	Junction	Lefthand Creek at Hwy 36	58.12														
R481	Reach	LHC downstream of Hwy 36	58.12									15090	0.016	0.150	0.150	0.150	0.30
SV92D	Subbasin	LHC area above 63rd Street	4.66	41.0	60.9	0.8	0.1	2.6	1.3	411.9	1.2						
J92D	Junction	LHC at 63rd Street	62.78														
R480	Reach	LHC upstream of Hwy 119	62.78									21613	0.003	0.150	0.150	0.150	0.18
SV92C	Subbasin	LHC Area upstream of Hwy 119	6.56	40.3	60.3	0.8	0.1	4.0	1.6	73.1	2.0						
J92C	Junction	LHC at Diagonal Highway (near Airport Rd)	69.34														
R380	Reach	LHC above Hwy 287 (Main St.)	69.34									16169	0.004	0.150	0.150	0.150	0.14
SV92B	Subbasin	LHC Area above Hwy 287 (Main St.)	2.35	48.4	67.8	0.8	0.1	1.9	1.6	34.9	1.8						
J92B	Junction	LHC at Hwy 287 (Main St.)	71.69														
R280	Reach	LHC above SVC Confluence	71.69									6276	0.003	0.150	0.150	0.150	0.05
SV92A	Subbasin	LHC Area above SVC Confluence	0.26	44.6	68.6	0.8	0.1	1.2	0.7	23.0	1.2						
J92A	Junction	LHC at SVC Confluence	71.94														
J24	Junction	SVC at Confluence with LHC	367.75														
R230	Reach	SVC between LHC and Dry Creek #2	367.75									13759	0.004	0.100	0.100	0.100	0.10
SV95	Subbasin	SVC Area between LHC and DC#2	2.97	75.2	88.3	0.4	0.1	4.5	2.1	18.5	2.9						
J14	Junction	SVC upstream of Dry Creek #2	370.72														
SV97C	Subbasin	DC#2 Area above Boulder Reservoir	18.16	58.9	78.9	0.4	0.1	8.4	3.9	118.3	3.2						
J97C	Junction	Dry Creek #2 at Hwy 119	18.16														
R510	Reach	Dry Creek #2 above Hwy 287	18.16									44225	0.004	0.050	0.050	0.050	
SV97B	Subbasin	Dry Creek #2 Area above Hwy 287	11.54	69.1	82.9	0.4	0.1	8.5	3.6	31.7	3.9						
J97B	Junction	Dry Creek #2 at Hwy 287	29.70														
R310	Reach	Dry Creek #2 above SVC Confluence	29.70									21825	0.003	0.050	0.050	0.050	
SV97A	Subbasin	Dry Creek #2 Area above SVC Confluence	5.11	69.4	83.1	0.4	0.1	6.0	4.7	25.0	3.9						
J15	Junction	Dry Creek No. 2 at SVC Confluence	34.81														
J13	Junction	SVC at Confluence with Dry Creek #2	405.53														
R240	Reach	SVC below Dry Creek #2	405.53									2305	0.002	0.100	0.100	0.100	
SV88	Subbasin	Area around Calkins Lake/Union Reservoir	14.20	70.5	87.2	0.4	0.1	7.9	4.1	33.2	3.9						
SV96	Subbasin	SVC Area south of channel near County Line	0.44	61.4	78.2	0.4	0.1	1.5	0.6	70.4	1.1						
J16	Junction	SVC at Spring Gulch (Union Reservoir Ditch)	420.17														
R250	Reach	SVC above BC Confluence	420.17									11900	0.002	0.100	0.100	0.100	0.10
SV90	Subbasin	SVC Area above BC Confluence	3.36	66.8	81.6	0.4	0.1	5.6	2.6	42.8	2.9						
J33	Junction	SV upstream of Boulder Creek	423.54														
Boulder Creek	Source	Boulder Creek Model Inflow Hydrographs	446.81														
J32	Junction	SVC at Confluence with Boulder Creek (Hwy 119)	870.35														
R170	Reach	SVC below BC Confluence	870.35									6610	0.002	0.100	0.100	0.100	0.05
SV85	Subbasin	SVC Area below BC Confluence	2.47	61.0	78.9	0.4	0.1	3.3	1.7	51.0	2.0						
SV83	Subbasin	Unnamed North Trib to SVC above I-25	7.84	54.1	76.3	0.4	0.1	6.4	3.2	60.4	3.1						
J85	Junction	SVC Upstream of I-25	880.66														
R105	Reach	SVC upstream of I-25	880.66									7168	0.002	0.100	0.100	0.100	0.05
SV80C	Subbasin	Idaho Creek Tributary Area	6.35	59.7	76.4	0.4	0.1	5.1	2.3	38.8	2.7						
J80C	Junction	Idaho Creek Trib. to SVC at I-25	6.35														
SV80B	Subbasin	SVC Area upstream of I-25	1.89	54.8	72.6	0.4	0.1	2.4	0.5	43.4	1.3						
J42	Junction	SVC at I-25	888.90														
R100	Reach	SVC below I-25	888.90									9558	0.002	0.050	0.050	0.050	
SV81B	Subbasin	Godding Hollow Headwaters	11.34	56.9	70.9	0.4	0.1	5.6	2.3	51.4	2.7						
J81B	Junction	Upper Godding Hollow Tributary	11.34														
R290	Reach	Godding Hollow	11.34									35193	0.004	0.050	0.050	0.050	
SV81A	Subbasin	Godding Hollow and Tri-Town Tribs.	16.86	60.9	74.3	0.4	0.1	6.3	3.1	19.5	3.6						
J81A	Junction	Godding Hollow and Tri-Town at SVC	28.21														
SV80A	Subbasin	SVC Area below I-25	4.35	56.3	73.9	0.4	0.1	2.1	1.4	1.8	2.9						
J6	Junction	SVC at Colorado Blvd.	921.46														
R90	Reach	SVC between Colorado Blvd. and SH 66	921.46									20931	0.001	0.100	0.100	0.100	
SV73	Subbasin	SVC Area at Colorado Blvd.	7.52	55.1	74.0	0.4	0.1	6.9	3.1	28.3	3.5						
SV74	Subbasin	SVC Area East of Firestone	13.00	55.6	71.3	0.4	0.1	9.0	3.4	36.0	3.8						
J3	Junction	SV at SH 66	941.98														
R50	Reach	SVC below SH 66	941.98									14393	0.001	0.100	0.100	0.100	
SV72	Subbasin	SVC Area below SH 66	4.89	55.9	74.2	0.4	0.1	6.0	2.5	22.7	3.2						
SV71B	Subbasin	Unnamed SVC Trib near Mead	6.45	48.8	71.2	0.4	0.1	4.2	1.7	69.3	2.1						
J71B	Junction	Unnamed SVC Trib near Mead at I-25	6.45														
R40	Reach	Unnamed SVC Trib through Thomas Lake	6.45									26213	0.006	0.050	0.050	0.050	
SV71A	Subbasin	Unnamed SVC Trib Area near Thomas Lake	11.38	53.3	74.8	0.4	0.1	5.0	3.6	67.8	2.9						
J71A	Junction	Unnamed SVC Trib below Thomas Lake	17.83														
J56	Junction	SV at County Road 34	964.71														
R30	Reach	SVC below CR 34	964.71									6362	0.001	0.100	0.100	0.100	
SV70	Subbasin	SVC Area near CR 34	0.88	47.4	70.2	0.4	0.1	2.3	1.2	19.7	1.9						
SV69	Subbasin	Maintenoma Reservoir Trib. Area	7.04	50.9	73.3	0.4	0.1	5.9	3.2	55.9	3.0						
J61	Junction	SV above UPRR Bridge	972.63														
R10	Reach	SVC above South Platte Confluence	972.63									16255	0.001	0.100	0.100	0.100	
SV68	Subbasin	SVC Area above South Platte Confluence	5.26	45.6	72.7	0.4	0.1	5.2	1.7	11.4	3.0						
Outlet1	Sink	SVC at South Platte River	977.89														

Appendix D.2

Rainfall Data

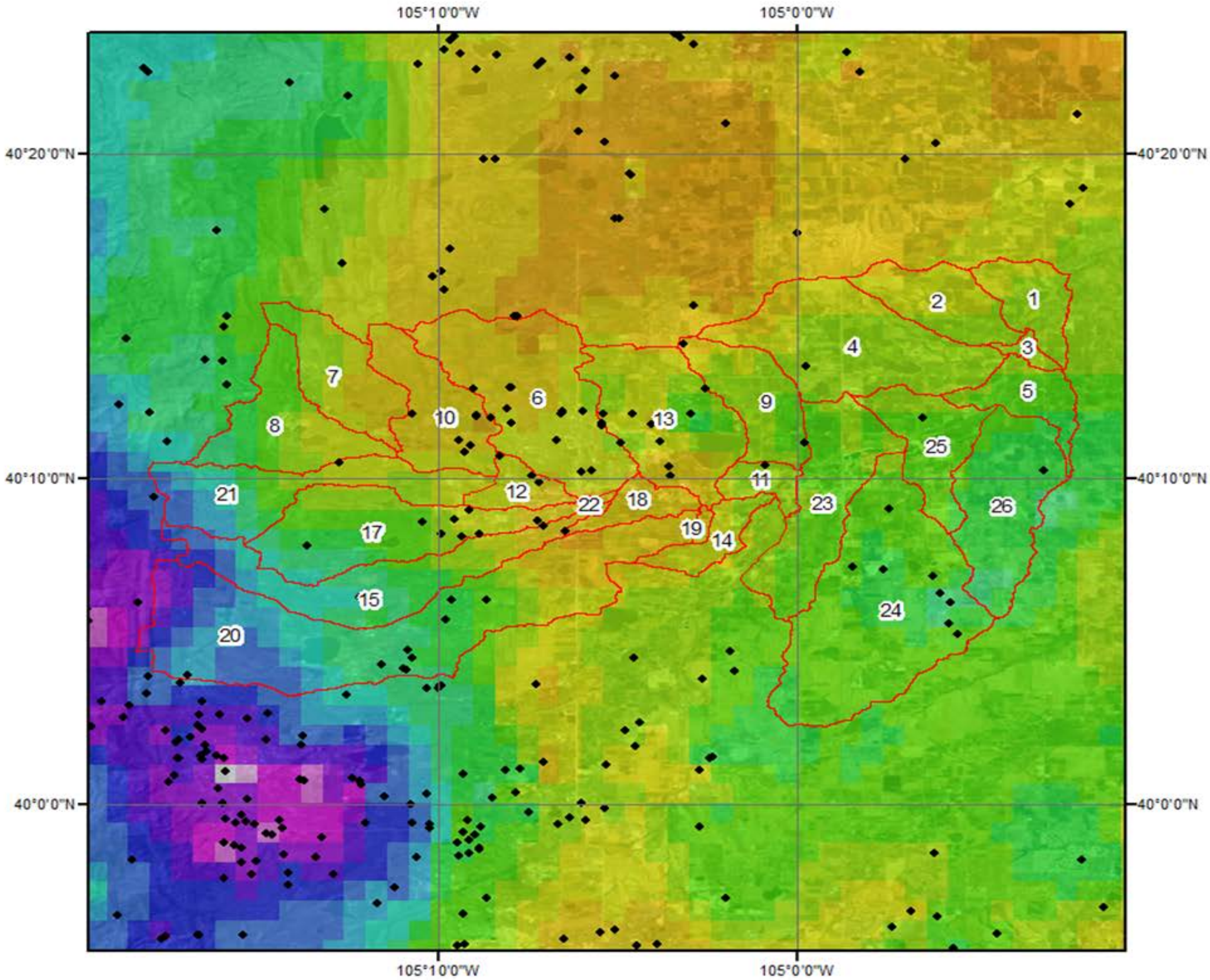
10-day Precipitation Map

NRCS 24-hour Rainfall Distributions

NOAA Atlas 14 Point Precipitation Frequency Estimates

Depth-Area Reduction Factor (DARF) Application

Appendix D.2



Total 10-day Precipitation (in)
Sept 8, 2013 - Sept 17, 2013
SPAS #1302

Gauges

◆ 1302 Stations

Miles: 0, 2.5, 5, 10
Kilometers: 0, 2.5, 5, 10, 15

Precipitation (inches)

0.13 - 1.00
1.01 - 2.00
2.01 - 3.00
3.01 - 4.00
4.01 - 5.00
5.01 - 6.00
6.01 - 7.00
7.01 - 8.00
8.01 - 9.00
9.01 - 10.00
10.01 - 11.00
11.01 - 12.00
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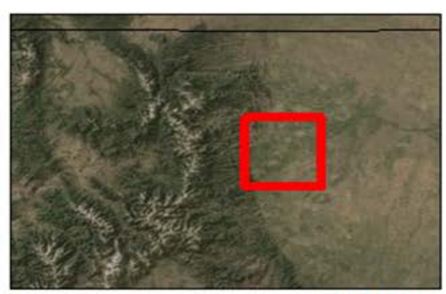
20.01 - 21.00

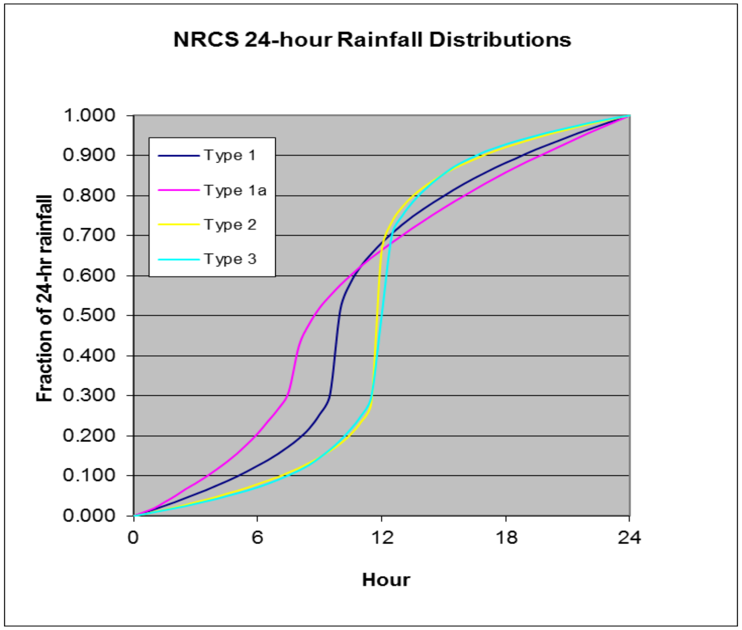


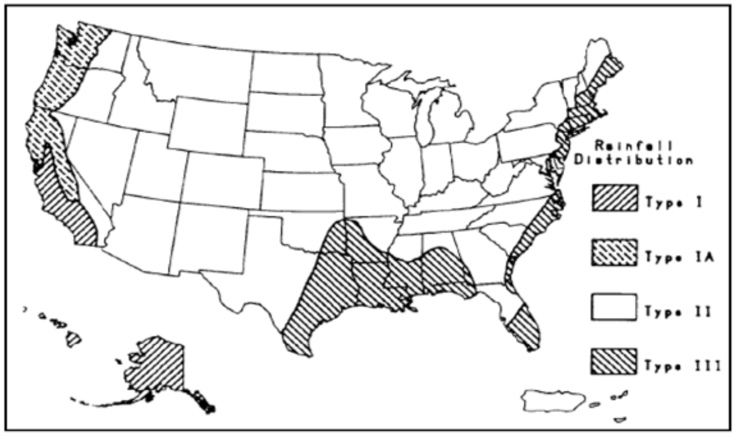
05/08/2014

Appendix D.2 (continued)

NRCS 24-hour Rainfall Distributions

Time hours	t/T	Type 1 Storm	Type 1A Storm	Type II Storm	Type III Storm
		(Cumulative Precipitation)/(Total Storm Precipitation)			
0.0	0.000	0.000	0.000	0.000	0.000
0.5	0.021	0.008	0.010	0.005	0.005
1.0	0.042	0.017	0.020	0.011	0.010
1.5	0.063	0.026	0.035	0.016	0.015
2.0	0.083	0.035	0.050	0.022	0.020
2.5	0.104	0.045	0.067	0.028	0.025
3.0	0.125	0.055	0.082	0.035	0.031
3.5	0.146	0.065	0.098	0.041	0.037
4.0	0.167	0.076	0.116	0.048	0.043
4.5	0.188	0.087	0.135	0.056	0.050
5.0	0.208	0.099	0.156	0.063	0.057
5.5	0.229	0.112	0.180	0.071	0.064
6.0	0.250	0.126	0.206	0.080	0.072
6.5	0.271	0.140	0.237	0.089	0.081
7.0	0.292	0.156	0.268	0.098	0.091
7.5	0.313	0.174	0.310	0.109	0.102
8.0	0.333	0.194	0.425	0.120	0.114
8.5	0.354	0.219	0.480	0.133	0.128
9.0	0.375	0.254	0.520	0.147	0.146
9.5	0.396	0.303	0.550	0.163	0.166
10.0	0.417	0.515	0.577	0.181	0.189
10.5	0.438	0.583	0.601	0.204	0.217
11.0	0.458	0.624	0.624	0.235	0.250
11.5	0.479	0.655	0.645	0.283	0.298
12.0	0.500	0.682	0.664	0.663	0.500
12.5	0.521	0.706	0.683	0.735	0.702
13.0	0.542	0.728	0.701	0.772	0.750
13.5	0.563	0.748	0.719	0.799	0.784
14.0	0.583	0.766	0.736	0.820	0.811
14.5	0.604	0.783	0.753	0.838	0.834
15.0	0.625	0.799	0.769	0.854	0.854
15.5	0.646	0.815	0.785	0.868	0.872
16.0	0.667	0.830	0.800	0.880	0.886
16.5	0.688	0.844	0.815	0.891	0.898
17.0	0.708	0.857	0.830	0.902	0.910
17.5	0.729	0.870	0.844	0.912	0.919
18.0	0.750	0.882	0.858	0.921	0.928
18.5	0.771	0.893	0.871	0.929	0.936
19.0	0.792	0.905	0.884	0.937	0.943
19.5	0.813	0.916	0.896	0.945	0.950
20.0	0.833	0.926	0.908	0.952	0.957
20.5	0.854	0.936	0.920	0.959	0.963
21.0	0.875	0.946	0.932	0.965	0.969
21.5	0.896	0.956	0.944	0.972	0.975
22.0	0.917	0.965	0.956	0.978	0.981
22.5	0.938	0.974	0.967	0.984	0.986
23.0	0.958	0.983	0.978	0.989	0.991
23.5	0.979	0.991	0.989	0.995	0.996
24.0	1.000	1.000	1.000	1.000	1.000

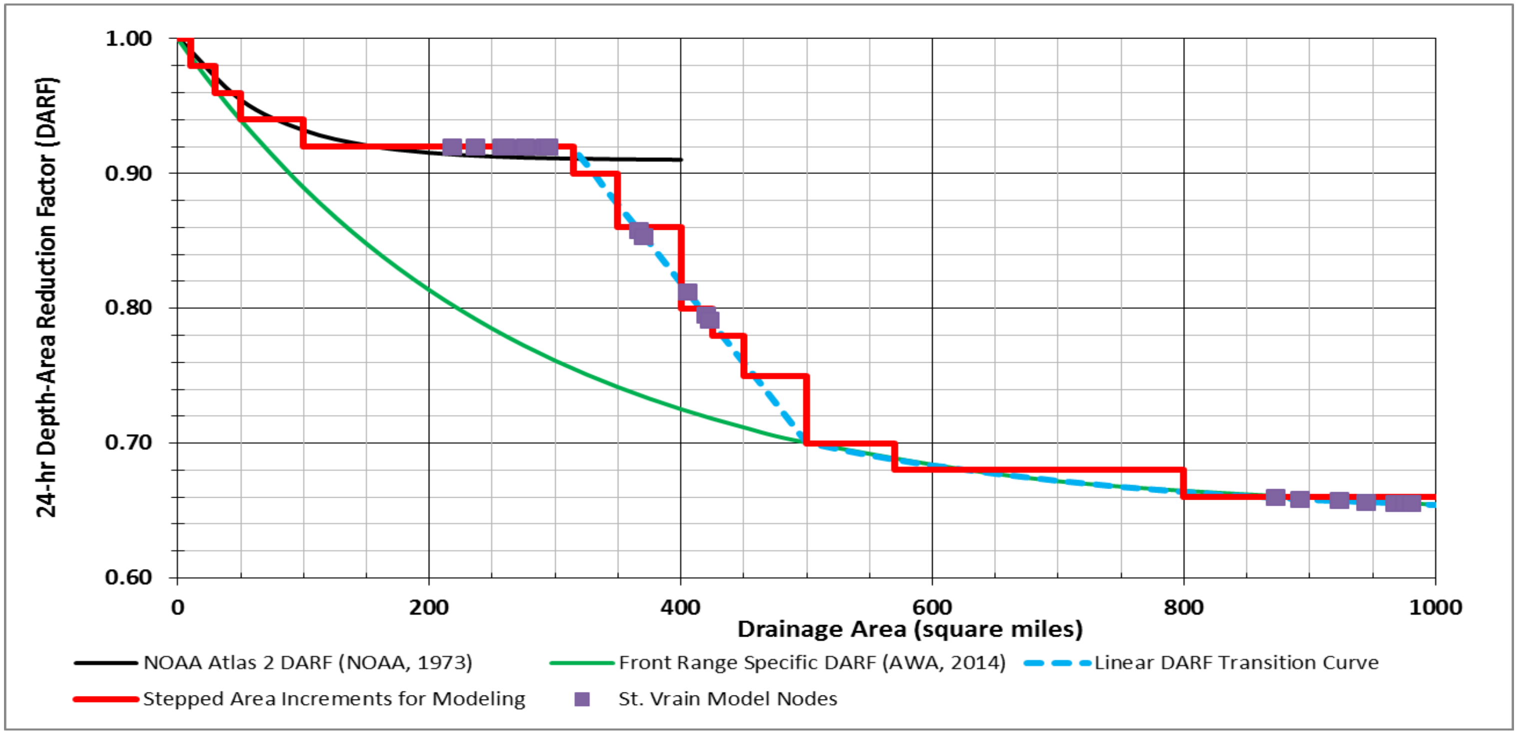




Lower St. Vrain - Phase 2
NOAA Atlas 14, Volume 8, Version 2
Point Precipitation Frequency Estimates

Basin ID	Centroid Lat.	Centroid Long.
SV68	40.259	-104.89
SV69	40.257	-104.934
SV70	40.233	-104.895
SV71A	40.23	-104.956
SV71B	40.244	-105.004
SV72	40.209	-104.894
SV73	40.184	-104.938
SV74	40.159	-104.903
SV75	40.229	-105.135
SV76	40.217	-105.209
SV77A	40.215	-105.244
SV77B	40.184	-105.234
SV80A	40.186	-104.974
SV80B	40.174	-104.989
SV80C	40.128	-104.999
SV81A	40.121	-104.952
SV81B	40.071	-104.977
SV83	40.208	-105.02
SV84	40.202	-105.168
SV85	40.164	-105.016
SV86A	40.166	-105.186
SV86B	40.154	-105.265
SV87	40.161	-105.13
SV88	40.198	-105.06
SV89	40.161	-105.13
SV90	40.136	-105.04
SV92A	40.15	-105.094
SV92B	40.127	-105.136
SV92C	40.112	-105.187
SV92D	40.12	-105.254
SV93	40.184	-105.106
SV94A	40.144	-105.146
SV94B	40.138	-105.213
SV95	40.153	-105.073
SV96	40.142	-105.044
SV97A	40.128	-105.081
SV97B	40.1	-105.145
SV97C	40.082	-105.246

100% - No DARF Adjustment (0 to 10 sq.mi.)					
Basin ID	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**500-yr**
SV68	2.79	3.53	4.16	4.84	6.68
SV69	2.81	3.56	4.20	4.90	6.77
SV70	2.78	3.52	4.14	4.82	6.64
SV71A	2.82	3.57	4.22	4.92	6.79
SV71B	2.84	3.61	4.27	4.99	6.90
SV72	2.78	3.51	4.13	4.81	6.60
SV73	2.80	3.54	4.17	4.85	6.65
SV74	2.78	3.50	4.11	4.78	6.52
SV75	2.84	3.64	4.35	5.13	7.25
SV76	2.89	3.71	4.42	5.20	7.30
SV77A	2.89	3.71	4.41	5.19	7.30
SV77B	2.91	3.73	4.43	5.20	7.26
SV80A	2.83	3.58	4.21	4.90	6.73
SV80B	2.84	3.59	4.23	4.92	6.75
SV80C	2.85	3.60	4.23	4.91	6.68
SV81A	2.81	3.55	4.16	4.82	6.56
SV81B	2.84	3.57	4.18	4.83	6.54
SV83	2.85	3.62	4.27	4.98	6.88
SV84	2.87	3.67	4.37	5.14	7.21
SV85	2.86	3.62	4.26	4.96	6.79
SV86A	2.89	3.70	4.39	5.14	7.14
SV86B	2.97	3.78	4.49	5.25	7.29
SV87	2.90	3.70	4.39	5.12	7.08
SV88	2.87	3.65	4.32	5.04	6.97
SV89	2.90	3.70	4.39	5.12	7.08
SV90	2.88	3.65	4.3	4.99	6.8
SV92A	2.92	3.72	4.4	5.13	7.04
SV92B	2.95	3.75	4.43	5.15	7.04
SV92C	2.96	3.77	4.45	5.19	7.11
SV92D	3.02	3.85	4.55	5.31	7.31
SV93	2.88	3.68	4.37	5.11	7.11
SV94A	2.92	3.72	4.4	5.13	7.06
SV94B	2.95	3.77	4.47	5.22	7.23
SV95	2.91	3.7	4.36	5.07	6.94
SV96	2.88	3.65	4.3	4.99	6.81
SV97A	2.94	3.74	4.4	5.12	6.97
SV97B	2.98	3.79	4.47	5.19	7.08
SV97C	3.07	3.9	4.6	5.35	7.31



Area Range (mi^2)		24-hr DARF
Low	High	
0	10	1.00
10	30	0.98
30	50	0.96
50	100	0.94
100	315	0.92
315	350	0.90
350	400	0.86
400	425	0.80
425	450	0.78
450	500	0.75
500	570	0.70
570	800	0.68
800	1000	0.66

Application of Rainfall Depth-Area Reduction Factors for HEC-HMS Model
In order to evaluate the impacts of the rainfall depth-area reduction factors on the St. Vrain watershed, several model scenarios were run using adjusted rainfall depths. The eight different scenarios included the unadjusted NOAA rainfall depths and seven levels of reduced NOAA rainfall depths (98%, 96%, 94%, 92%, 86%, 80%, and 66%). Each of these eight scenarios were run for all five predictive storms (10-, 25-, 50-, 100-, and 500-yr). The Upper St. Vrain, Upper Lefthand, and Boulder Creek models were also rerun for the lower DARF values to develop the required discharges (in Appendix D.3).

The results from each rainfall depth scenario were saved in a summary spreadsheet and the appropriate peak discharge at any given model node was determined based on the tributary area at that node. The drainage area for model nodes along the St. Vrain are shown as blue squares on the chart above. Appendix D.4 provides the appropriate peak discharge at each model node with respect to drainage area and DARF adjustment.

Appendix D.3

HEC-HMS Inflow Hydrographs

For Upper St. Vrain Creek, Upper Lefthand Creek

And Boulder Creek

2013 Flood 10-day Period

2013 Flood Maximum 24-hour Rainfall Period

10-year Predictive Storm

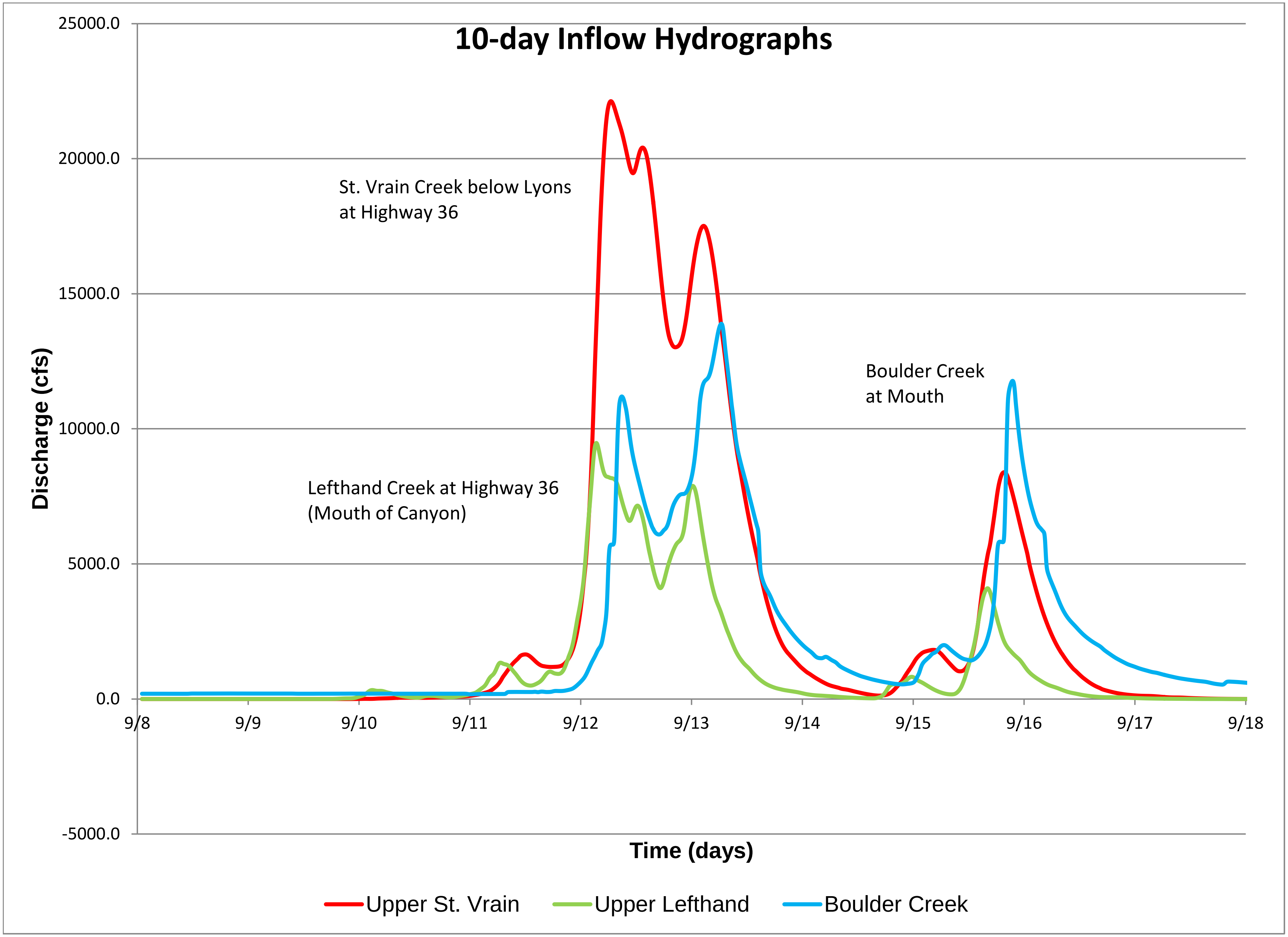
25-year Predictive Storm

50-year Predictive Storm

100-year Predictive Storm

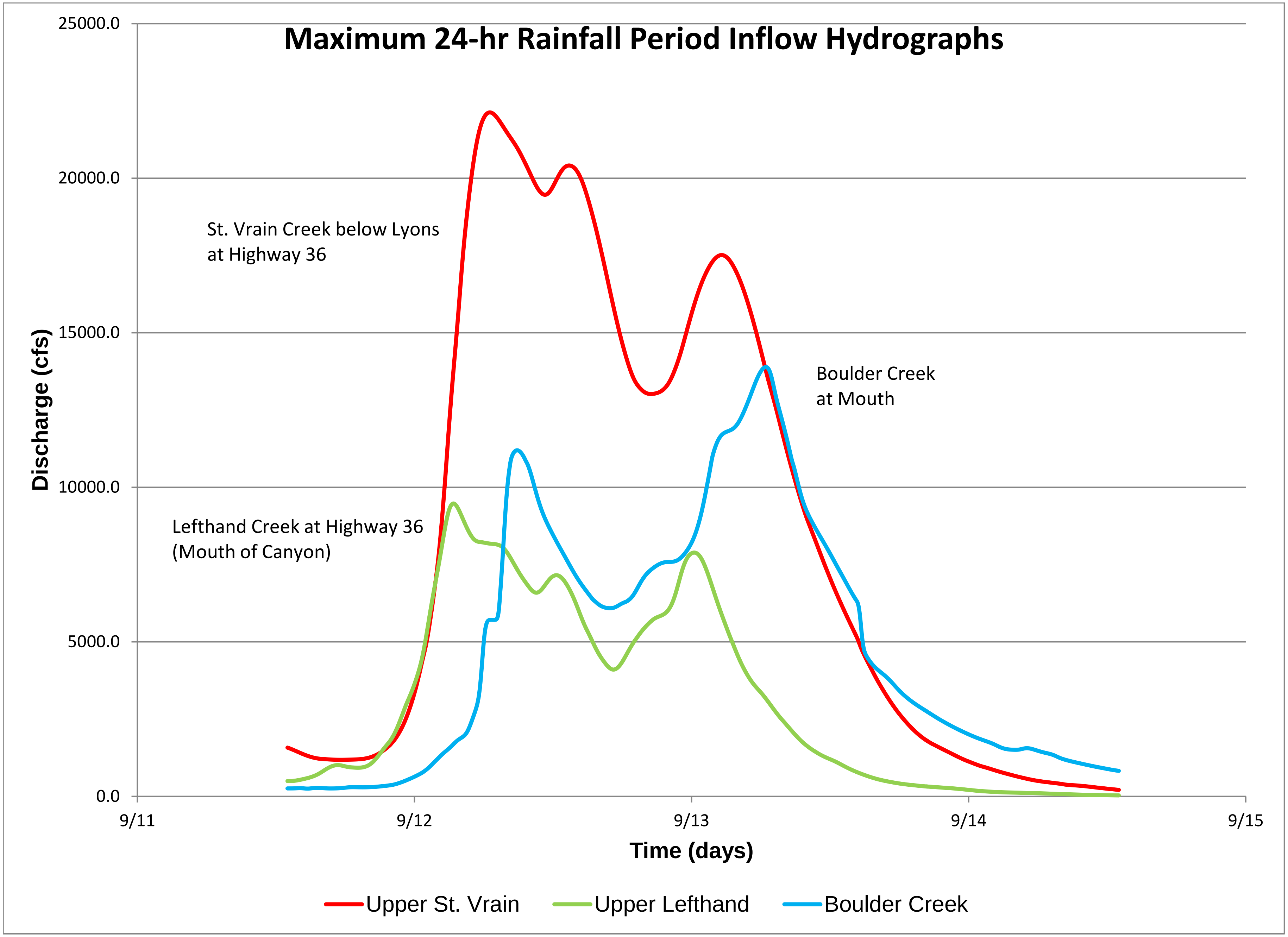
500-year Predictive Storm

Appendix D.3

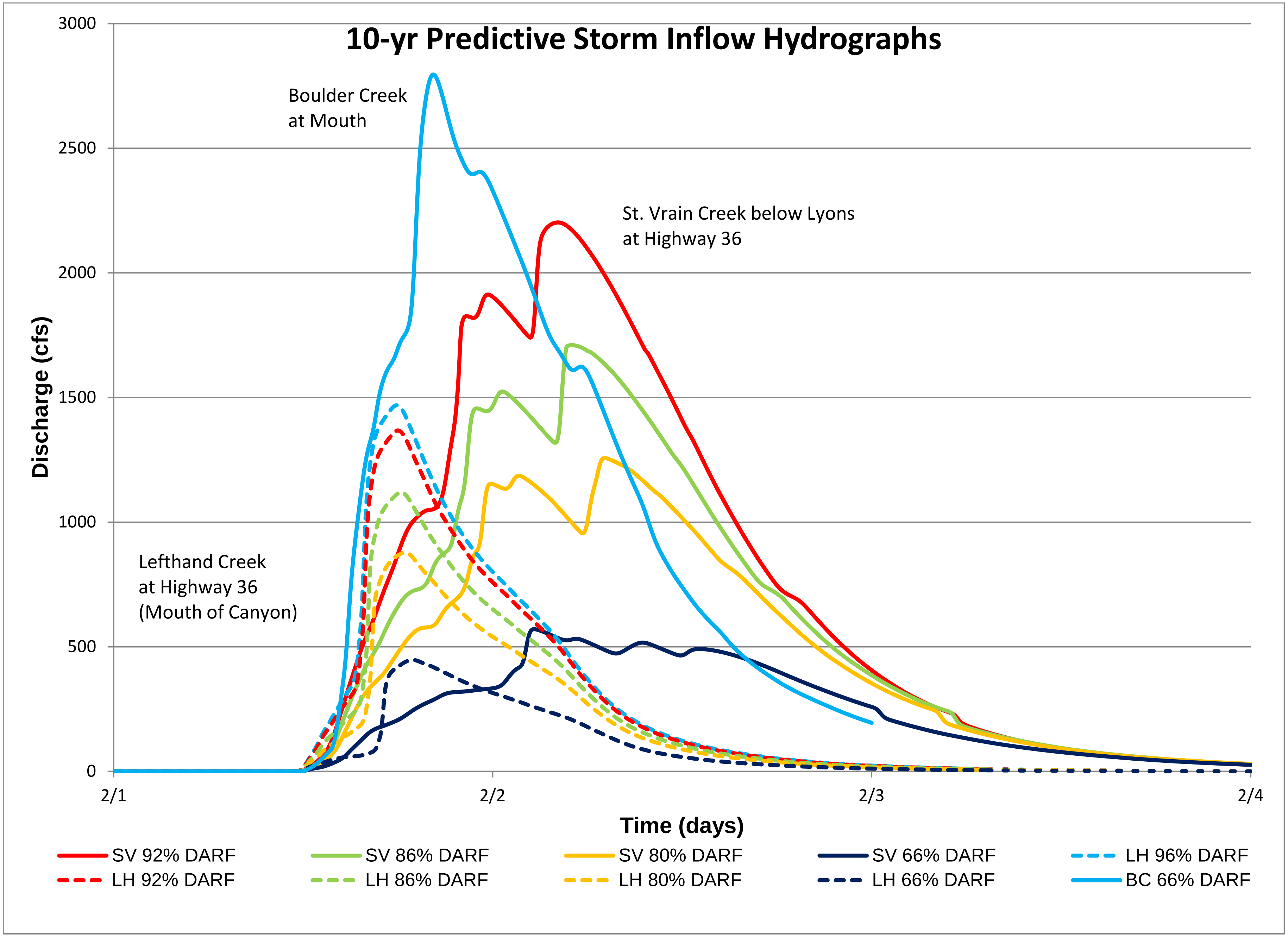


10-day Inflow Hydrographs
25000.0
20000.0
15000.0
10000.0
5000.0
0.0
-5000.0
Discharge (cfs)
St. Vrain Creek below Lyons
at Highway 36
Boulder Creek
at Mouth
Lefthand Creek at Highway 36
(Mouth of Canyon)
9/8
9/9
9/10
9/11
9/12
9/13
9/14
9/15
9/16
9/17
9/18
Time (days)
Upper St. Vrain
Upper Lefthand
Boulder Creek

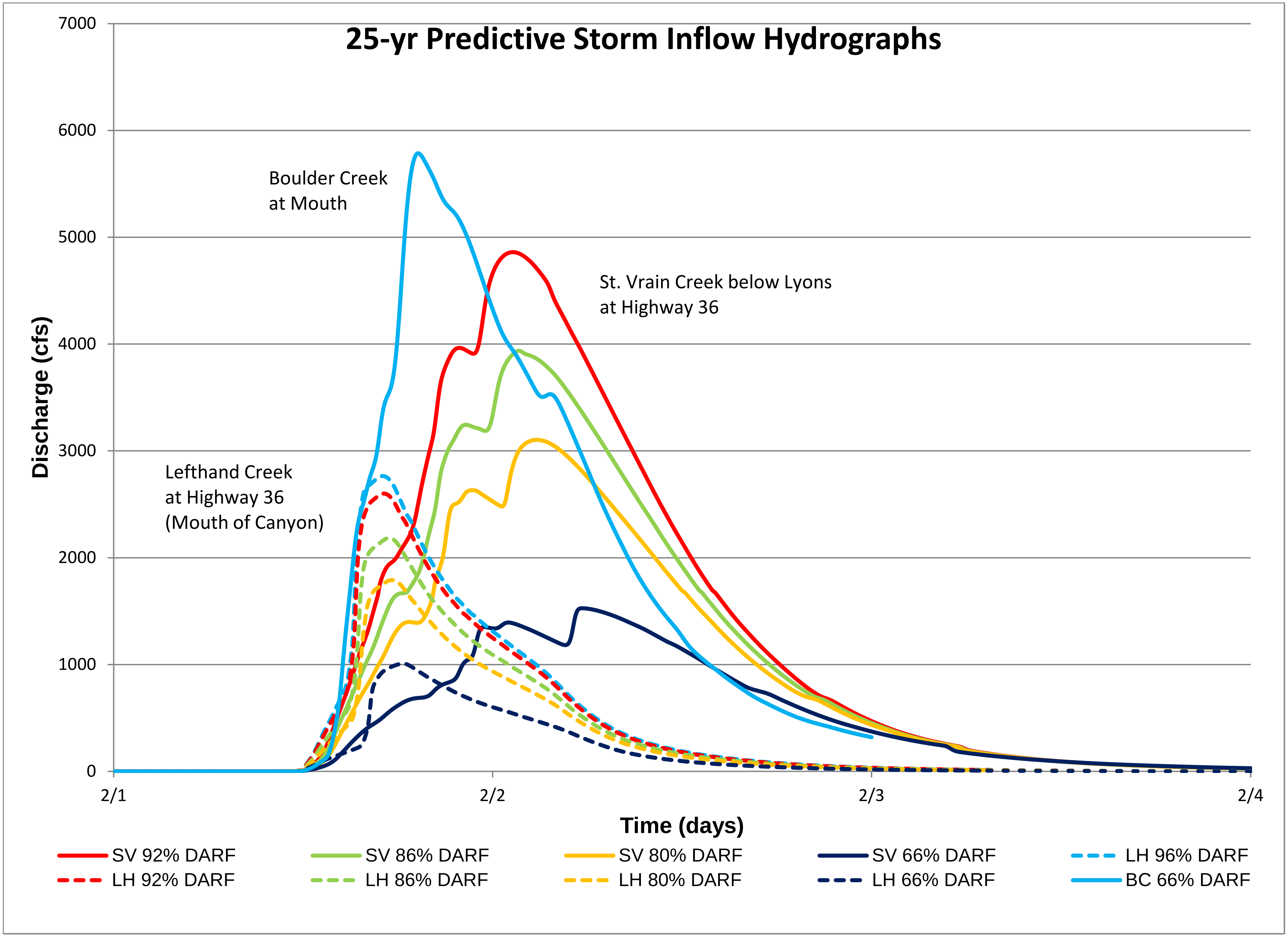
Appendix D.3 (continued)

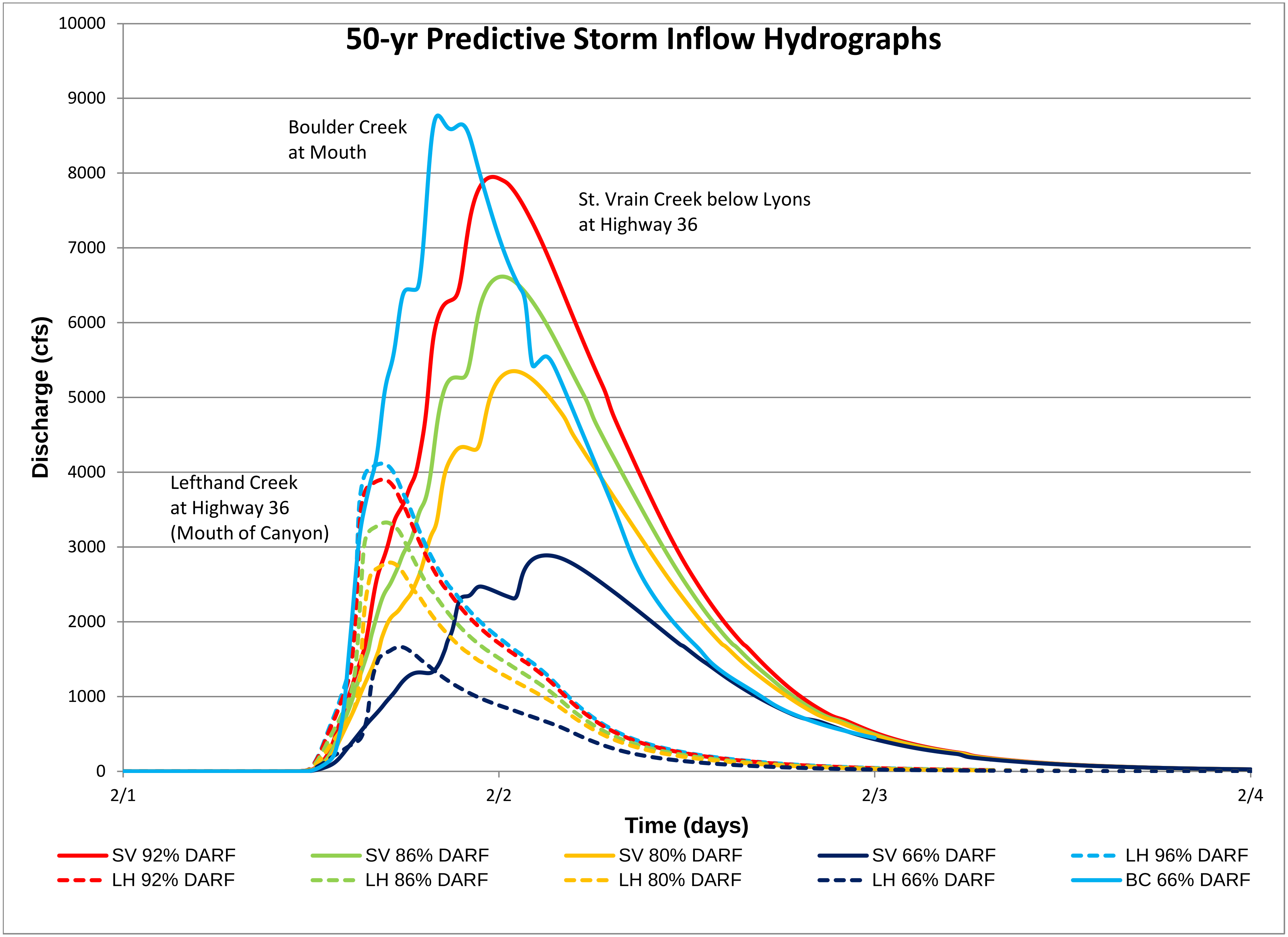


Appendix D.3 (continued)

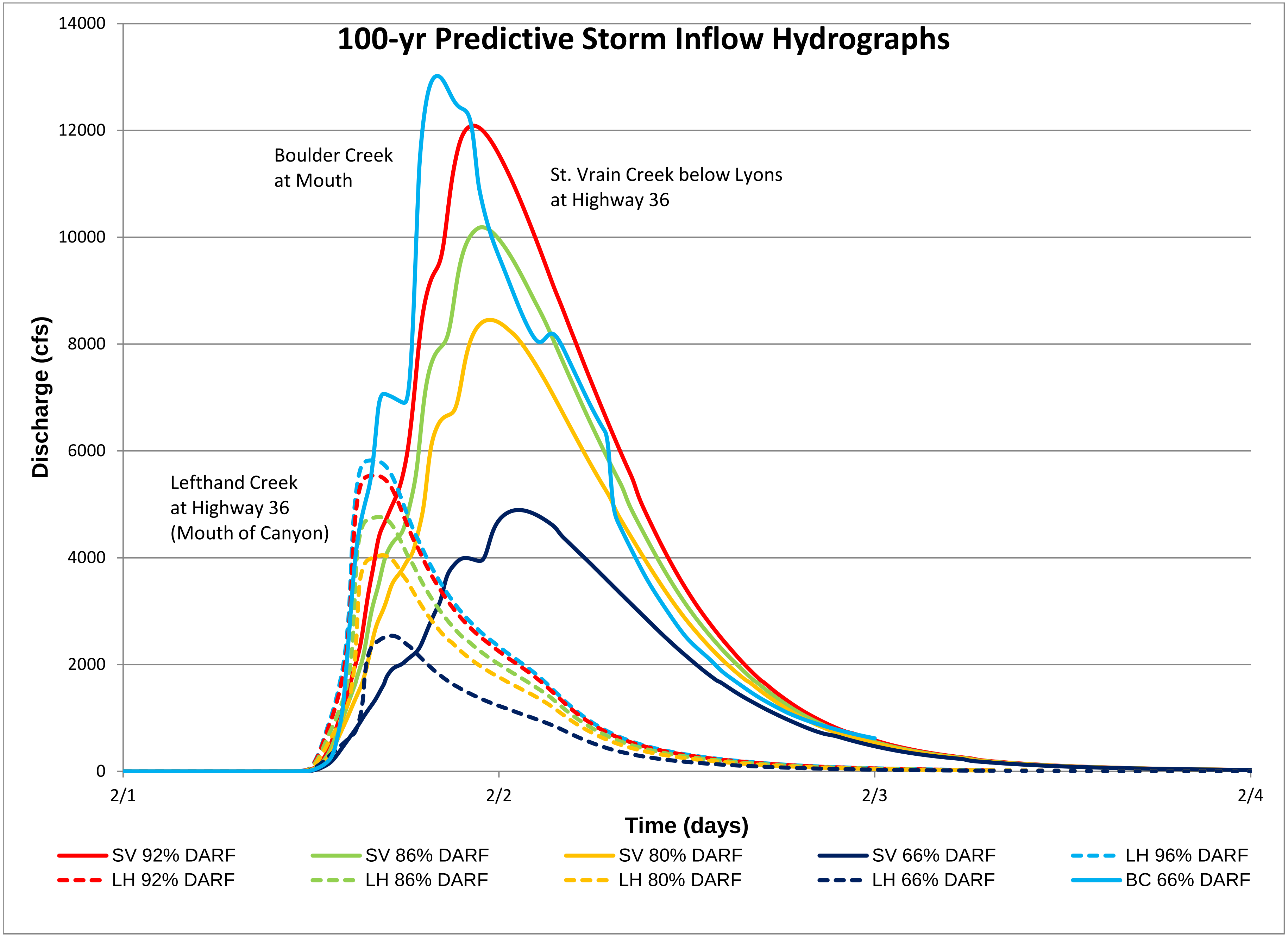


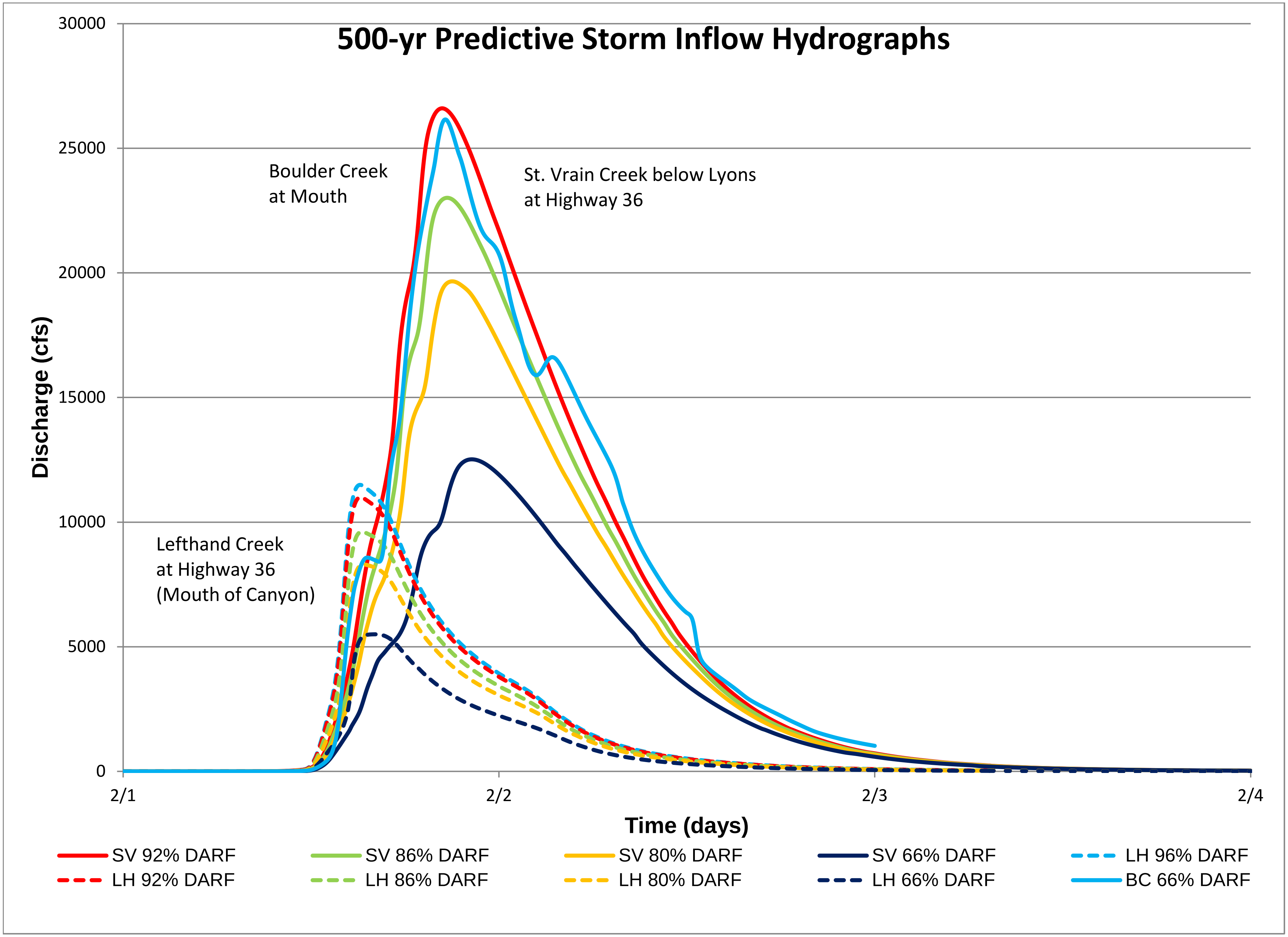
Appendix D.3 (continued)




50-yr Predictive Storm Inflow Hydrographs
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Discharge (cfs)
Boulder Creek
at Mouth
St. Vrain Creek below Lyons
at Highway 36
Lefthand Creek
at Highway 36
(Mouth of Canyon)
2/1
2/2
2/3
2/4
Time (days)
SV 92% DARF
SV 86% DARF
SV 80% DARF
SV 66% DARF
LH 96% DARF
LH 92% DARF
LH 86% DARF
LH 80% DARF
LH 66% DARF
BC 66% DARF

Appendix D.3 (continued)




500-yr Predictive Storm Inflow Hydrographs
30000
25000
20000
15000
10000
5000
0
Discharge (cfs)
Boulder Creek
at Mouth
St. Vrain Creek below Lyons
at Highway 36
Lefthand Creek
at Highway 36
(Mouth of Canyon)
2/1
2/2
2/3
2/4
Time (days)
SV 92% DARF
SV 86% DARF
SV 80% DARF
SV 66% DARF
LH 96% DARF
LH 92% DARF
LH 86% DARF
LH 80% DARF
LH 66% DARF
BC 66% DARF

Appendix D.4

HEC-HMS Model Results

Appendix D.4

Lower St. Vrain		Area	Estimated Peak	HEC-HMS 2013 Flood Calibrated Model 10-day	HEC-HMS (Cal) 2013 Flood Max 24hr Period CN calibrated	HEC-HMS Model (Max24hr CN Calibrated) NOAA Design Storms (DARF 66% to 100%)					FIS Regulatory Peak Discharge				Ayres 2013 Flood Frequency Analysis				
						10-yr	25-yr	50-yr	100-yr	500-yr	10-yr	50-yr	100-yr	500-yr	10-yr	50-yr	100-yr	500-yr	
Design Point	Description	(sq. mi.)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	DARF %
Upper St. Vrain	**Upper SVC Model Inflow Hydrographs**	**218.39**	**23000**	**22,127**	**22,127**	**2,202**	**4,860**	**7,949**	**12,089**	**26,599**	**2,040**	**5,570**	**8,880**	**20,260**	**2,652**	**6,239**	**8,840**	**19,330**	**92%**
J78	SVC at Hwy 36	218.39		22,127	22,127	2,202	4,860	7,949	12,089	26,599									92%
R78	SVC below Hwy 36	218.39		20,018	20,018	2,202	4,859	7,948	12,086	26,594									92%
SV77A	SVC Area below Hwy 36	3.88		370	398	490	784	1,053	1,365	2,245									100%
J77	SVC below Union Road	222.27		20,271	20,320	2,212	4,912	8,073	12,268	26,984	2,480	6,060	8,970	20,270					92%
R70	SVC above 75th Street	222.27		16,581	16,621	2,210	4,907	8,051	12,250	26,957									92%
SV76	SVC Area north of Hygiene Road	8.37		377	419	687	1,055	1,393	1,776	2,842									100%
SV77B	Foothills Reservoir Area near Hygiene Road	6.75		494	552	523	833	1,119	1,447	2,369									100%
J49	SVC upstream of 75th Street	237.39		17,253	17,399	2,364	5,289	8,581	13,182	28,989	3,050	6,850	9,750	21,440					92%
R140	**SVC above Golden Ponds (near Airport Rd)**	**237.39**	**14000**	**14,113**	**14,232**	**2,355**	**5,283**	**8,572**	**13,167**	**28,954**	**3,160**	**6,890**	**9,580**	**19,680**	**2,993**	**17,250**	**35,722**	**186,890**	**92%**
SV84	SVC Area above McIntosh Lake (Airport Road)	9.29		337	309	721	1,088	1,425	1,808	2,869									100%
SV86B	Lykins Gulch Headwater Area	7.50		901	966	684	1,068	1,430	1,834	2,966									100%
J86	Lykins Gulch at Boulder Feeder Canal	7.50		901	966	684	1,068	1,430	1,834	2,966									100%
R160	Lykins Gulch	7.50		899	964	682	1,067	1,426	1,828	2,954									100%
SV86A	Lykins Gulch Area above SVC	4.65		665	663	505	731	929	1,147	1,737									100%
J28	Lykins Gulch at SVC Confluence	12.14		1,527	1,589	1,118	1,706	2,256	2,861	4,516									98%
J29	SVC at Golden Ponds (Lykins Ditch Confluence)	258.82		15,531	15,743	2,688	5,760	9,267	14,422	31,739	3,690	7,610	10,160	20,500					92%
R200	SVC above Dry Creek #1 Confluence	258.82		14,030	14,222	2,651	5,721	9,249	14,386	31,669									92%
SV87	SVC Area above Dry Creek #1 Confluence	2.75		233	233	449	622	772	931	1,360									100%
J22	SVC above Dry Creek #1 Confluence	261.57		14,122	14,323	2,701	5,746	9,292	14,480	31,942	3,660	7,430	9,820	19,940					92%
SV94B	Dry Creek #1 Headwaters	10.73		1,029	1,118	747	1,188	1,600	2,064	3,361									98%
J94B	Dry Creek #1 at Clover Basin Ditch	10.73		1,029	1,118	747	1,188	1,600	2,064	3,361	709	1,604	2,150	3,923					98%
R260	Dry Creek #1 above SVC	10.73		1,027	1,116	746	1,185	1,596	2,057	3,350									98%
SV94A	Dry Creek #1 Area above SVC	2.98		459	451	369	514	639	775	1,134									100%
J23	Dry Creek #1 at SVC Confluence above BNSF Railroad	13.71		1,445	1,529	1,055	1,631	2,159	2,748	4,382	710	1,719	2,315	4,172					98%
J215	SVC at BNSF Railroad	275.28		14,976	15,256	3,523	6,016	9,726	15,248	33,647	4,110	8,240	10,580	21,200					92%
R205	SVC between BNSF Railroad and Hwy 287	275.28		14,961	15,241	3,501	5,986	9,715	15,225	33,601									92%
SV89	SVC Area between BNSF Railroad and Hwy 287	0.95		92	93	188	259	320	385	559									100%
J21	**SVC below Hwy 287 (Main St.) at Dry Creek #1 Old Channel**	**276.24**	**14500**	**14,990**	**15,273**	**3,588**	**5,988**	**9,722**	**15,244**	**33,676**	**4,110**	**8,240**	**10,580**	**21,200**					**92%**
R210	SVC below Hwy 287 (Main St.)	276.24		14,251	14,520	3,584	5,978	9,717	15,232	33,654									92%
SV75	Terry Lake Area (The Slough)	10.72		627	347	969	1,463	1,915	2,434	3,874									98%
J75	Spring Gulch (The Slough) at Ute Hwy 66	10.72		627	347	969	1,463	1,915	2,434	3,874									98%
R190	Spring Gulch (The Slough)	10.72		626	346	966	1,459	1,910	2,427	3,861									98%
SV93	Spring Gulch Area (The Slough)	8.85		566	531	936	1,312	1,642	1,999	2,971									100%
J36	The Slough (Spring Gulch) at SVC Confluence	19.57		1,119	834	1,828	2,694	3,467	4,338	6,730	1,950	3,150	3,650	4,200					98%
J35	SVC at Martin Street (The Slough)	295.81		14,771	15,128	5,147	6,719	10,377	16,332	36,163									92%
R2120	SVC above LHC Confluence	295.81		14,043	14,382	5,145	6,717	10,365	16,317	36,147									92%
Upper Lefthand	Upper LHC Model Inflow Hydrographs	58.12		9,474	9,474	1,469	2,765	4,117	5,822	11,493	1,035	4,145	6,700	14,990					96%
J92E	Lefthand Creek at Hwy 36	58.12		9,474	9,474	1,463	2,751	4,117	5,794	11,458									94%
R481	LHC downstream of Hwy 36	58.12		7,003	7,003	1,461	2,748	4,113	5,792	11,430									94%
SV92D	LHC area above 63rd Street	4.66		417	545	259	612	996	1,485	2,990									100%
J92D	**LHC at 63rd Street**	**62.78**	**7000**	**7,207**	**7,322**	**1,510**	**2,840**	**4,253**	**5,994**	**11,837**	**860**	**3,800**	**6,600**	**14,590**					94%
R480	LHC upstream of Hwy 119	62.78		5,820	5,920	1,398	2,735	4,146	5,862	11,436									94%
SV92C	LHC Area upstream of Hwy 119	6.56		486	616	234	554	903	1,349	2,728									100%
J92C	**LHC at Diagonal Highway (near Airport Rd)**	**69.34**	**8700**	**5,928**	**6,095**	**1,401**	**2,815**	**4,269**	**6,037**	**11,787**	**750**	**3,500**	**6,330**	**13,990**					94%
R380	LHC above Hwy 287 (Main St.)	69.34		5,024	5,171	1,378	2,641	4,038	5,754	11,448									94%
SV92B	LHC Area above Hwy 287 (Main St.)	2.35		253	335	199	380	563	775	1,394									100%
J92B	**LHC at Hwy 287 (Main St.)**	**71.69**	**5000**	**5,086**	**5,256**	**1,378**	**2,644**	**4,062**	**5,810**	**11,557**					**916**	**3,332**	**5,486**	**16,415**	94%
R280	LHC above SVC Confluence	71.69		4,799	4,962	1,373	2,581	3,993	5,737	11,400									94%
SV92A	LHC Area above SVC Confluence	0.26		14	9	29	56	83	114	205									100%
J92A	**LHC at SVC Confluence**	**71.94**	**4800**	**4,800**	**4,964**	**1,373**	**2,581**	**3,993**	**5,739**	**11,410**	**520**	**2,480**	**4,610**	**10,320**					94%
J24	**SVC at Confluence with LHC**	**367.75**	**18500**	**18,651**	**19,172**	**4,740**	**7,367**	**11,935**	**17,399**	**40,122**	**5,250**	**10,950**	**14,850**	**28,670**					**86%**
R230	SVC between LHC and Dry Creek #2	367.75		16,808	17,280	4,691	7,334	11,879	17,330	39,954									86%
SV95	SVC Area between LHC and DC#2	2.97		189	156	326	464	582	711	1,054									100%
J14	**SVC upstream of Dry Creek #2 (County Line Rd.)**	**370.72**		**16,894**	**17,377**	**4,869**	**7,380**	**11,953**	**17,410**	**40,205**	**5,120**	**10,790**	**14,610**	**28,470**					**86%**
SV97C	DC#2 Area above Boulder Reservoir	18.16		964	1,003	1,159	1,817	2,424	3,096	4,956									98%
J97C	Dry Creek #2 at Hwy 119	18.16		964	1,003	1,159	1,817	2,424	3,096	4,956	200	560	800	1,300					98%
R510	Dry Creek #2 above Hwy 287	18.16		963	1,001	1,153	1,797	2,424	3,072	4,921									98%
SV97B	Dry Creek #2 Area above Hwy 287	11.54		1,361	1,308	756	1,139	1,482	1,857	2,869									98%
J97B	Dry Creek #2 at Hwy 287	29.70		1,893	2,105	1,840	2,698	3,604	4,622	7,442	900	1,900	2,600	4,295					98%
R310	Dry Creek #2 above SVC Confluence	29.70		1,891	2,103	1,830	2,677	3,491	4,520	7,382									98%
SV97A	Dry Creek #2 Area above SVC Confluence	5.11		601	576	337	507	655	821	1,262									100%
J15	Dry Creek No. 2 at SVC Confluence	34.81		2,268	2,560	2,007	2,894	3,775	4,919	8,107	900	1,900	2,600	4,240					96%
J13	SVC at Confluence with Dry Creek #2	405.53		18,819	19,788	5,441	8,360	10,983	16,916	38,308									80%
R240	SVC below Dry Creek #2	405.53		18,778	19,746	5,421	8,314	10,959	16,872	38,232									80%
SV88	Area around Calkins Lake/Union Reservoir	14.20		998	666	1,091	1,581	2,009	2,485	3,778									98%
SV96	SVC Area south of channel near County Line	0.44		53	24	56	90	121	156	252									100%
J16	SVC at Spring Gulch (Union Reservoir Ditch)	420.17		19,269	20,293	6,007	9,215	11,431	17,484	39,446	6,010	12,580	16,440	31,790	3,053	7,716	11,146	25,097	80%
R250	SVC above BC Confluence	420.17		17,351	18,271	5,966	9,124	11,374	17,378	39,252									80%
SV90	SVC Area above BC Confluence	3.36		273	192	251	383	503	634	994									100%
J33	SV upstream of Boulder Creek	423.54		17,458	18,395	6,052	9,267	11,546	17,450	39,423	6,010	12,500	16,440	31,790					80%
Boulder Creek	**Boulder Creek Model Inflow Hydrographs**	**446.81**		**13,890**	**13,890**	**4,787**	**9,002**	**13,735**	**18,465**	**37,253**	**2,000**	**7,200**	**12,000**	**31,300**	**2,001**	**3,972**	**5,119**	**8,722**	**78%**
J32	SVC at Confluence with Boulder Creek (Hwy 119)	870.35		25,757	25,222	6,649	11,622	17,573	23,540	43,043	6,110	12,500	16,630	42,400					66%
R170	SVC below BC Confluence	870.35		24,441	23,894	6,617	11,602	17,476	23,492	42,739									66%
SV85	SVC Area below BC Confluence	2.47		320	191	201	319	427	551	894									100%
SV83	Unnamed North Trib to SVC above I-25	7.84		576	300	391	643	878	1,153	1,941									100%
J85	SVC Upstream of I-25	880.66		24,534	24,187	6,730	11,815	17,777	23,918	43,499	5,910	12,140	16,320	41,590					66%
R105	SVC upstream of I-25	880.66		23,234	22,815	6,674	11,765	17,633	23,824	43,042									66%
SV80C	Idaho Creek Tributary Area	6.35		699	542	347	566	768	1,001	1,651									100%
J80C	Idaho Creek Trib. to SVC at I-25	6.35		699	542	347	566	768	1,001	1,651									100%
SV80B	SVC Area upstream of I-25	1.89		298	223	136	240	341	459	803									100%
J42	**SVC at I-25**	**888.90**	**23500**	**23,316**	**23,062**	**6,737**	**11,901**	**17,827**	**24,097**	**43,526**	**6,070**	**12,500**	**16,510**	**41,960**					**66%**
R100	SVC below I-25	888.90		23,280	22,987	6,674	11,882	17,791	24,050	43,432	5,760	12,350	16,350	41,360					66%
SV81B	Godding Hollow Headwaters	11.34		1,161	1,309	394	697	990	1,327	2,331									98%
J81B	Upper Godding Hollow Tributary	11.34		1,161	1,309	394	697	990	1,327	2,331									98%
R290	Godding Hollow	11.34		1,157	1,299	393	694	985	1,320	2,287									98%
SV81A	Godding Hollow and Tri-Town Tribs.	16.86		1,599	1,785	598	1,009	1,390	1,828	3,104									98%
J81A	Godding Hollow and Tri-Town at SVC	28.21		2,655	3,012	973	1,677	2,343	3,109	5,010									98%
SV80A	SVC Area below I-25	4.35		431	320	190	321	446	593	1,020									100%
J6	SVC at Colorado Blvd.	921.46		23,750	24,586	6,897	12,381	18,529	25,086	45,353	5,920	12,900	16,760	41,900					66%
R90	SVC between Colorado Blvd. and SH 66	921.46		23,580	24,099	6,770	12,187	18,175	24,616	44,580									66%
SV73	SVC Area at Colorado Blvd.	7.52		629	463	278	468	651	864	1,480									100%
SV74	SVC Area East of Firestone	13.00		1,243	1,220	346	608	864	1,169	2,063									98%
J3	**SV at SH 66**	**941.98**	**23000**	**23,869**	**24,731**	**6,840**	**12,384**	**18,503**	**25,107**	**45,531**	**5,510**	**12,370**	**16,530**	**40,520**					**66%**
R50	SVC below SH 66	941.98		23,723	24,183	6,663	11,982	17,815	24,151	44,544									66%
SV72	SVC Area below SH 66	4.89		494	339	191	322	447	595	1,021									100%
SV71B	Unnamed SVC Trib near Mead	6.45		429	232	292	526	759	1,038	1,859									100%
J71B	Unnamed SVC Trib near Mead at I-25	6.45		429	232	292	526	759	1,038	1,859									100%
R40	Unnamed SVC Trib through Thomas Lake	6.45		428	232	291	525	757	1,035	1,854									100%
SV71A	Unnamed SVC Trib Area near Thomas Lake	11.38		729	430	498	842	1,178	1,563	2,688									98%
J71A	Unnamed SVC Trib below Thomas Lake	17.83		1,157	641	760	1,329	1,886	2,541	4,464									98%
J56	**SV at County Road 34**	**964.71**	**27000**	**23,893**	**24,425**	**6,708**	**12,123**	**18,070**	**24,558**	**45,379**	**5,520**	**12,400**	**16,560**	**40,590**					**66%**
R30	SVC below CR 34	964.71		23,832	24,078	6,517	11,682	17,378	23,644	44,407									66%
SV70	SVC Area near CR 34	0.88		105	38	37	68	99	136	248									100%
SV69	Maintenoma Reservoir Trib. Area	7.04		441	180	278	477	671	900	1,572									100%
J61	SV above UPRR Bridge	972.63		23,894	24,125	6,525	11,712	17,435	23,739	44,622	5,410	12,200	16,540	40,230	5,086	10,649	13,990	24,789	66%
R10	SVC above South Platte Confluence	972.63		23,869	23,948	6,435	11,394	17,065	23,338	44,205									66%
SV68	SVC Area above South Platte Confluence	5.26		413	101	194	336	474	636	1,120									100%
Outlet1	SVC at South Platte River	977.89		23,894	23,967	6,439	11,405	17,089	23,381	44,315	5,350	12,120	16,520	40,080					66%

Appendix D.5

HEC-HMS Model Results for Profiles

Appendix D.5

Lower St. Vrain

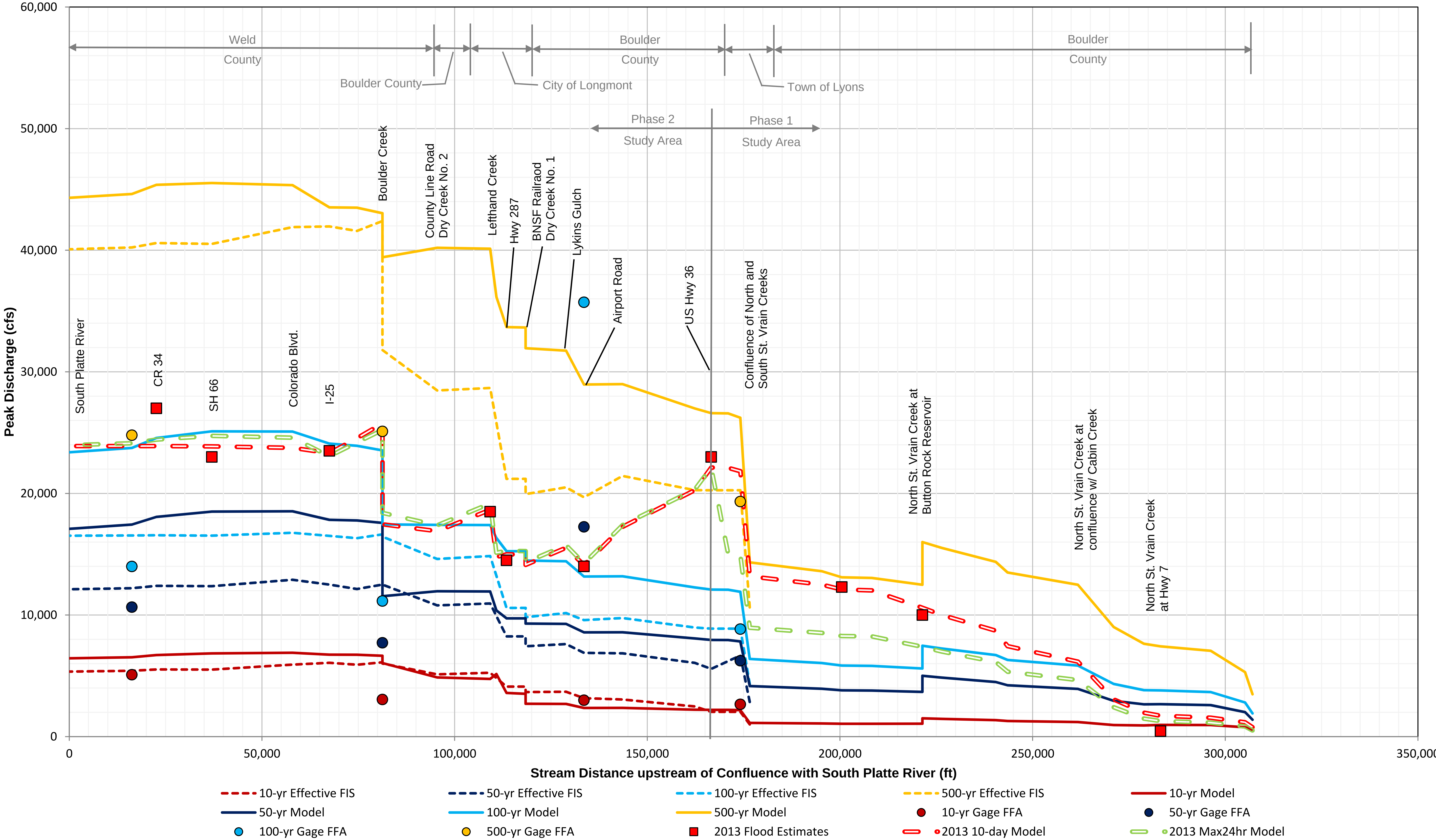
Design Point	Description	Approx. Station (ft)	Area (sq. mi.)	Estimated Peak (cfs)	2013 Flood Calibrated Model 10-day (cfs)	2013 Flood Max 24hr Period CN Calibrated (cfs)	NOAA Design Storms (CN Calib & DARF) 10-yr (cfs)	NOAA 25-yr (cfs)	NOAA 50-yr (cfs)	NOAA 100-yr (cfs)	NOAA 500-yr (cfs)	FIS Regulatory Peak Discharge 10-yr (cfs)	FIS 50-yr (cfs)	FIS 100-yr (cfs)	FIS 500-yr (cfs)	Ayres 2013 Flood Frequency Analysis 10-yr (cfs)	Ayres 50-yr (cfs)	Ayres 100-yr (cfs)	Ayres 500-yr (cfs)
J219	North SVC headwaters	307,072	13		773	511	563	972	1,390	1,906	3,482								
J211	North SVC at confluence with Cony Creek	305,142	21		1,178	838	768	1,375	2,009	2,807	5,297								
J227	North SVC at Copeland Falls	296,232	29		1,551	1,152	954	1,747	2,590	3,662	7,059								
J252A	**North SVC at Hwy 7 (Peak Estimation Point #63 by Bob Jarrett)**	**283,201**	**33**	**450**	**1,713**	**1,279**	**963**	**1,785**	**2,667**	**3,799**	**7,423**								
J252	North SVC at confluence with Horse Creek below Hwy 7	278,890	37		1,972	1,481	920	1,737	2,656	3,819	7,636								
J239	North SVC at confluence with Rock Creek	271,028	53		3,061	2,418	952	1,872	2,943	4,330	9,016								
J249	North SVC at confluence with Cabin Creek	261,767	76		6,162	4,649	1,198	2,446	3,921	5,843	12,487								
J244	North SVC at Coulson Gulch confluence	243,513	83		7,441	5,335	1,279	2,627	4,222	6,303	13,499								
J224	North SVC at confluence with Dry SVC	240,461	91		8,706	6,139	1,351	2,786	4,486	6,704	14,371								
J261	North SVC inflow to Button Rock Reservoir	226,404	98		10,007	6,997	1,457	3,013	4,850	7,247	15,522								
J168	**North SVC inflow to Button Rock Reservoir**	**221,413**	**101**	**10,000**	**10,591**	**7,389**	**1,502**	**3,108**	**5,002**	**7,472**	**16,002**								
Button Rock	**Discharge from Button Rock Reservoir**	**221,413**	**101**		**10,591**	**7,389**	**1,059**	**2,221**	**3,674**	**5,603**	**12,490**								
J278	North SVC below Longmont Reservoir at Hwy 36	208,295	111		12,023	8,240	1,054	2,289	3,788	5,818	13,049								
J286A	**North SVC at Peak Discharge Estimation Point #58 by Bob Jarrett**	**200,500**	**112**	**12,300**	**12,094**	**8,270**	**1,056**	**2,299**	**3,804**	**5,842**	**13,100**								
J286	North SVC at Apple Valley Road	195,311	118		12,501	8,529	1,081	2,377	3,938	6,052	13,599								
J260	North SVC above confluence with South SVC	176,600	124		13,182	8,952	1,123	2,502	4,160	6,386	14,329	1,000	2,850	4,310	10,630				
J258	Confluence of North and South SVC	174,159	215		21,827	14,879	2,178	4,786	7,828	11,910	26,222	2,040	6,670	8,880	20,260	2,652	6,239	8,840	19,330
J255	St Vrain Creek at confluence with Stone Canyon	171,001	218		22,102	15,016	2,202	4,857	7,943	12,080	26,581								
Upper St. Vrain	**Upper SVC Model Inflow Hydrographs**	**166,600**	**218**	**23,000**	**22,127**	**22,127**	**2,202**	**4,860**	**7,949**	**12,089**	**26,599**	**2,040**	**5,570**	**8,880**	**20,260**				
J77	SVC below Union Road	162,345	222		20,271	20,320	2,212	4,912	8,073	12,268	26,984	2,480	6,060	8,970	20,270				
J49	SVC upstream of 75th Street	143,598	237		17,253	17,399	2,364	5,289	8,581	13,182	28,989	3,050	6,850	9,750	21,440				
R140	**SVC above Golden Ponds (near Airport Rd)**	**133,577**	**237**	**14,000**	**14,113**	**14,232**	**2,355**	**5,283**	**8,572**	**13,167**	**28,954**	3,160	6,890	9,580	19,680	**2,993**	**17,250**	**35,722**	**186,890**
J29	SVC at Golden Ponds (Lykins Ditch Confluence)	128,928	259		15,531	15,743	2,688	5,760	9,267	14,422	31,739	3,690	7,610	10,160	20,500				
J22	SVC above Dry Creek #1 Confluence	118,437	262		14,122	14,323	2,701	5,746	9,292	14,480	31,942	3,660	7,430	9,820	19,940				
J215	SVC at BNSF Railroad	118,437	275		14,976	15,256	3,523	6,016	9,726	15,248	33,647	4,110	8,240	10,580	21,200				
J21	**SVC below Hwy 287 (Main St.) at Dry Creek #1 Old Channel**	**113,489**	**276**	**14,500**	**14,990**	**15,273**	**3,588**	**5,988**	**9,722**	**15,244**	**33,676**	**4,110**	**8,240**	**10,580**	**21,200**				
J35	SVC at Martin Street (The Slough)	110,854	296		14,771	15,128	5,147	6,719	10,377	16,332	36,163								
J24	**SVC at Confluence with LHC**	**109,243**	**368**	**18,500**	**18,651**	**19,172**	**4,740**	**7,367**	**11,935**	**17,399**	**40,122**	**5,250**	**10,950**	**14,850**	**28,670**				
J14	SVC upstream of Dry Creek #2 (County Line Rd.)	95,484	371		16,894	17,377	4,869	7,380	11,953	17,410	40,205	5,120	10,790	14,610	28,470				
J33	SV upstream of Boulder Creek	81,279	424		17,458	18,395	6,052	9,267	11,546	17,450	39,423	6,010	12,500	16,440	31,790	3,053	7,716	11,146	25,097
J32	SVC at Confluence with Boulder Creek (Hwy 119)	81,279	870		25,757	25,222	6,649	11,622	17,573	23,540	43,043	6,110	12,500	16,630	42,400				
J85	SVC Upstream of I-25	74,668	881		24,534	24,187	6,730	11,815	17,777	23,918	43,499	5,910	12,140	16,320	41,590				
J42	**SVC at I-25**	**67,500**	**889**	**23,500**	**23,316**	**23,062**	**6,737**	**11,901**	**17,827**	**24,097**	**43,526**	**6,070**	**12,500**	**16,510**	**41,960**				
J6	SVC at Colorado Blvd.	57,942	921		23,750	24,586	6,897	12,381	18,529	25,086	45,353	5,920	12,900	16,760	41,900				
J3	**SV at SH 66**	**37,011**	**942**	**23,000**	**23,869**	**24,731**	**6,840**	**12,384**	**18,503**	**25,107**	**45,531**	**5,510**	**12,370**	**16,530**	**40,520**				
J56	**SV at County Road 34**	**22,617**	**965**	**27,000**	**23,893**	**24,425**	**6,708**	**12,123**	**18,070**	**24,558**	**45,379**	**5,520**	**12,400**	**16,560**	**40,590**				
J61	SV above UPRR Bridge	16,255	973		23,894	24,125	6,525	11,712	17,435	23,739	44,622	5,410	12,200	16,540	40,230	**5,086**	**10,649**	**13,990**	**24,789**
Outlet1	SVC at South Platte River	0	978		23,894	23,967	6,439	11,405	17,089	23,381	44,315	5,350	12,120	16,520	40,080				
J49	J49 (Confluence of LHC and Spring Gulch)	113,954	16		1,325	684	333	606	898	1,246	2,349								
J49A	**J49A (Lefthand Creek at Lickskillet Road)**	**111,050**	**18**	**1,300**	**1,548**	**856**	**364**	**666**	**988**	**1,371**	**2,581**	**400**	**1,800**	**3,180**	**9,060**				
J57a	Lefthand Creek above confluence w/ James Creek	88,604	23		2,626	1,619	502	920	1,359	1,886	3,536	430	2,100	3,690	10,430				
J57	J57 (Confluence of LHC and JC)	86,604	42		6,687	4,368	1,212	2,214	3,227	4,472	8,439								
J54	**J54 (Lefthand Creek below Old Stage Road)**	**72,982**	**47**	**3,520**	**7,734**	**5,148**	**1,260**	**2,336**	**3,438**	**4,804**	**9,220**	**830**	**2,850**	**4,940**	**11,630**				
J67	J67 (LHC confluence with Spruce Gulch)	67,808	51		8,461	5,539	1,340	2,493	3,678	5,151	9,975	890	3,190	5,420	12,540	1,091	3,110	4,810	12,965
J74	J74 (Confluence of LHC and Geer Canyon)	62,792	58		9,394	6,332	1,457	2,743	4,085	5,771	11,388								
Upper Lefthand	Upper LHC Model Inflow Hydrographs	59,149	58		9,474	9,474	1,469	2,765	4,117	5,822	11,493	1,035	4,145	6,700	14,990				
J92D	**LHC at 63rd Street**	**44,059**	**63**	**7,000**	**7,207**	**7,322**	**1,510**	**2,840**	**4,253**	**5,994**	**11,837**	**860**	**3,800**	**6,600**	**14,590**				
J92C	**LHC at Diagonal Highway (near Airport Rd)**	**22,446**	**69**	**8,700**	**5,928**	**6,095**	**1,401**	**2,815**	**4,269**	**6,037**	**11,787**	**750**	**3,500**	**6,330**	**13,990**				
J92B	**LHC at Hwy 287 (Main St.)**	**6,276**	**72**	**5,000**	**5,086**	**5,256**	**1,378**	**2,644**	**4,062**	**5,810**	**11,557**					**916**	**3,332**	**5,486**	**16,145**
J92A	**LHC at SVC Confluence**	**0**	**72**	**4,800**	**4,800**	**4,964**	**1,373**	**2,581**	**3,993**	**5,739**	**11,410**	**520**	**2,480**	**4,610**	**10,320**				
J97C	Dry Creek #2 at Hwy 119	66,050	18		964	1,003	1,159	1,817	2,424	3,096	4,956	200	560	800	1,300				
J97B	Dry Creek #2 at Hwy 287	21,825	30		1,893	2,105	1,840	2,698	3,604	4,622	7,442	900	1,900	2,600	4,295				
J15	Dry Creek No. 2 at SVC Confluence	0	35		2,268	2,560	2,007	2,894	3,775	4,919	8,107	900	1,900	2,600	4,240				
Boulder Creek	Boulder Creek Model Inflow Hydrographs	0	447		13,890	13,890	4,787	9,002	13,735	18,465	37,253	2,000	7,200	12,000	31,300	2,001	3,972	5,119	8,722

Appendix D.6

St. Vrain Creek Peak Discharge Profiles

Appendix D.6

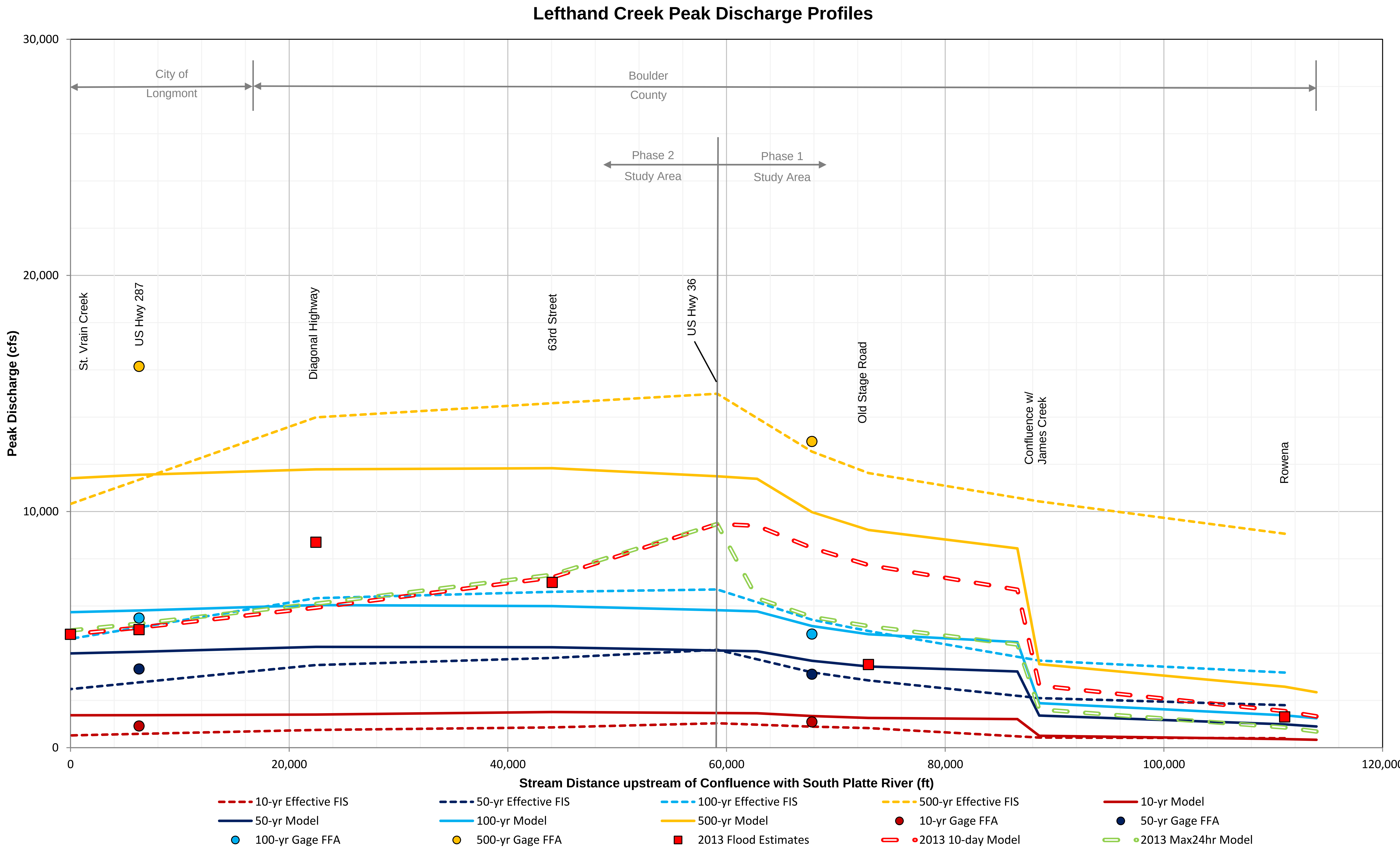
St. Vrain Creek Peak Discharge Profiles



Appendix D.7

Lefthand Creek Peak Discharge Profiles

Appendix D.7



Lefthand Creek Peak Discharge Profiles
City of Longmont
Boulder County
Phase 2 Study Area
Phase 1 Study Area
St. Vrain Creek
US Hwy 287
Diagonal Highway
63rd Street
US Hwy 36
Old Stage Road
Confluence w/ James Creek
Rowena
Peak Discharge (cfs)
30,000
20,000
10,000
0
0
20,000
40,000
60,000
80,000
100,000
120,000
Stream Distance upstream of Confluence with South Platte River (ft)
10-yr Effective FIS
50-yr Effective FIS
100-yr Effective FIS
500-yr Effective FIS
10-yr Model
50-yr Model
100-yr Model
500-yr Model
10-yr Gage FFA
50-yr Gage FFA
100-yr Gage FFA
500-yr Gage FFA
2013 Flood Estimates
2013 10-day Model
2013 Max24hr Model

Appendix D.8

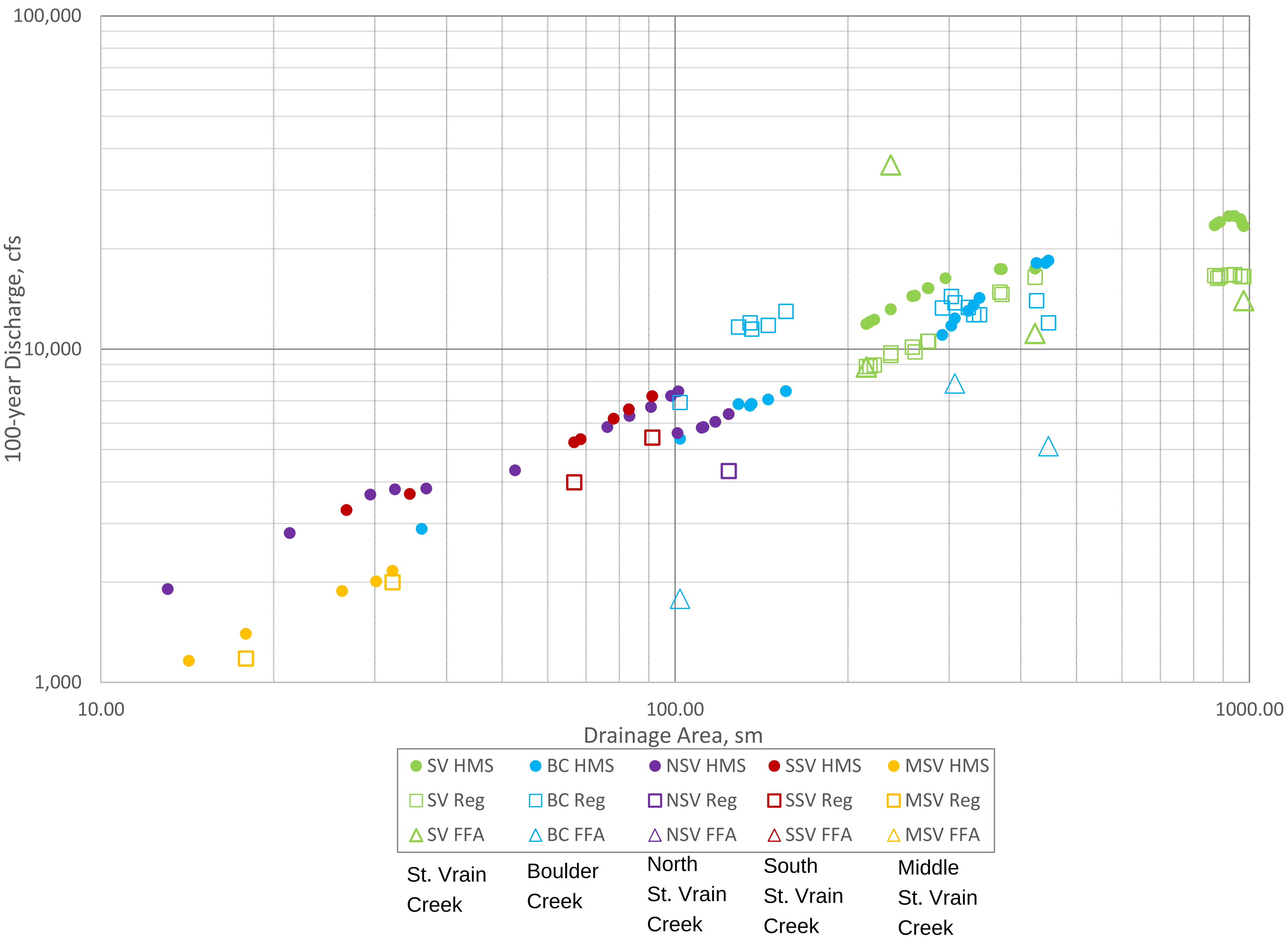
Discharge Comparison Plots

St. Vrain Watershed

Lefthand Watershed

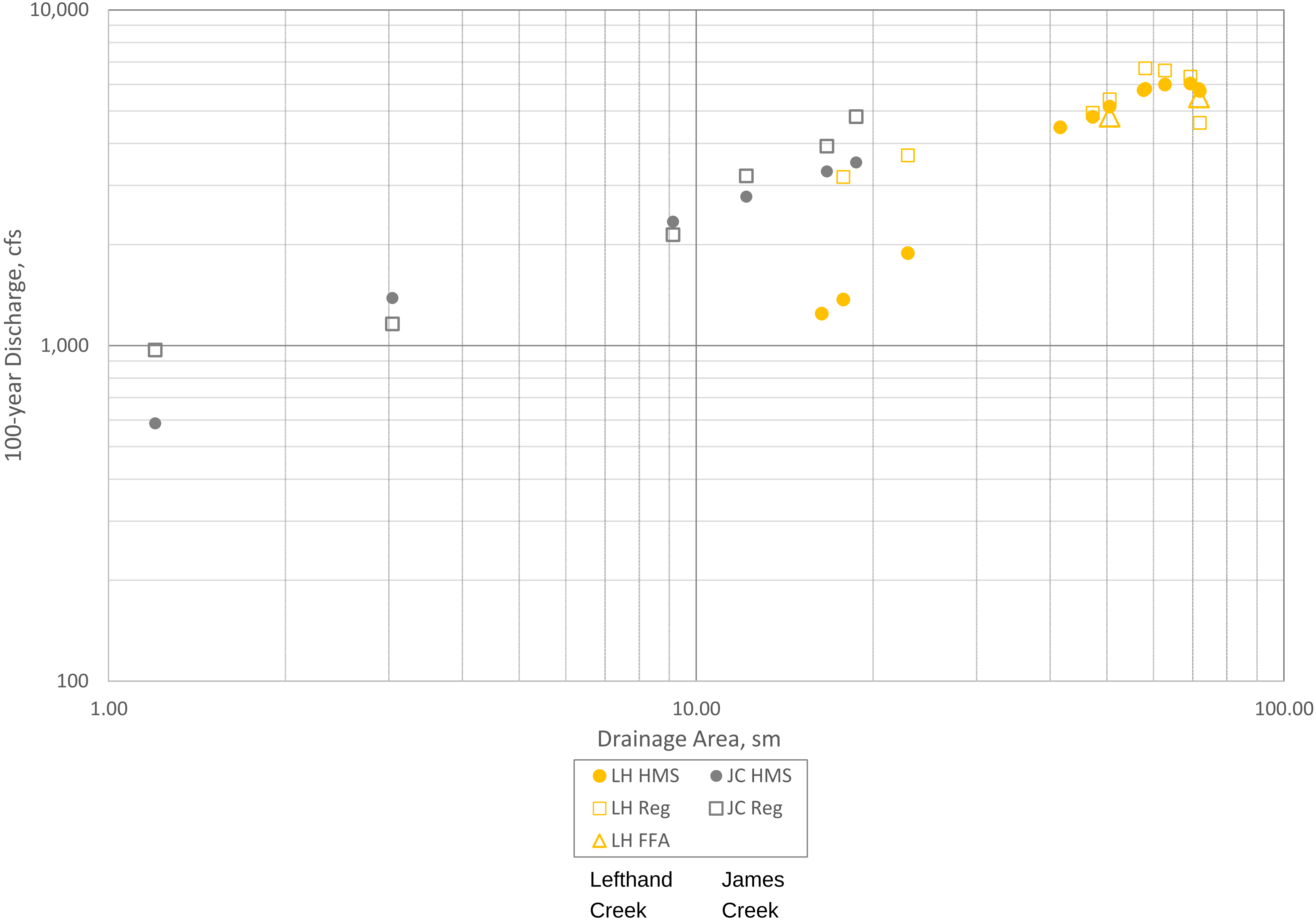
Big Thompson and St. Vrain Watersheds

Appendix D.8

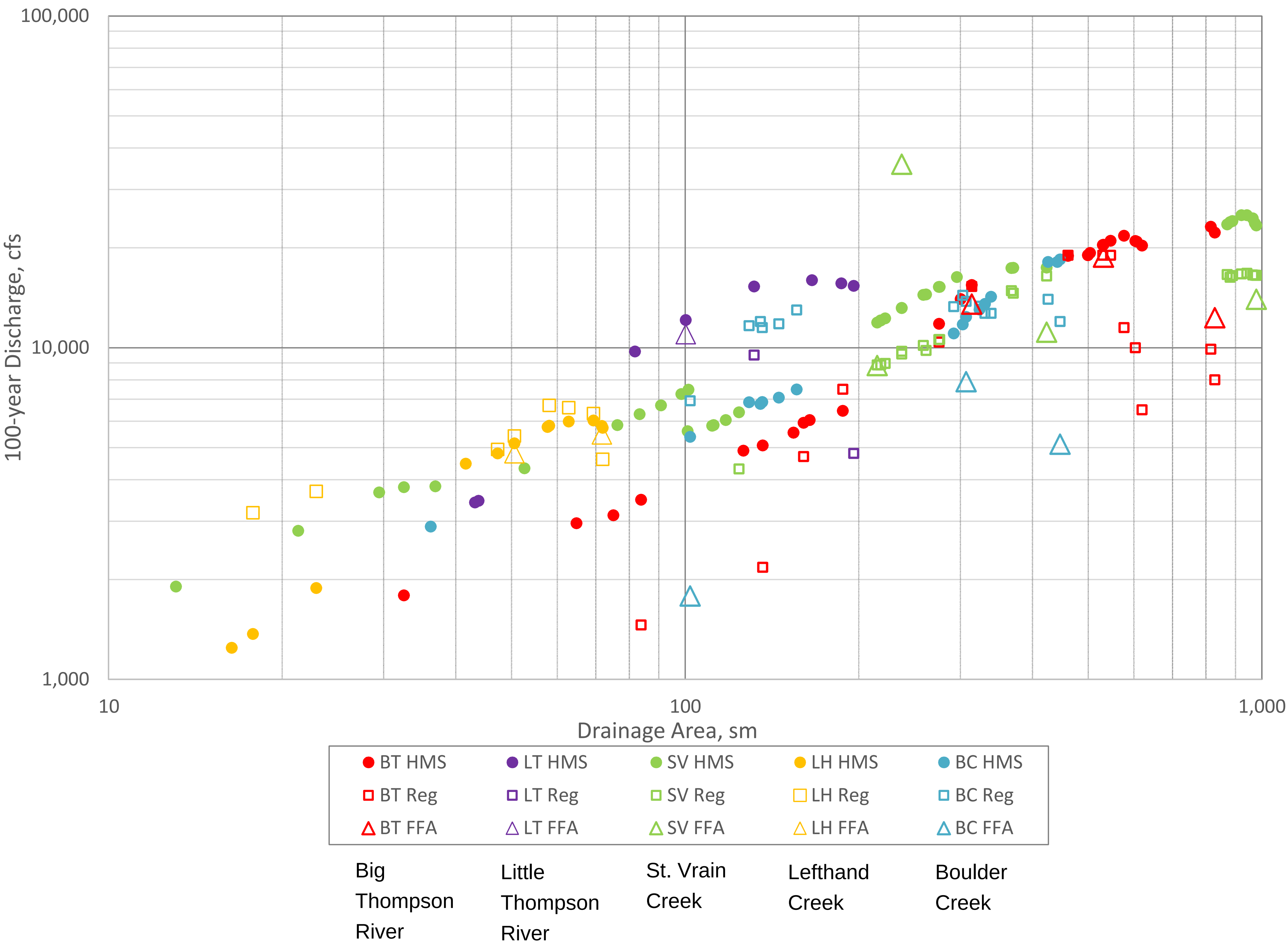


100,000
10,000
1,000
100-year Discharge, cfs
10.00
100.00
1000.00
Drainage Area, sm
SV HMS
BC HMS
NSV HMS
SSV HMS
MSV HMS
SV Reg
BC Reg
NSV Reg
SSV Reg
MSV Reg
SV FFA
BC FFA
NSV FFA
SSV FFA
MSV FFA
St. Vrain Creek
Boulder Creek
North St. Vrain Creek
South St. Vrain Creek
Middle St. Vrain Creek

Appendix D.8 (continued)



Appendix D.8 (continued)



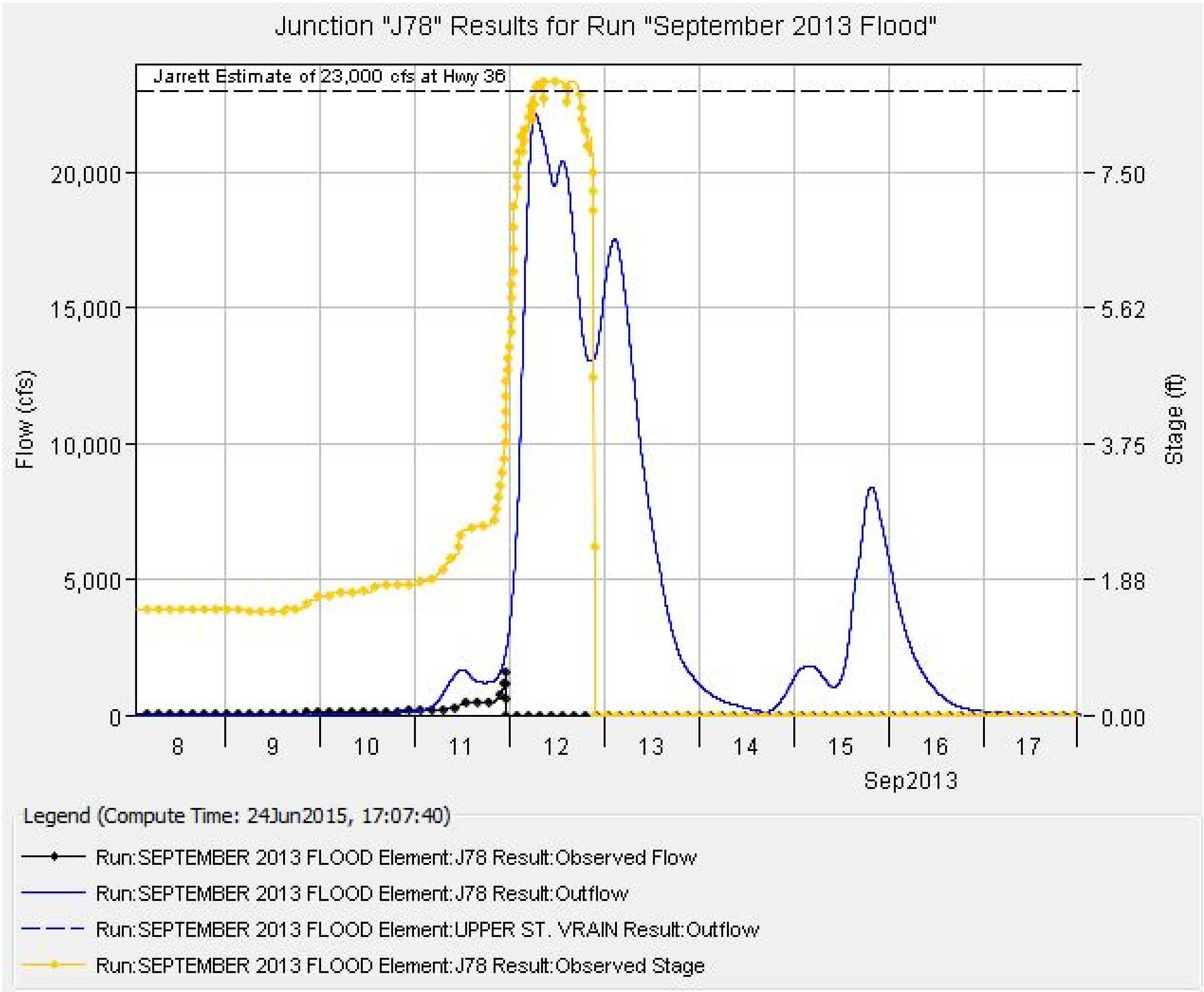
Appendix D.9

Calibrated Model Comparison Plots

St. Vrain Creek at Highway 36 below Lyons
St. Vrain Creek at Airport Road
St. Vrain Creek at SVLONGCO Gage near Hover Road
St. Vrain Creek at Highway 287
Lefthand Creek at 63rd Street
Lefthand Creek at Diagonal Highway
Lefthand Creek at Highway 287
Lefthand Creek at Ken Pratt Blvd (Highway 119)
St. Vrain Creek at Ken Pratt Blvd (Highway 119)
St. Vrain Creek at SVCBLOCO Gage above Boulder Creek
St. Vrain Creek at I-25
St. Vrain Creek at State Highway 66
St. Vrain Creek at County Road 34
St. Vrain Creek at SVCPLACO Gage at Mouth

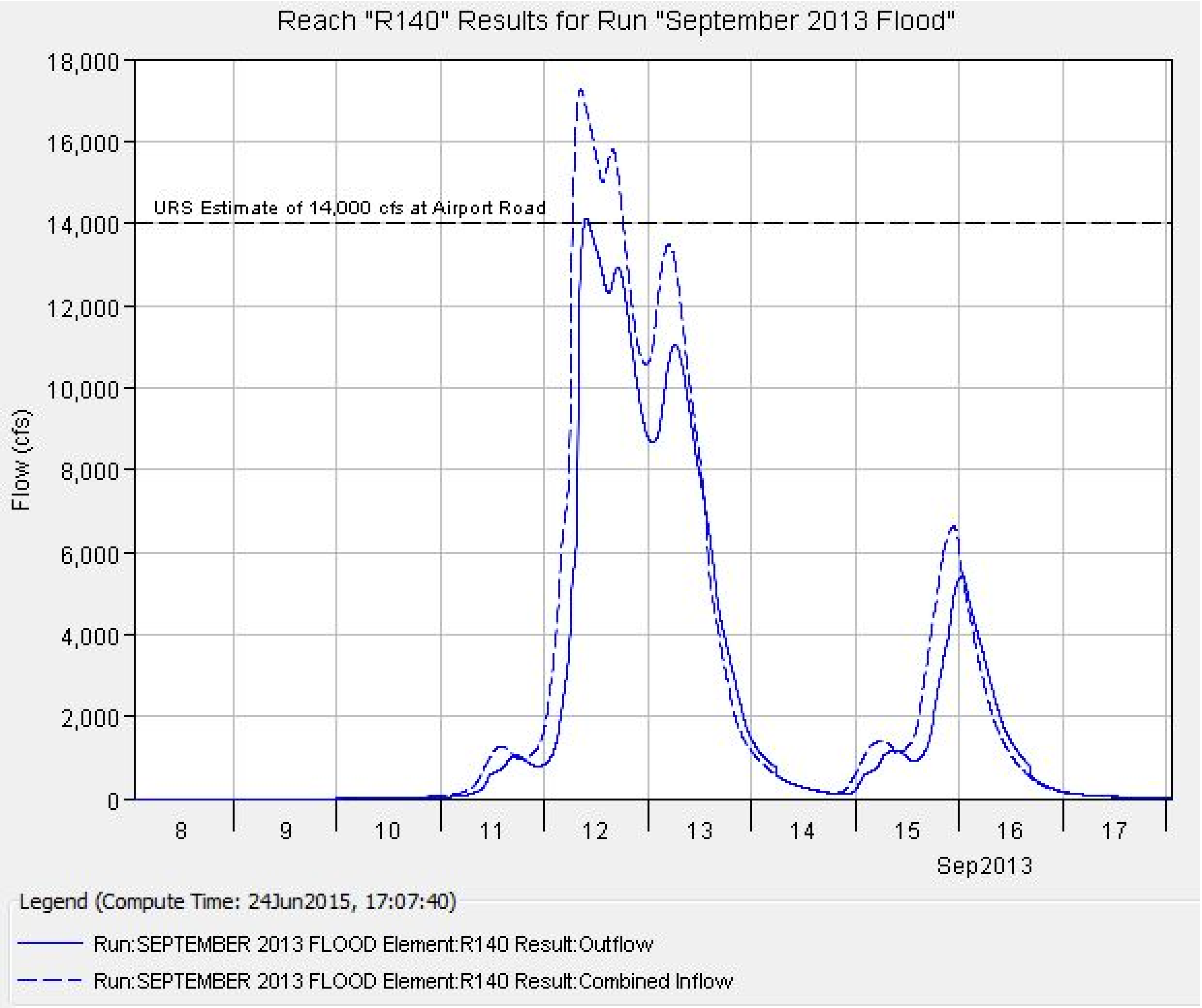
Appendix D.9

St. Vrain Creek – Highway 36 below Lyons



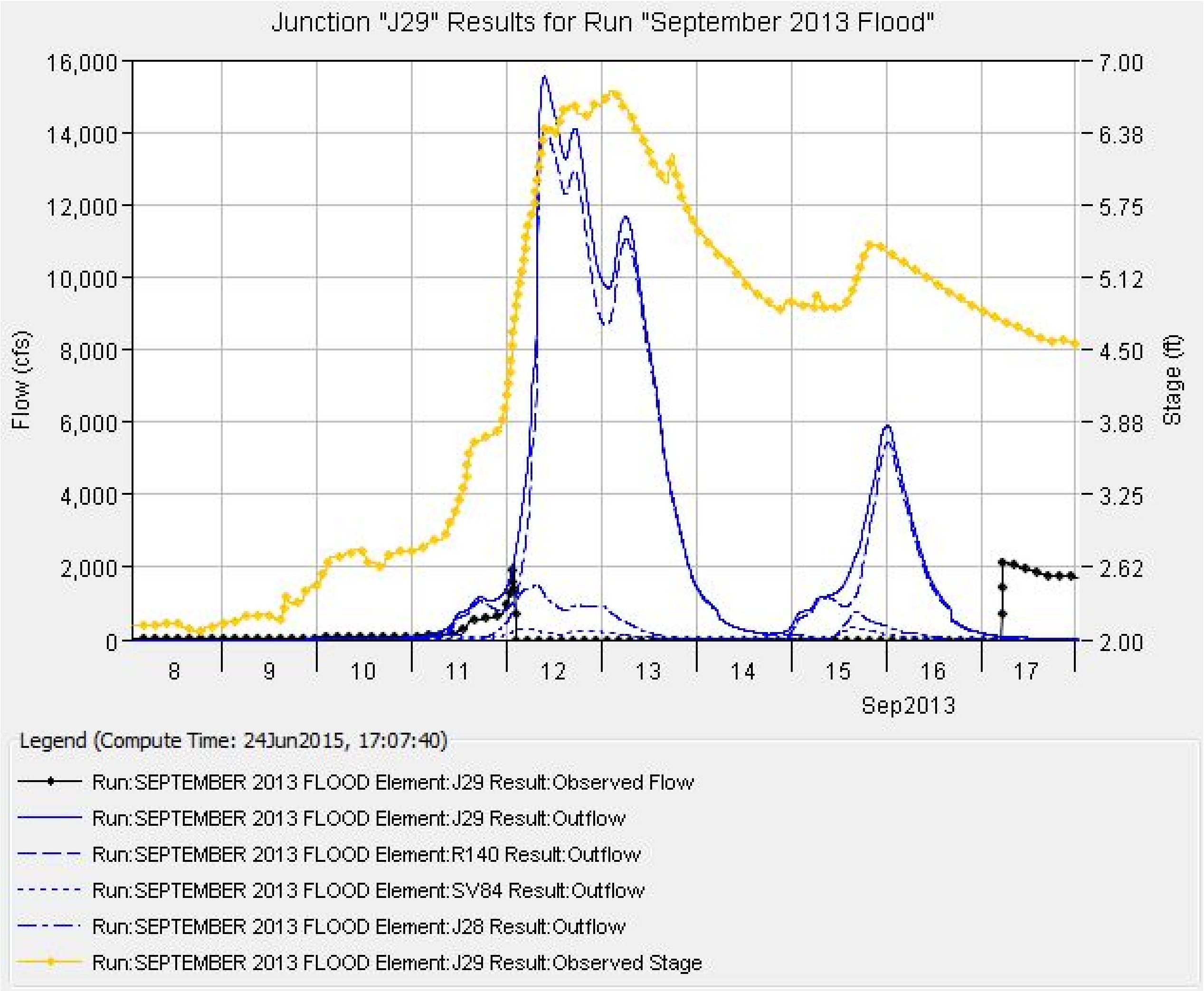
Appendix D.9 (continued)

St. Vrain Creek – Airport Road



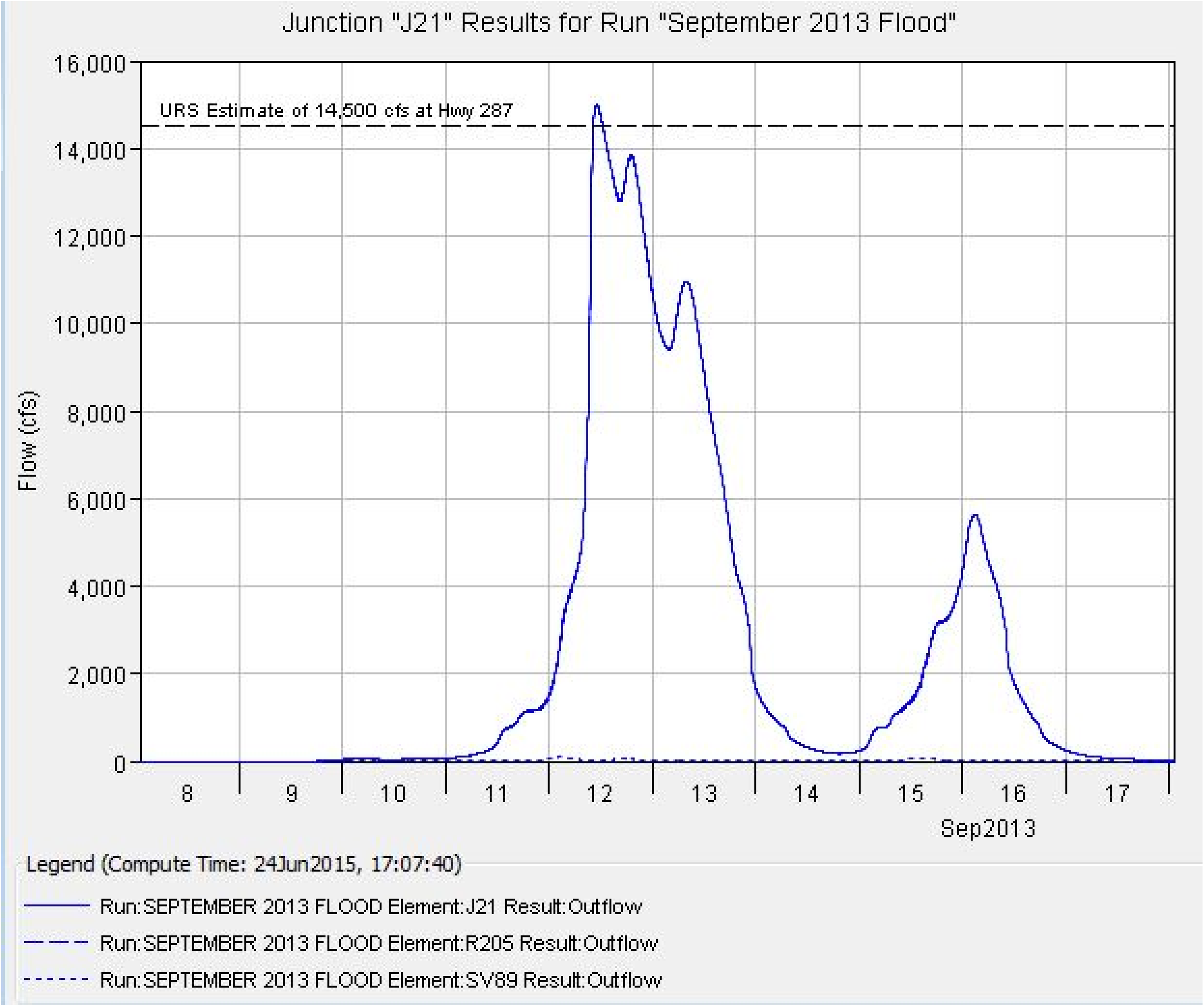
Appendix D.9 (continued)

St. Vrain Creek – SVLONGCO Gage near Hover Road



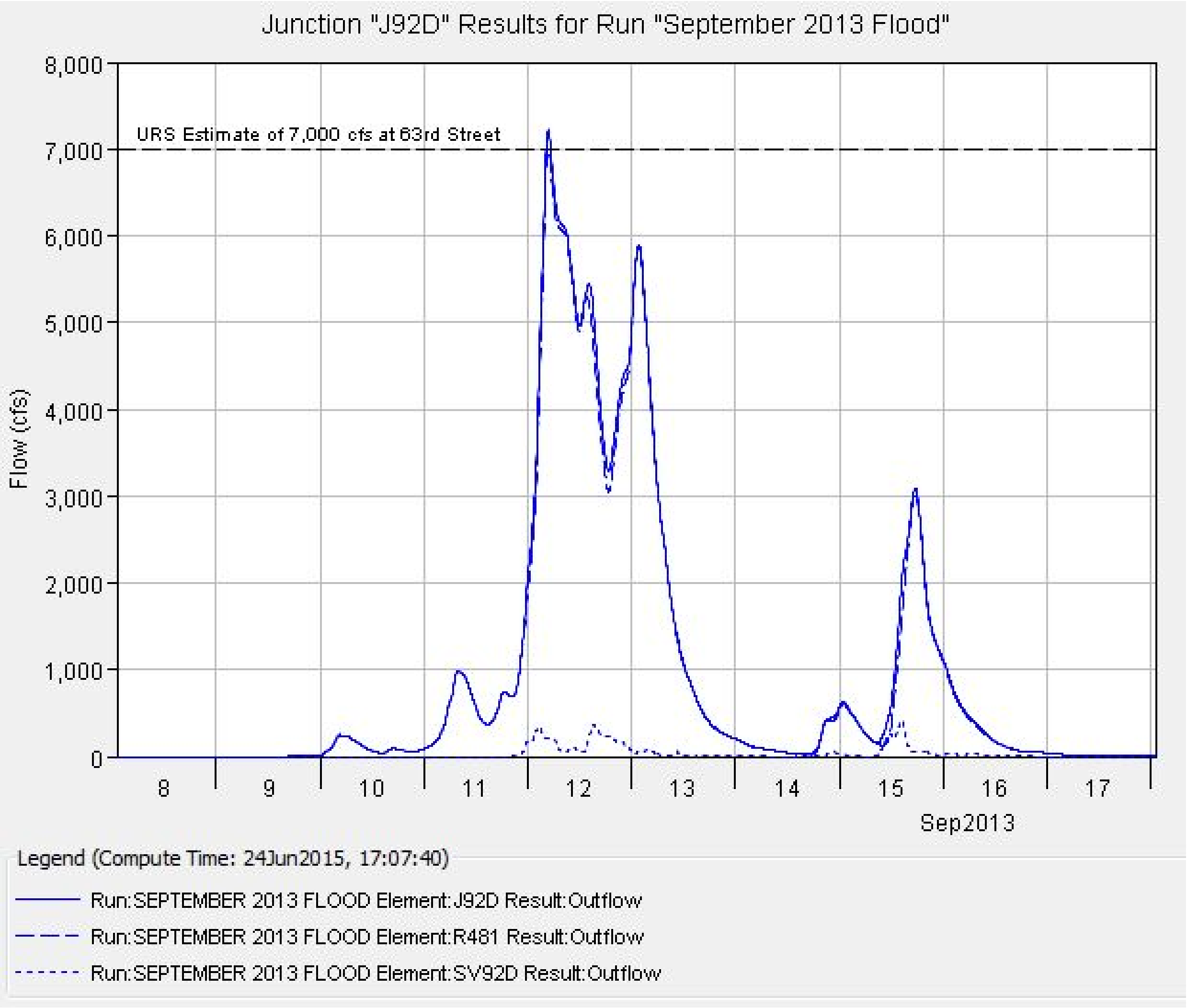
Appendix D.9 (continued)

St. Vrain Creek – Highway 287



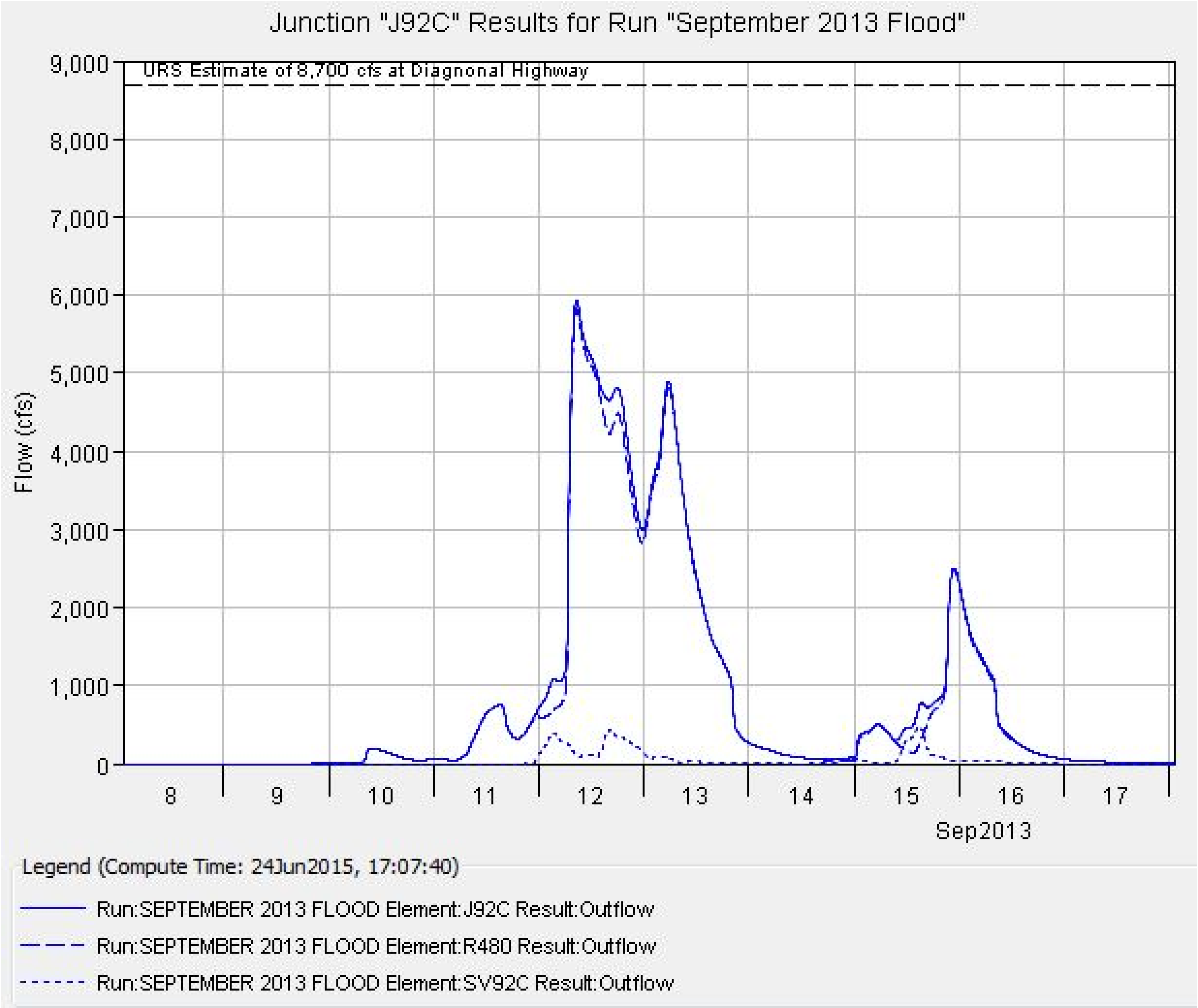
Appendix D.9 (continued)

Lefthand Creek – 63rd Street



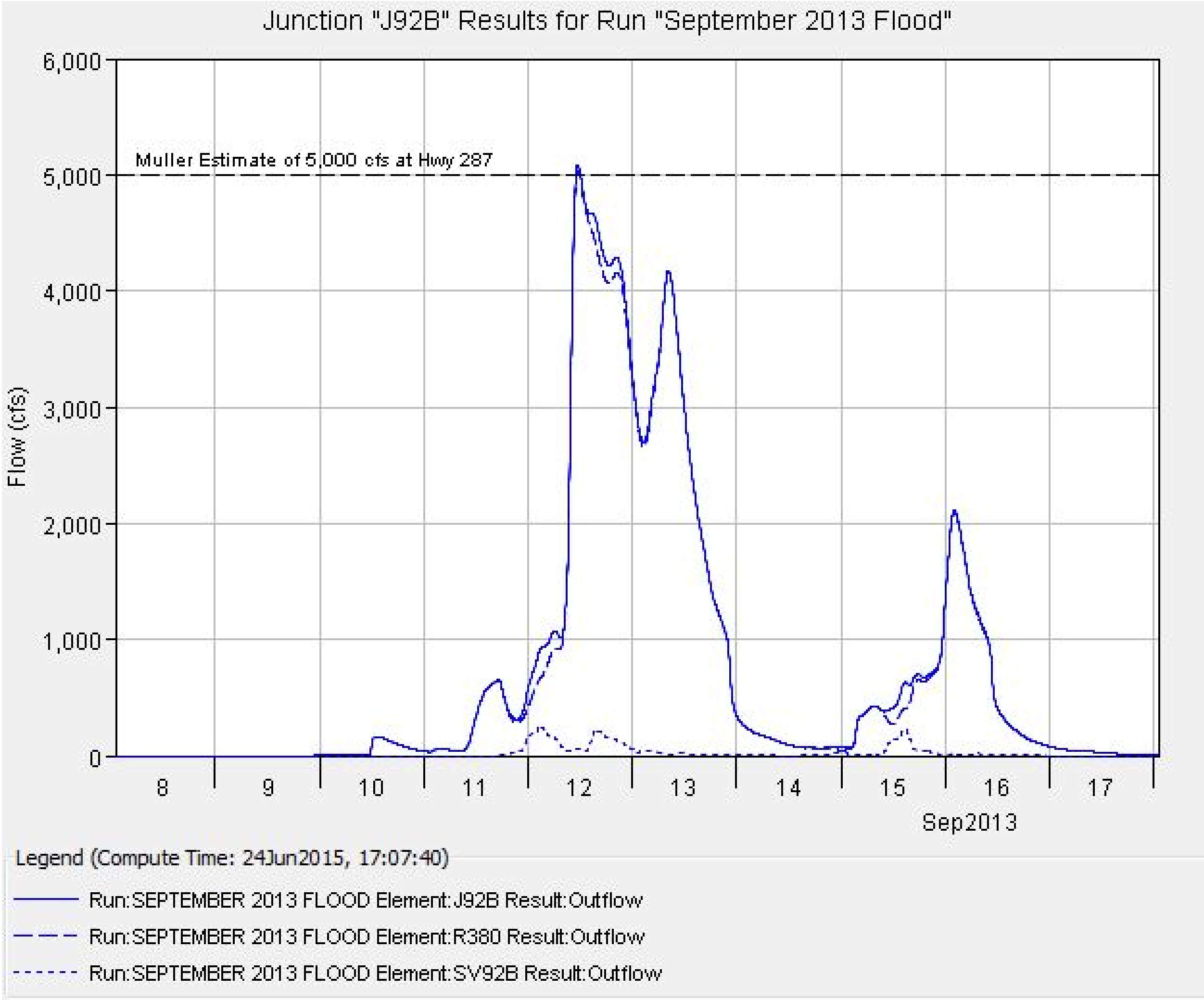
Appendix D.9 (continued)

Lefthand Creek – Diagonal Highway



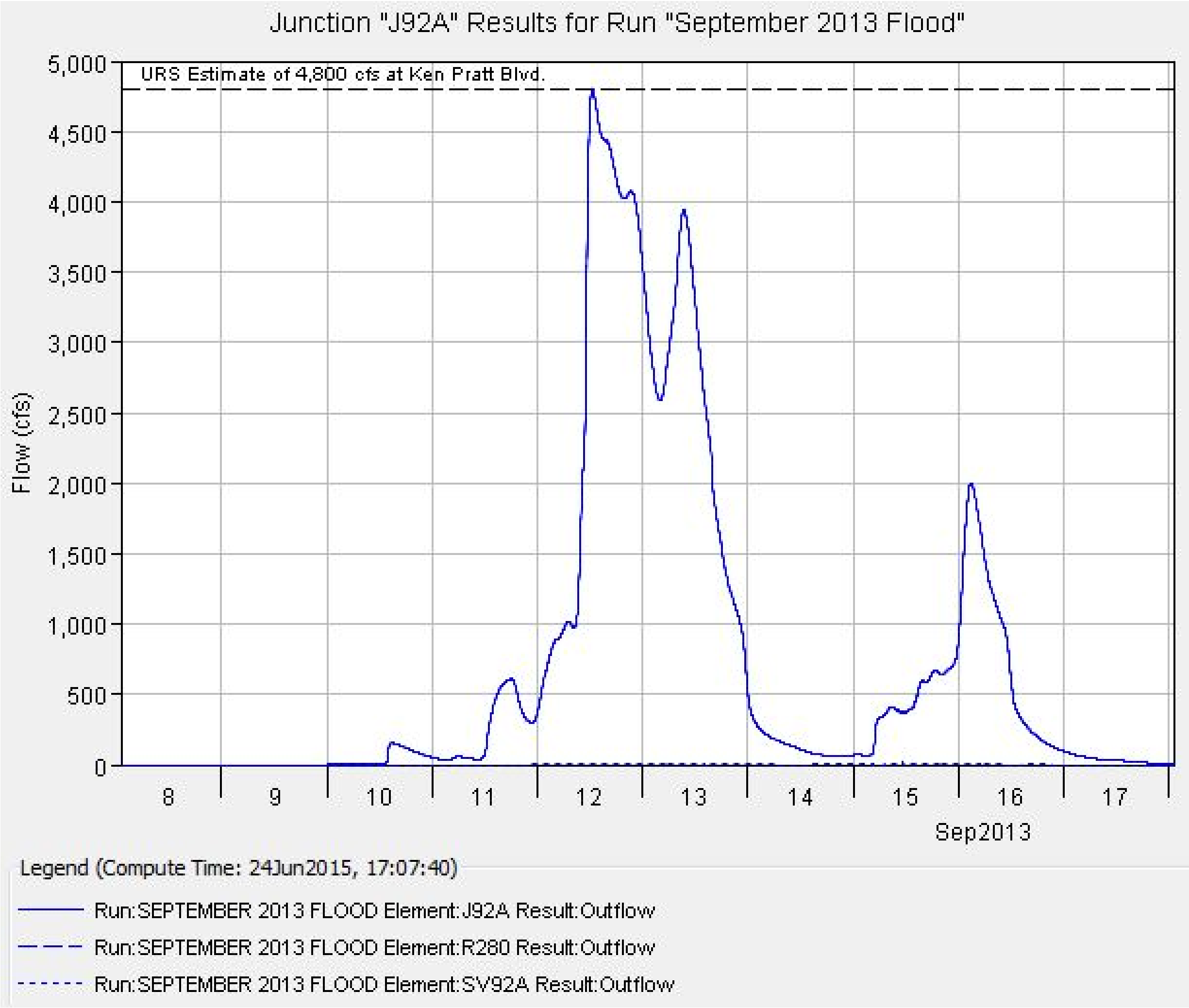
Appendix D.9 (continued)

Lefthand Creek – Highway 287



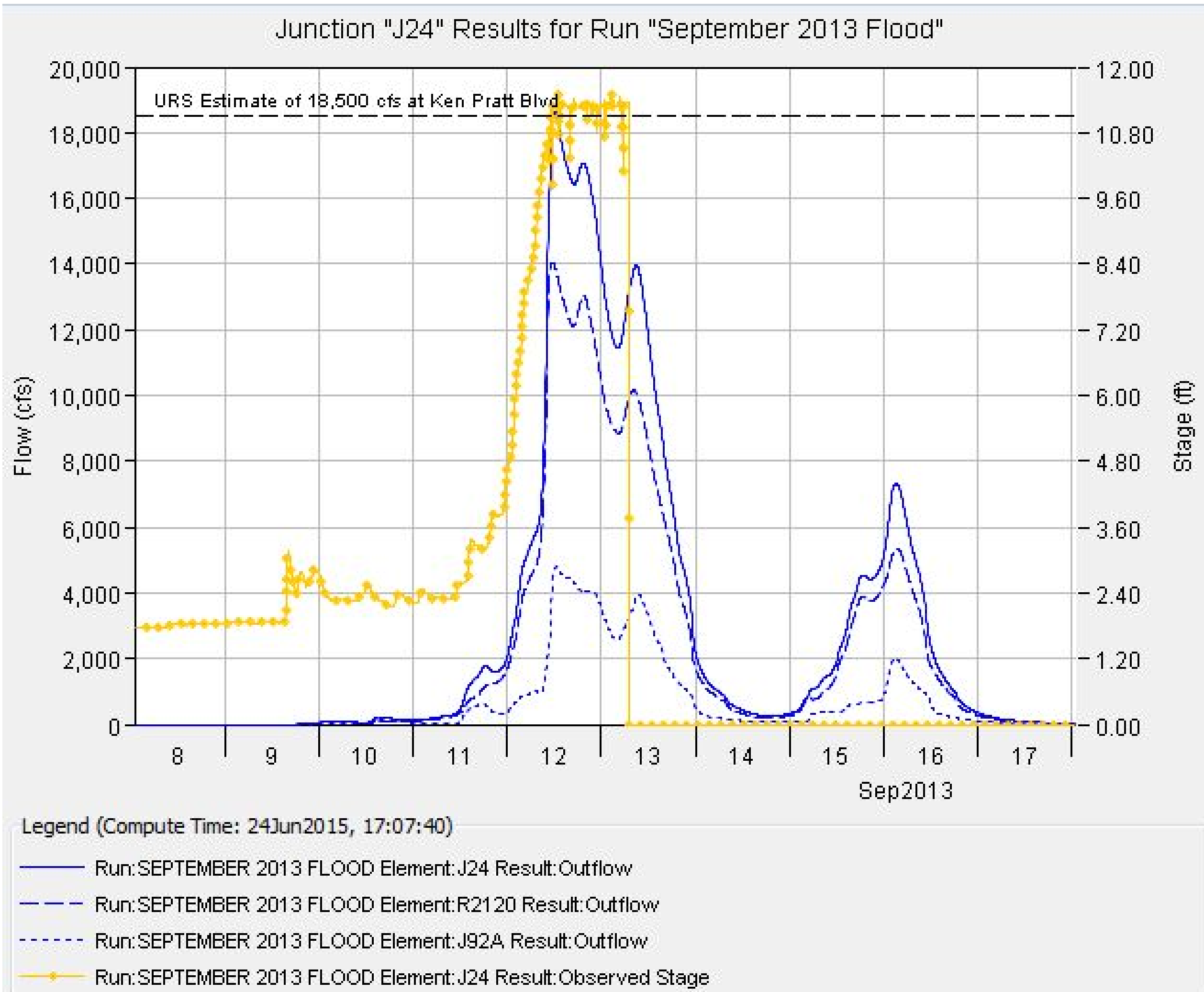
Appendix D.9 (continued)

Lefthand Creek – Ken Pratt Boulevard (Hwy 119) near confluence with St. Vrain Creek



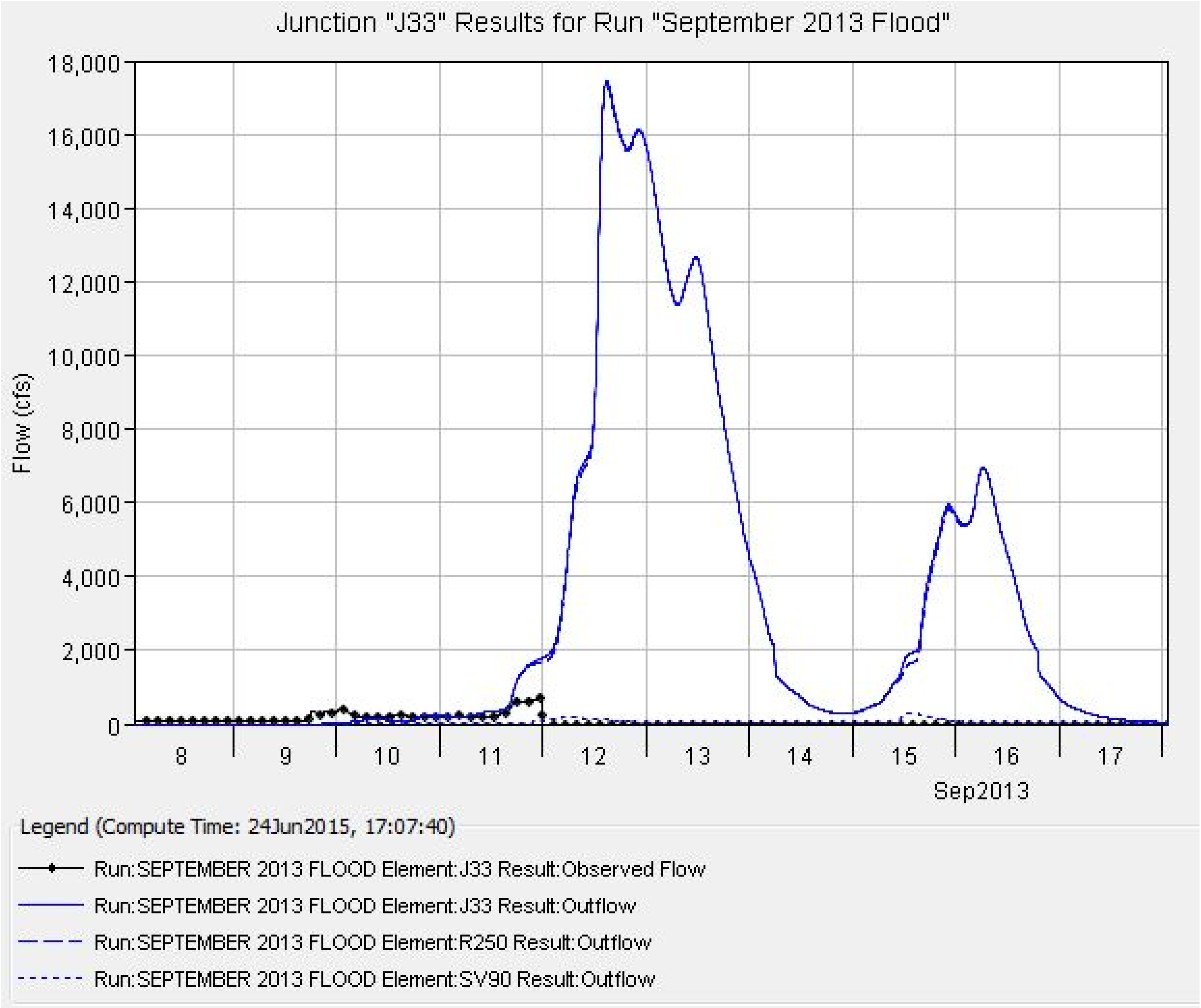
Appendix D.9 (continued)

St. Vrain Creek – Ken Pratt Boulevard (Hwy 119) below confluence with Lefthand Creek



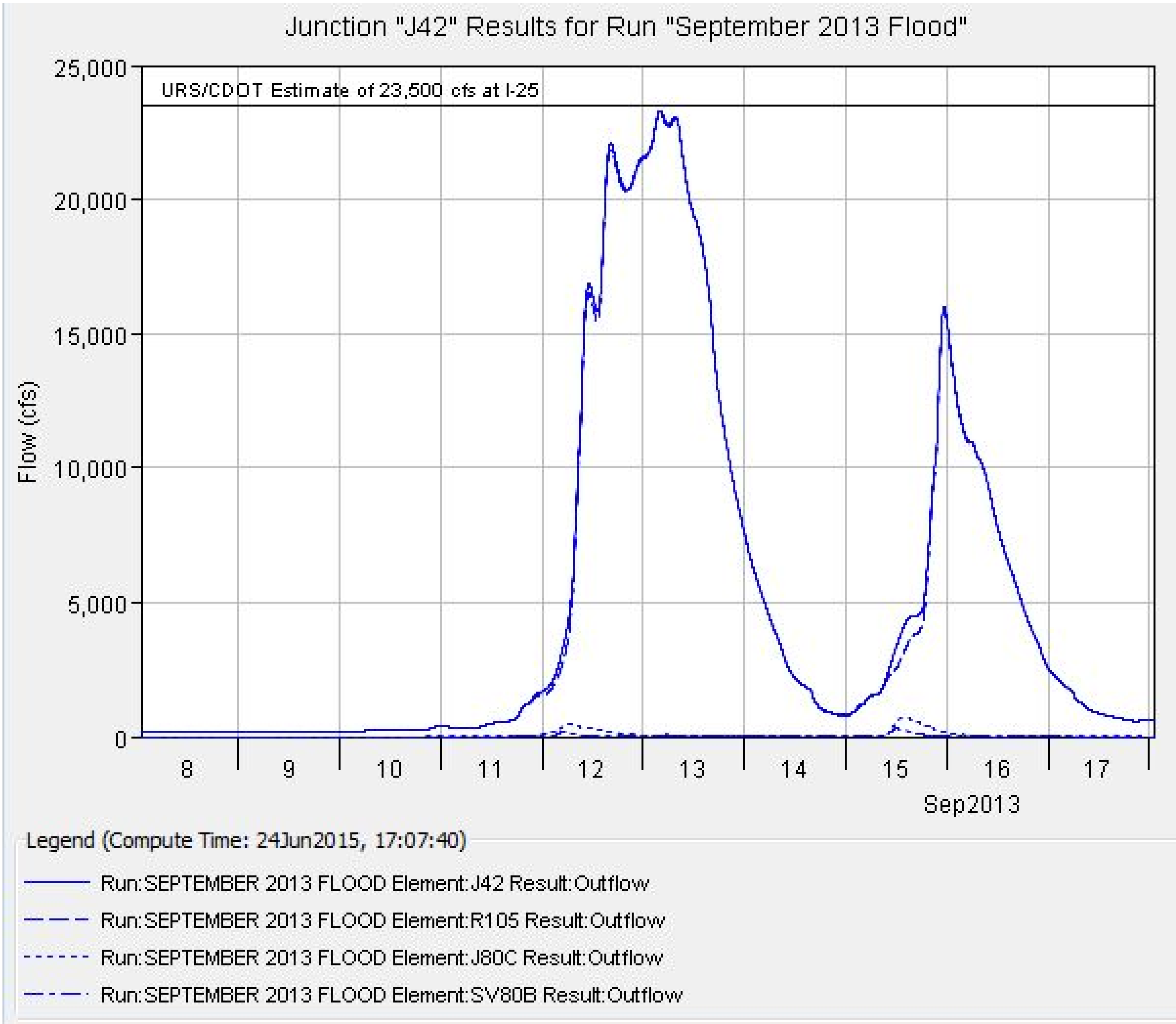
Appendix D.9 (continued)

St. Vrain Creek – SVCBLOCO Gage above confluence with Boulder Creek



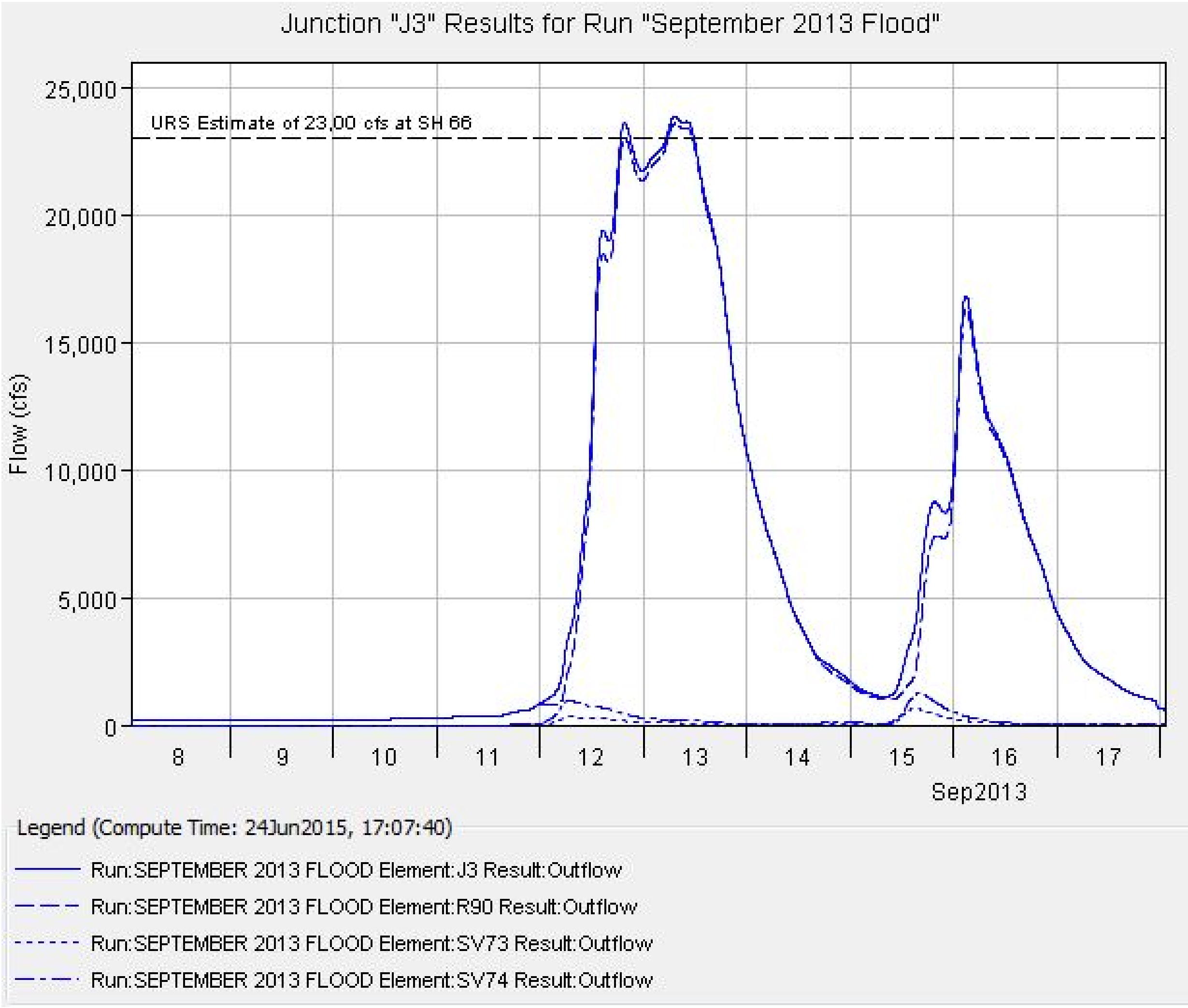
Appendix D.9 (continued)

St. Vrain Creek – I-25



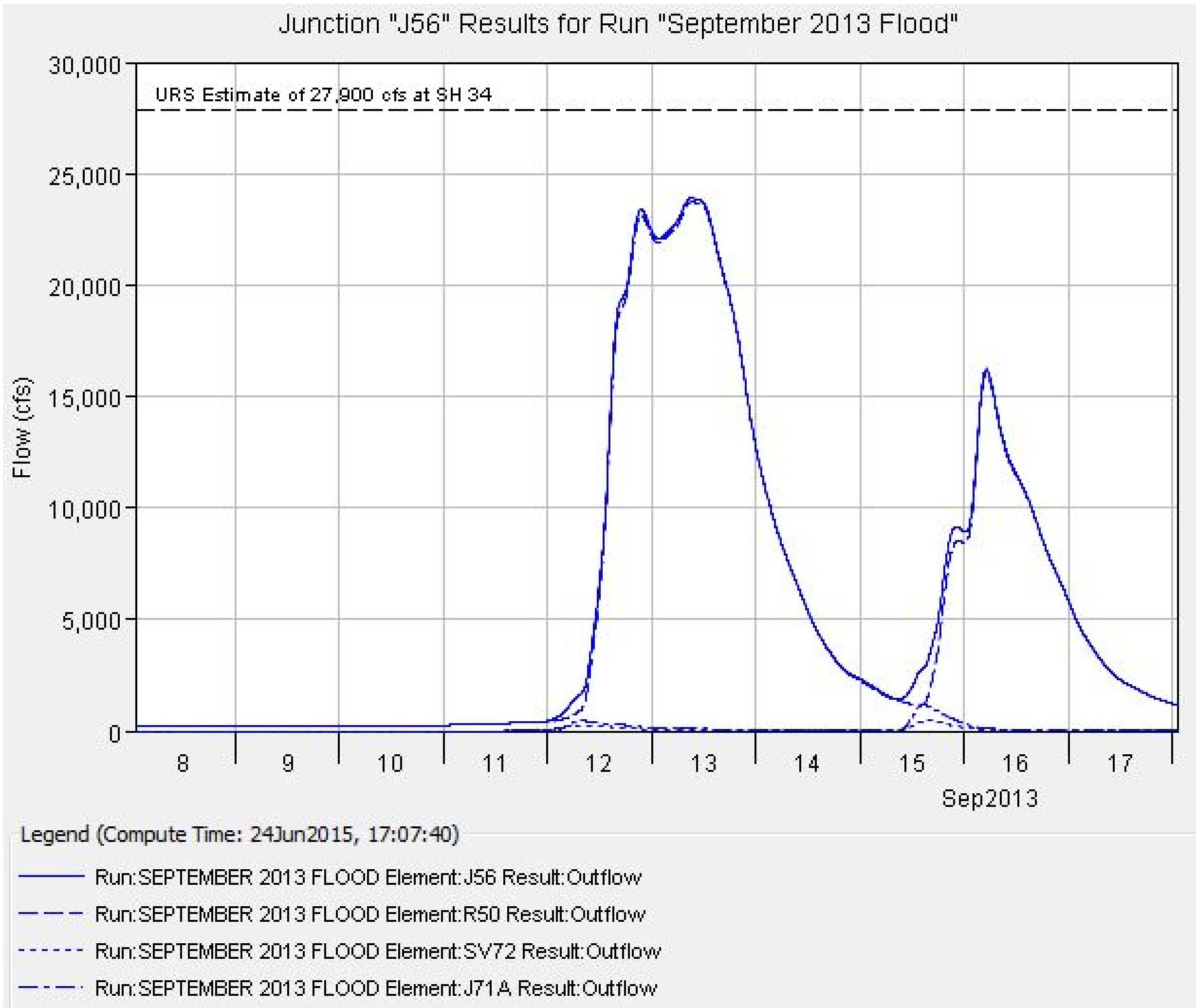
Appendix D.9 (continued)

St. Vrain Creek – State Highway 66



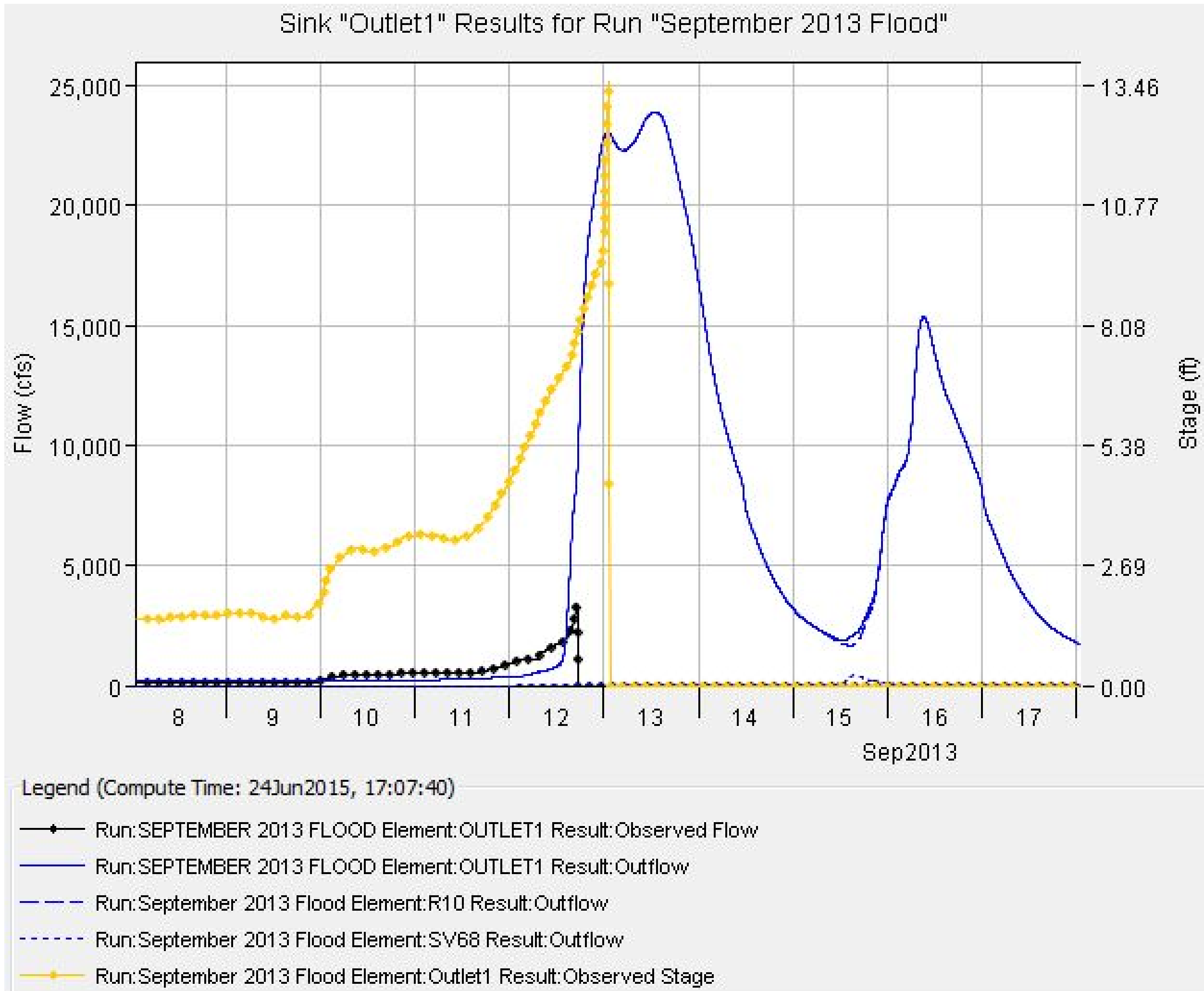
Appendix D.9 (continued)

St. Vrain Creek – County Road 34



Appendix D.9 (continued)

St. Vrain Creek – SVCPLACO Gage at Mouth



Appendix E

Boulder Creek Watershed Calibration

Boulder Creek 10-Day Model Calibration Documentation

PREPARED FOR:	Colorado Department of Transportation
COPY TO:	Colorado Water Conservation Board Jacobs Engineering Group
PREPARED BY:	CH2M HILL
DATE:	February 13, 2015
PROJECT NUMBER:	494613

This memorandum documents the development and calibration of a 10-day hydrologic model of Boulder Creek above its confluence with St. Vrain Creek, east of Longmont, CO. The hydrologic model was calibrated to estimates of peak discharge, runoff volume, and times-of-peak discharge along Boulder Creek across a 10-day period encompassing the September 2013 Front Range rainfall event. The hydrologic model and resultant output hydrograph were developed for use as input to the St. Vrain Creek calibrated hydrologic model (by others).

Hydrologic Analysis

Project Area Description

The Boulder Creek watershed was modeled in two phases as part of a broader post-flood hydrology analysis for the Colorado Department of Transportation (CDOT) and Colorado Water Conservation Board (CWCB). Phase 1 work and analysis report were completed in August 2014 (CH2M HILL, 2014) and included 129 drains of mountainous portions of the Boulder Creek watershed above the confluence of Boulder Creek and Fourmile Creek, near Orodell, CO. The Phase 1 watershed was subdivided and modeled with 44 subbasins. The Phase 2 watershed includes the entirety of the Boulder Creek watershed above its confluence with St. Vrain Creek, and together, the Phase 1 and Phase 2 watersheds total 447 square miles. The additional watershed area associated with Phase 2 was subdivided into 83 subbasins that cover mountainous terrain along South Boulder Creek, foothills terrain of the Front Range, and plains terrain east of the Front Range. **Figure 1** shows the location of the Boulder Creek watershed and the extent of Phase 1 and Phase 2 study areas.

Overall Modeling Approach

A hydrologic model was developed and calibrated to a 10-day period encompassing the September 2013 rainfall event. The United States Army Corp of Engineer's (USACE's) HEC-HMS version 3.5 (USACE, 2010) was selected to model the hydrologic conditions within the Boulder Creek watershed as the result of FEMA's approval of HEC-HMS to model single-event flood hydrographs (FEMA, 2013a) and the ability to incorporate complex calibration data and modeling parameters into the program. Hydrologic conditions unique to the September 2013 event (such as measured rainfall) were used to calibrate model parameters to match modeled peak discharges, timing, and volumes to observations collected during the September 2013 event. **Figures 2a through 2f** depict the HEC-HMS model components for Phase 2 of the Boulder Creek hydrologic model.

Calibration Data

Data used to calibrate the Boulder Creek hydrologic model included estimates of peak discharge, timing of peak discharge, and total runoff volume. The following peak discharge estimates were used in the calibration: critical-depth method estimates by Applied Weather Associates (AWA) and its subconsultant, Bob Jarrett (Jarrett, in press); bridge hydraulic estimates by URS (URS, in press); estimates reported by the

CWCB and Wright Water Engineers (WWE, 2014) performed for the City of Boulder; and gage measurements by the United States Geological Survey (USGS), Colorado Division of Water Resources (CDWR), and Urban Drainage and Flood Control District (UDFCD). Peak discharge estimates and locations along Boulder Creek are provided in **Table 1**; peak discharge estimates and locations along Boulder Creek tributaries are provided as **Table A-1** in **Appendix A**. Physical locations of both Boulder Creek and tributary peak discharge estimates are identified in **Figure 1**.

TABLE 1
Boulder Creek Peak Discharge Estimates and Locations

Model Phase	Site Description	Calibration Source	Peak Discharge (cfs)
Phase 1	Boulder Creek near Orodell, CO	Jarrett, in press	2,020
		CDWR Gage	1,720
Phase 2	Boulder Creek at Broadway	CDWR Gage	4,965
Phase 2	Boulder Creek at 28th Street	CWCB	5,300
Phase 2	Boulder Creek at Valmont Road	URS, 2015	5,700
Phase 2	Boulder Creek at 75th Street	USGS Gage	8,400
Phase 2	Boulder Creek at US Highway 287	URS, 2015	9,000
Phase 2	Boulder Creek at Mouth (St. Vrain Creek)	USGS Gage	8,910 [a]

[a] Daily average

cfs = cubic feet per second

Timing of peak discharges along principal waterways were estimated from the following sources: discharge measurements at USGS stream gages, discharge measurements at CDWR stream gages, and stage measurements at UDFCD stream gages. Locations and associated timing of peak discharges for Boulder Creek are provided in **Table 2**; locations and times-of-peak discharge for tributary stream are provided in **Table A-2** of **Appendix A**.

TABLE 2
Boulder Creek Time-of-Peak Discharge Measurements and Location

Model Phase	Site Description	Calibration Source	Time-of-Peak Discharge
Phase 1	Boulder Creek near Orodell, CO	CDWR Gage	9/12 23:30
Phase 2	Boulder Creek at confluence with Fourmile Creek	UDFCD Gage	9/12 22:48
Phase 2	Boulder Creek at Broadway	CDWR Gage	9/12 22:30
Phase 2	Boulder Creek at 75th Street	USGS Gage	9/13 02:50
Phase 2	Boulder Creek at Mouth (St. Vrain Creek)	UDFCD Gage	9/13 05:30

Volume estimates were available at stream gages that remained fully operational throughout the September 2013 rainfall event; volume estimates at these stream gages are provided in **Table 3**.

TABLE 3
Boulder Creek Runoff Volume Measurements

Model Phase	Site Description	Calibration Source	Volume (acre-feet)
Phase 1	Boulder Creek near Orodell, CO	CDWR	7,515
Phase 2	Boulder Creek at Broadway	CDWR	21,549
Phase 2	Boulder Creek at 75th Street	USGS	32,840
Phase 1	Middle Boulder Creek at Nederland, CO	CDWR	2,714
Phase 2	South Boulder Creek near Pinecliffe, CO	CDWR	5,554
Phase 2	South Boulder Creek below Gross Reservoir	CDWR	1,910

Subwatershed Areas

The Boulder Creek Study area upstream of the confluence with St. Vrain Creek was delineated using a total of 127 subbasins; subbasin areas are provided in **Table A-3** in **Appendix A**. In general, Phase 2 subbasins followed subbasin delineations used in the South Boulder Creek Climatology and Hydrology Report (HDR, 2007) and UDFCD's Electronic Data Management Map (UDFCD, 2014). Minor adjustments to subbasins used in the aforementioned studies were made based on review of USGS topography maps and to coincide subbasin boundaries with locations of calibration data or Federal Emergency Management Agency (FEMA) discharge summary locations. Subbasins were named in accordance with abbreviations of the FEMA-studied feature they first drained to [1], e.g., "MBC" for Middle Boulder Creek. In general, Phase 1 subbasins (prefixes MBC, NBC, FMC and subbasins BC-2A to BC-6B) were composed of mountainous terrain. Phase 2 subbasins were more variable; in general, most Boulder Creek tributaries originated in mountainous terrain and flowed through more developed plains regions before discharging to Boulder Creek.

During model development, it was recognized that due to the embankments separating the gravel pits from Boulder Creek, rainfall on the gravel pits adjacent to Boulder Creek did not reach Boulder Creek as a hydrologic response such that the gravel pits effectively acted as sinks. To avoid overestimating runoff potential of subbasins with a significant number of such gravel pits, "water" land cover units were not included in the area or CN calculations for subbasins with a significant area covered by gravel pits (BC-10, BC-11, BC-12B, BC-13, BC-16, BC-17, BC-18A, BC-20, BC-21A, BC-21B, BC-22, and CC-3). As a result of adjusting model parameters for disconnected gravel pits, the effective drainage area of the Boulder Creek watershed was reduced by 3.1 square miles.

Rainfall

AWA provided recorded rainfall data for the September 2013 storm event in 5-minute intervals from 1 a.m. on September 8, 2013, to 1 a.m. on September 18, 2013 (AWA, 2014). Individual rainfall hyetographs were generated for each subbasin using weighting techniques to transfer precipitation gage measurements collected during the event to the centroid of each subbasin. The total 10-day rainfall for each Phase 2 subbasin is illustrated in **Figure 3**. In general, measured rainfall was greatest along the foothills with 10 to 20 inches of rain observed along South Boulder Creek below Gross Reservoir and foothills tributaries, e.g., Fourmile Canyon Creek. Measured rainfall in the upper mountains (above Gross Reservoir) and east of 75th Street received considerably less rain, on the order of 6 inches.

[1] BC = Boulder Creek; BCC = Bear Canyon Creek; CC = Coal Creek; DC = Dry Creek No. 3; FCC = Fourmile Canyon Creek; FMC = Fourmile Creek; MBC = Middle Boulder Creek; NBC = North Boulder Creek; RC = Rock Creek; SBC = South Boulder Creek; SC = Skunk Creek; TCC = Twomile Canyon Creek; WC-1 = Wonderland Creek

Loss Method

Consistent to the Phase 1 hydrologic model, the NRCS (formerly SCS) method was selected to convert input rainfall to infiltration losses and runoff. NRCS's Technical Release 55: Urban Hydrology for Small Watersheds ("TR-55," NRCS, 1986) and engineering judgment were used to develop CNs for each subbasin. TR-55 provides CNs for a given land cover description and hydrologic soil group (a measure of the infiltration capacity of the underlying soil alone). Land cover was delineated using the National Land Cover Dataset (USGS, 2006) to identify forests, barren ground, urbanized areas, wetland, etc., across the subbasins on a 100-foot by 100-foot grid. After comparison to recent aerials, the land cover was adjusted in urbanized areas to account for development since 2006. Delineation of hydrologic soil groups was accomplished using the USDA's Web Soil Survey (USDA, 2013). The two overlapping datasets were then joined by intersecting the two datasets such that each land cover unit was further subdivided by hydrologic soil group. These results were then exported to Microsoft® Excel® where a CN was applied for each unique land cover condition and hydrologic soil group using engineering judgment to correlate observed land cover conditions with a representative land cover description provided in TR-55. Microsoft® Excel® was then used to area-weight these results, per TR-55 methodology, to estimate a single, representative CN for each subbasin.

To provide flexibility in model calibration and to incorporate observations that several NLCD land use categories were applied to a diverse range of land condition (e.g., "Scrub/shrub" in the upper portions of South Boulder Creek was observed to be "greener" alpine valley vegetation and trees, whereas in lower elevations areas it was observed to be "drier" grasses interspersed with some sagebrush), three distinct "land use zones" were developed to differentiate major changes in land use category based on aerial imagery. In general, these "land use zones" were representative of high-elevation mountains, foothills, and plains.

Initial calibrations of CN suggested that direct application of TR-55 CNs overestimated runoff in the Boulder Creek watershed, even when low-runoff potential "good" conditions were applied and adjusted to Antecedent Moisture Condition I (dry conditions). Recognizing that the NRCS infiltration methodology was developed for single-event, 24-hour storms, and that the modeled infiltration capacity based on TR-55 methodology would likely be exceeded during the 10-day September 2013 event, 24-hour CNs provided in TR-55 were adjusted to 10-day CNs per guidance in NRCS's TR-60: Earth Dams and Reservoirs (NRCS, 2005). The adjustment of CN to 10-day values was applied only to subbasins where more than 6 inches of rain were observed, as recommended in TR-60.

Unit Hydrograph

Snyder's Unit Hydrograph was used to transform runoff volume to an outflow hydrograph. The Snyder's Unit Hydrograph was used due to its acceptance in the Colorado Floodplain and Stormwater Criteria Manual (CWCB, 2008). The shape of the Snyder unit hydrograph is controlled by two factors: a peaking factor, C_p, which typically ranges from 0.4 to 0.8, and the lag time representative of the time elapsed between the centroid of a hyetograph and the peak of resultant outflow hydrograph. Lag time was estimated using the following equation (Equation CH9-511 provided in the Colorado Floodplain and Stormwater Criteria Manual (CWCB, 2008):

$$TLAG = 22.1\, K_n * \left(\frac{L * L_c}{\sqrt{S}}\right)^{0.33}$$

Where K_n is the roughness factor for the basin channels, L is the length of longest watercourse, in miles, Lc is the length along longest watercourse measured upstream to a point opposite the centroid of the basin, in miles, and S is the representative slope of the longest watercourse, in feet per mile. Physical parameters were estimated using ArcHydro tools in ArcGIS to analyze the National Elevation Dataset (NED) digital elevation model (USGS, 2013a). Initial estimates of the K_n parameter were assigned values between 0.05 and 0.15 depending on the landuse along the flowpath, per Table CH9-T505 of the Colorado Floodplain and Stormwater Criteria Manual (CWCB, 2008). Initial estimates of C_p varying from 0.4 for undeveloped subbasins to 0.60 for developed subbasins were assigned. Both C_p and K_n were adjusted during the calibration process, as described in subsequent sections. In general, estimated lag times of mountainous and foothills subbasins were much less than similarly-sized plains and upper mountain subbasins. Lag times for each individual subbasin is provided in **Table A-4** in **Appendix A**.

Channel Routing

The Muskingum-Cunge routing methodology was selected to route inflow hydrographs along basin streams because of its solution of the continuity and momentum equations to estimate lag time and flow attenuation; thus, the Muskingum-Cunge method is based on channel hydraulics including channel roughness, cross section, and slope. Eight-point cross sections were used to model the channel cross section shape because the 8-point cross section allowed for the incorporation of channel floodplains that convey a significant portion of high-flows. Eight-point cross sections were derived using GIS and manually transposed to the hydrologic model. The NED 1/3 arc-second data (USGS, 2013a) was utilized to develop cross sections along South Boulder Creek upstream of Rollinsville, Twomile Canyon Creek, Wonderland Creek, Skunk Creek, and Dry Creek; 2013 post-flood LiDAR (FEMA, 2013b) was used to develop cross-sections along the remaining stream corridors. A single cross section was selected for each reach based on visual identification of a representative cross section, erring slightly towards flatter, wider reaches that are likely to provide the majority of floodplain storage and flow attenuation. The location of the Phase 2 model reach locations are provided in the connectivity maps, **Figures 2a through 2f**, and the model eight-point cross sections are provided in **Figure 4a through 4h**.

Reservoir Routing

Four major reservoirs are located within the Boulder Creek watershed: Barker Reservoir on Middle Boulder Creek, Gross Reservoir on South Boulder Creek, Baseline Reservoir on Dry Creek (with diversions from South Boulder Creek), and Valmont Reservoir on South Boulder Creek. All four reservoirs were considered in the development and calibration of the hydrologic model due to their storage of a significant portion of the inflow during the September 2013 storm event. At Barker Reservoir, a peak inflow of approximately 400 cfs on September 13 was measured by the upstream CDWR gage, whereas flow releases, as measured by the City of Boulder, from Barker Reservoir were approximately 4 cfs until the emergency spillway activated several days later on September 15. Although water supply reservoirs are typically not modeled in hydrologic models, the decision was made to account for Barker Reservoir in the calibrated model because the 11 feet of storage available in the reservoir immediately prior to the storm event (according to City of Boulder measurements) was used to store the majority of the September 13 peak flow, thus reducing downstream discharges. Neglecting this available storage and routing the entirety of the inflow hydrograph to the downstream reaches would result in the underestimation of calibration parameters because modeled discharges would have included discharge from Middle Boulder Creek that did not actually occur and, thus, decrease the runoff contribution from other sub-watersheds. Therefore, to account for the storage provided by Barker Reservoir, reservoir releases in the hydrologic model were controlled according to hourly outflow discharges from September 11 to September 25, 2013, as provided by the City of Boulder (City of Boulder, 2013).

Similar to Barker Reservoir, Gross Reservoir also provided significant attenuation of the September 2013 storm event: a peak discharge in excess of 800 cfs was observed at CDWR's Pinecliffe gage located 2.5 miles upstream of Gross Reservoir, whereas flows measured by the USGS gage downstream of Gross Reservoir, depicted in **Figure 5**, did not exceed 300 cfs (of which 171 cfs was a direct release from Gross Reservoir (Denver Water, 2013), see **Table A-5** in **Appendix A**). To account for the storage and attenuation provided by Gross Reservoir, reservoir releases in the hydrologic model were controlled according to the September 2013 daily flow release scheduled provided by Denver Water (Denver Water, 2013) and included as **Table A-5** in **Appendix A**. Review of Denver Water flow release records and the downstream CDWR gage suggests that the Gross Reservoir spillway did not engage during the 10-day analysis period.

While Baseline Reservoir is an off-channel reservoir, personnel communications with the City of Lafayette indicated that overland flooding from South Boulder Creek flowed, uncontrolled, into Baseline Reservoir (Bradley Dallam, PE, personal communication, June 13, 2014). Measurements of gage height, reservoir contents, inflows, and outflows at Baseline Reservoir provided by the City of LaFayette, and included as **Table A-6** in **Appendix A** (Baseline Reservoir Company, 2013), support this notion as the reservoir rose several feet between September 11th and September 14th even though no controlled inflows were measured and controlled releases were recorded. The possibility that rainfall caused the reservoir to fill was also ruled out, as the drainage area to Baseline Reservoir contains little area outside of the reservoir itself and over the entire 10-day period, a maximum of 21 inches fell anywhere in the Boulder Creek watershed (i.e., the maximum rise in reservoir gage height over the 10-day period due to rainfall, assuming no outflows, would be 21 inches). Furthermore, two-dimensional hydraulic modeling of South Boulder Creek indicating flow from South Boulder Creek to Baseline Reservoir (HDR, 2007) provides further evidence that overland flooding from South Boulder Creek to Baseline Reservoir occurred during September 2013. To account for flow from South Boulder Creek to Baseline Reservoir, results from HDR's 2007 two-dimensional modeling of South Boulder Creek were used to develop an inflow-diversion relationship at South Boulder Road that was incorporated into the HEC-HMS model to divert flow from South Boulder Creek to Baseline Reservoir.

Based on the observation that the Leggett-Hillcrest-Valmont Reservoir Complex ("Valmont Reservoir") did not flood to South Boulder Creek per the City of Boulder's Urban Flooding Extents Map (City of Boulder, 2014), Valmont Reservoir was effectively removed from the model by setting the initial abstraction for the watershed draining to Valmont Reservoir (SBC-21) to 18 inches, a value greater than the observed rainfall. This approach effectively eliminated runoff from the subbasin but allowed efficient re-incorporation of SBC-21 into a future predictive hydrologic model where Valmont Reservoir, as a cooling reservoir for Xcel Energy, could not be relied upon to intercept the entirety of runoff draining to it.

Transbasin Transfers

Two transbasin inflows and one transbasin outflow were incorporated into the hydrologic model: Moffat Tunnel in the upper portion of South Boulder Creek, Boulder Creek Supply Canal from Boulder Reservoir, and South Platte Supply Canal downstream of the Boulder Creek at 75th gage, respectively. Gage measurements for each transbasin transfer were available from CDWR and subsequently incorporated into the hydrologic model as a source or diversion, as appropriate.

Calibration Process

Model calibration is the iterative process of adjusting model parameters so that simulated results match real-world observations (measurements). Model calibration requires careful consideration of which modeling parameters are best considered "fixed" and which are most appropriately adjusted to avoid the manipulation of parameters beyond physical reality to achieve desired results. For example, modeled discharges may be "calibrated" to measured discharges by increasing basin roughness parameters to an unreasonably high value that results in an excessive time lag. While the model may be "calibrated" computationally, it would not be calibrated realistically because careful review of the calibrated parameters would suggest that the resultant time lags are not consistent with physical processes. In a similar sense, topographically-derived parameters including the slope of routing elements and subbasin area, were considered fixed – while these parameters affect the model results, there is little justification to change their value short of re-defining the watershed subbasins and flowpaths. Giving consideration to the empirical or physical nature of model parameters, it was determined that adjustment of the following parameters was most justified to calibrate the hydrologic model: Manning's roughness coefficient, subbasin roughness factor, K_n, Snyder's Peaking factor, C_p, CN, and initial abstraction.

After identifying parameters to be calibrated, model calibration should also consider the sensitivity of model results to these parameters – special attention should be paid to "sensitive" parameters that have large effect on model results. As various parameters were adjusted during the calibration process to match observations, the sensitivity of the model results to various parameters was noted. As a result of this process, this assessment of the sensitivity of the model results to the following parameters was made:

- **Subbasin Roughness Factor, K_n:** Of the calibration parameters, subbasin roughness factor had the greatest impact on modeled times-of-peak discharge. Subbasin roughness factor also had a significant impact on modeled peak discharges, however these impacts were generally mediated by concurrent decreases in Snyder's peaking factor and/or CN to calibrate peak discharges.
- **Manning's Roughness Coefficient, n:** Within smaller drainages, e.g., Fourmile Canyon Creek, drastic changes in Manning's roughness coefficient had a minor effect on estimated peak discharge (± 5 percent) and time-of-peak discharge (± 15 minutes). On the watershed scale, Manning's roughness coefficient had a moderate impact on estimated peak discharges as the non-concurrent or concurrent timing of tributary peak discharges could affect mainstem peak discharges as much as ± 20 percent.
- **Snyder's Peaking Factor, C_p:** No effect on modeled runoff volume; moderate effect on modeled peak discharge and time-of-peak. In general, decreased Snyder's peaking factor lengthened the duration of the hydrograph, decreased peak discharge, and resulted in later time-of-peak discharge. However, individual manipulation of Snyder's peaking factor by subbasin (dependent on subbasin topology and hydrologic conditions) could affect modeled time-of-peak discharge by as much as 2 hours as the impact of individual subbasins on downstream time-of-peak discharge increased or decreased.
- **CN:** One of two parameters that affected modeled runoff volume; greatest impact on modeled peak discharges. Also, in contrast to the Phase 1 24-hour model, CN had a significant impact on modeled tributary time-of-peak discharges due to significant and distinct rainfall bursts occurring on both the evening of September 11 and evening of September 12: at higher CNs, the modeled peak discharge occurred following the September 11th rainfall burst, whereas at lower CNs, the modeled peak discharge occurred following the September 12th rainfall burst. Aside from controlling the modeled peak-discharge causing rainfall burst, CN had little effect on the time-of-peak discharge (i.e., after calculating September 11th or September 12th rainfall burst as the peak-discharge causing event, there was little impact on timing.)
- **Initial Abstraction:** Moderate impact on modeled runoff volume; principal effect of adjusting initial abstraction was the modeled start of runoff with lower initial abstraction ratios resulting in earlier start of runoff. Adjustments of initial abstraction typically necessitated an opposite adjustment of CN and as such there was some minor correlation between initial abstraction and modeled time-of-peak discharge.

In general, calibration of the model followed the process outlined below:

1. Coarse-scale adjustment of CN to approximately match modeled runoff volume to runoff volume observed at undamaged stream gages.
2. As necessary if standard initial abstraction rate is not adequate, watershed-scale adjustment of initial abstraction value to initiate runoff at time runoff was observed at functioning stream gages.
3. Adjust subbasin roughness factor, K_n, and Manning's roughness coefficient, n, to match modeled times-of-peak discharge to observed times-of-peak discharge. To the extent possible, "global" rules were used to assign values, e.g., subbasins with a flowpath slope in excess of six percent were assigned a K_n of 0.08.
4. Adjust Snyder's peaking factor, C_p, to approximate modeled peak discharges to observed peak discharges. To the extent possible, "global" rules were used to assign values, e.g., subbasins with a flowpath slope less than one percent were assigned a C_p of 0.30.

5. Fine-scale adjustment (by land use and hydrologic soil group) of CN to improve calibration of modeled peak discharges while maintaining calibration of modeled runoff volume.
6. Review of modeled times-of-peak discharge and assessment of sensitivity to runoff from particular subbasins impacting modeled times-of-peak discharge.
7. Repetition of steps 2 to 6 until modeled results best achieved calibration targets, recognizing that some calibration targets were not attainable (e.g., significant downstream decrease in peak discharge along Coal Creek). In general, achieving calibration targets along Boulder Creek was prioritized over achieving calibration targets on tributary drainages, although consideration was given to both sets of targets.

Calibration Results

Comparisons of modeled peak discharges, times-of-peak discharge, and runoff volume to calibration targets along Boulder Creek are provided in **Table 4**, **Table 5**, and **Table 6**, respectively. In general, modeled peak discharge, runoff volume, and times-of-peak discharge along Boulder Creek met calibration targets to within a few percent of targets. To better assess calibration efforts at the Boulder Creek at Mouth gage which had an incomplete record of the event, manual discharge measurements and recorded stage measurements at the gage are compared to the modeled runoff hydrograph in **Figure 6**. In general, the modeled time-of-peak agrees well with the time-of-peak according to the gage's stage record. Similarly, the modeled hydrograph agrees well with field discharge measurements recorded on September 13th and September 16th.

While the modeled hydrograph did not compare favorably to the discharge recorded at the gage or the field discharge measurement made on September 12th, a review of field measurements at the USGS gage suggests that the September 12th field discharge measurement and gage rating curve prior to and including September 12th may not have been accurate. Review of the field notes for the September 12th, 11:40AM field measurement identify that this measurement was a poor quality measurement, water surface elevation changed 1.08 feet during the measurement, and the rating curve was adjusted 0.31 feet as a result (USGS, 2013b). Recognizing that a rapidly changing water surface height would induce significant error into the field-measured discharge, the uncertainty associated with the September 12th field discharge measurement was considered too great to use the September 12th field discharge measurement, related rating curve (which was revised by this measurement before a new rating curve wad developed on September 13th), and discharge record to review the hydrologic model calibration for the period prior to and including September 12th. This uncertainty may also explain the subsequent gap in the gage record (intentional omission of the recorded discharge due to uncertainty outside of acceptable limits). It should be noted that while the September 12th field measurement and gage's discharge record were not considered in the review of the hydrologic model calibration; review of USGS field measurement notes suggest that the September 13th and September 16th field discharge measurements were of higher quality (gage height variation recorded during subsequent field measurements was minimal) and suitable for comparison to the modeled hydrograph which agreed well with the higher-quality September 13th and September 16th field discharge measurements.

Comparisons of modeled peak discharges, times-of-peak discharge, and runoff volume to calibration targets along tributaries are provided in **Table A-7** and **Table A-8** of **Appendix A** and **Table 6**, respectively. Modeled peak times-of-peak discharge and runoff volume along tributaries generally matched calibration targets well. However, modeled peak discharges along tributaries, were generally less than the calibration target range (80 to 120 percent of observed peak discharge). Part of this discrepancy could be attributable to debris flows, avulsions, and flow bulking that were observed along several tributary streams (Wright Water Engineers, 2014) that would have skewed estimates of peak discharge. Evidence that supports this assumption is that unit discharge rates on many of these tributary systems were in excess of 250 cfs per square mile, whereas Boulder Creek varied from 17 cfs per square mile at Orodell to 37 cfs per square mile at Valmont Road. The scale of the hydrologic model may also explain some of the variance from calibration targets observed along tributary systems: the hydrologic model was developed to model high-flow hydrology along Boulder Creek, not necessarily within smaller tributaries where the importance of micro-scale hydrologic features not modeled, such as storm sewers and low-impact development, are more important to hydrologic processes. While it was possible to calibrate to observed peak discharges along tributary streams, to do so would have significantly over-calibrated peak discharges along Boulder Creek unless hydrologic parameters well outside of realistic values were assigned.

TABLE 4

Boulder Creek Comparison of 10 Day Modeled Discharges to Observed Discharges

HMS Node	Location	Drainage Area (mi^2) [a]	Observed Peak Discharge (cfs)	Modeled Peak Discharge (cfs)	% Difference
BC-J5	Boulder Creek near Orodell, CO (Jarrett, in press)	102	2,020	1,771	- 12.3%
	Boulder Creek near Orodell, CO (CDWR Gage)	102	1,720	1,771	+ 3.0%
BC-J7	Boulder Creek at Broadway	135	4,965	4,842	- 3.2%
BC-J8A	Boulder Creek at 28th Street	136	5,300	5,115	- 3.5%
BC-J9	Boulder Creek at Valmont Road	156	5,700	5,995	+ 39.4%
BC-J11A	Boulder Creek at 75th Street	307	8,400	8,457	+ 0.7%
BC-J13A	Boulder Creek at US Highway 287	331	9,000	9,004	± 0.0%
Outfall	Boulder Creek at St. Vrain Creek	447	N/E	13,890	- 15.4% [b]

N/E = No estimate

cfs = cubic feet per second

[a] Total watershed area, before subtracting out the area of gravel pits that were removed from the model.

[b] Based on comparison of 8,910 cfs daily average discharge at the gage, as reported by USGS, and modeled daily average discharge of 7,539 cfs

TABLE 5

Boulder Creek Comparison of 10 Day Modeled and Observed Times-of-Peak Discharge

HMS Node	Location	Observed Time-of-Peak Discharge	Modeled Time-of-Peak Discharge	Difference
BC-J5	Boulder Creek near Orodell, CO	9/12 23:30	9/12 23:25	- 5 min (early)
BC-J6	Boulder Creek at Fourmile Creek	9/12 22:48	9/12 22:35	- 13 min (early)
BC-J7	Boulder Creek at Broadway	9/12 22:14	9/12 22:40	+ 10 min (late)
BC-J11A	Boulder Creek at 75th Street	9/13 02:50	9/13 01:55	- 55 min (early)
Outfall	Boulder Creek at St. Vrain Creek	9/13 05:30	9/13 06:25	+ 55 min (late)

TABLE 6
Boulder Creek Comparison of 10-Day Modeled Runoff Volume to Observed Runoff Volume

HMS Node	Location	Observed Runoff Volume (acre-feet)	Modeled Runoff Volume (acre-feet)	% Difference
BC-J5	Boulder Creek near Orodell, CO	8,233	7,515	- 8.7%
BC-J7	Boulder Creek at Broadway	21,549	13,163	- 38.9%
BC-J11A	Boulder Creek at 75th Street	32,840	27,059	-17.6%
MBC-J5	Middle Boulder Creek at Nederland, CO	2,714	2,533	- 6.7%
SBC-J7	South Boulder Creek near Pinecliffe, CO	5,554	4,250	- 23.5%
SBC-J10A	South Boulder Creek below Gross Reservoir	1,910	2,003	- 4.6%

Calibrated curve number and initial abstraction values for modeled subbasins are provided in **Table A-3** of **Appendix A**. At the watershed-scale, the standard initial abstraction ratio of 0.20 accurately modeled the start of runoff in most cases; the headwater subbasin of Fourmile Canyon Creek was adjusted to 0.10 based on gage information and the 2010 Fourmile Canyon Fire that likely decreased infiltration capacity of the soils in this area. Observations of the calibrated CN suggest a low CN as compared to initial estimates of CN: CNs for urban areas typically had to be reduced commensurate to a level of imperviousness nearly half of the actual percent impervious and rural land use classifications were typically assigned as “good” condition.

Calibrated subbasin roughness values, K_n, peaking factor, C_p, and other unit hydrograph parameters for modeled subbasins are provided in **Table A-4** of **Appendix A**. Subbasin roughness factor, K_n, was found to be approximately proportional to the slope of the subbasin’s longest flow paths: subbasins with steeper flow paths were generally found to have lower subbasin roughness factors, which conforms to conceptual models of watershed runoff processes (steeper basins runoff quicker). Snyder’s peaking factor was also found to vary based on flowpath slope, with higher peaking factors associated with higher slopes (and lower subbasin roughness factors), which is consistent with literature discussions of Snyder’s peaking factor.

Calibrated model routing parameters are provided in **Table A-9** of **Appendix A**. Manning’s roughness values for channel routing elements were adjusted during the calibration process to match calibration targets. In general, calibrated Manning’s roughness coefficients ranged from 0.030 / 0.035 (channel / overbank) along improved drainageways (for example, Goose Creek) to values up to 0.10 / 0.15 in streams where landslides, debris flows, or flow bulking were observed (South Boulder Creek below Doudy Draw, Twomile Canyon Creek, and Fourmile Canyon Creek) or where numerous bridge crossings and obstructions likely caused backwater effects not otherwise accounted for by typical hydrologic routing methods (such as Boulder Creek through the City of Boulder).

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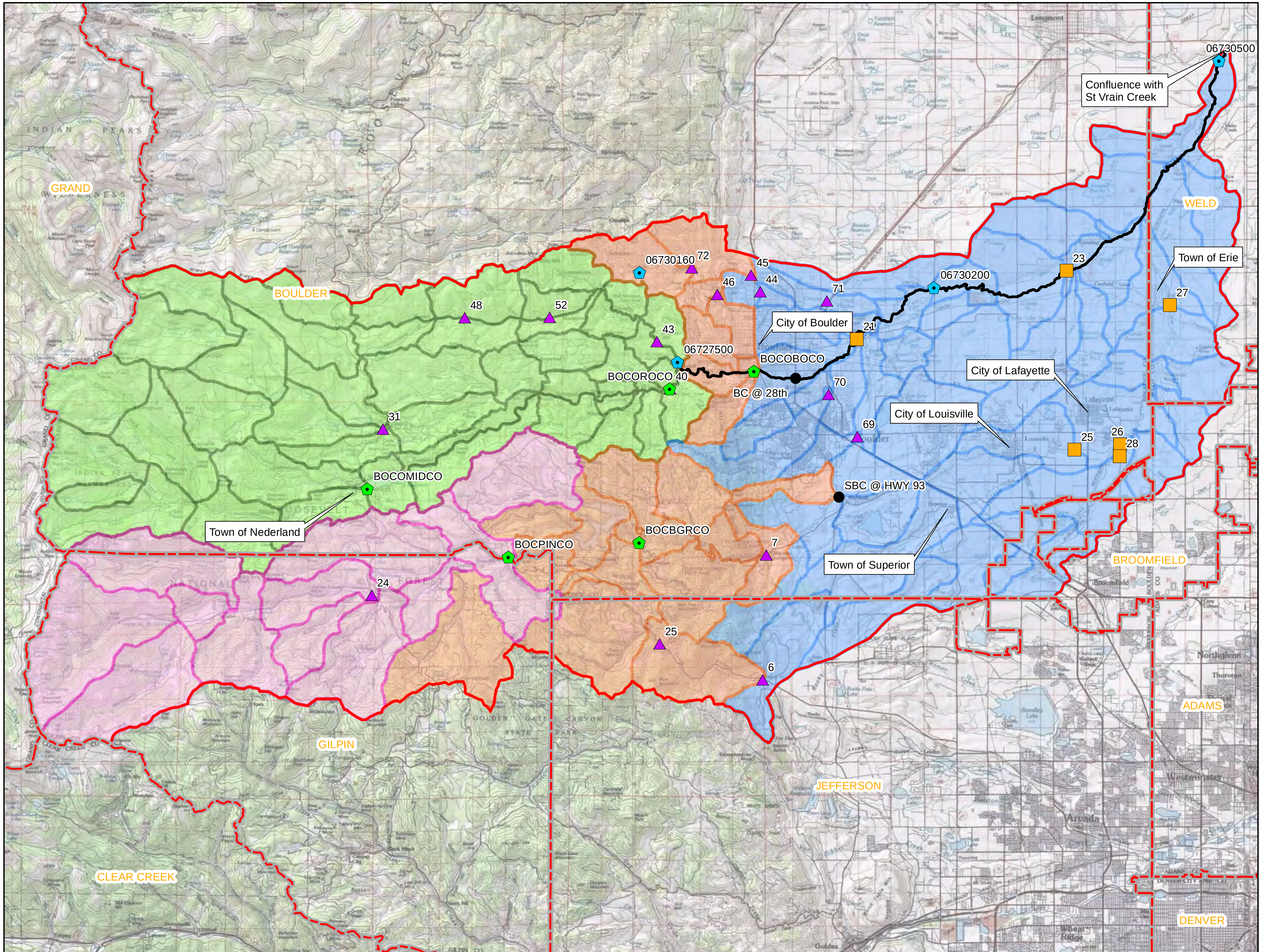
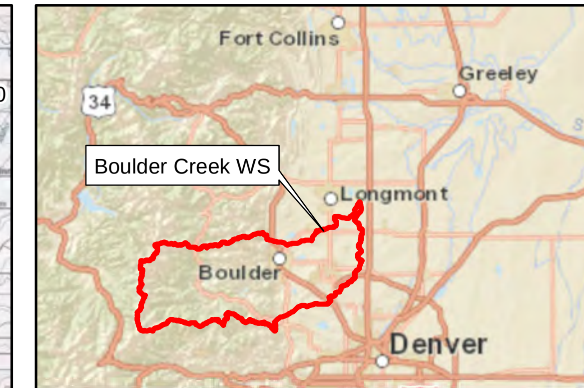
Figures





LEGEND

- Counties
- Boulder Creek Watershed
- Study Reach

Peak Discharge Estimation Locations

- Critical-Depth Method Estimate
- Bridge Hydraulics Estimate
- CDWR Stream Gage
- USGS Stream Gage
- Other

Boulder Creek Land Use Zones

- Phase 2 Plains
- Phase 2 Foothills
- Phase 2 Mountains
- Phase 1

N

0 1.5 3 4.5 6 Miles

1 inch = 3 miles

FIGURE 1
Boulder Creek Overview Map

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.

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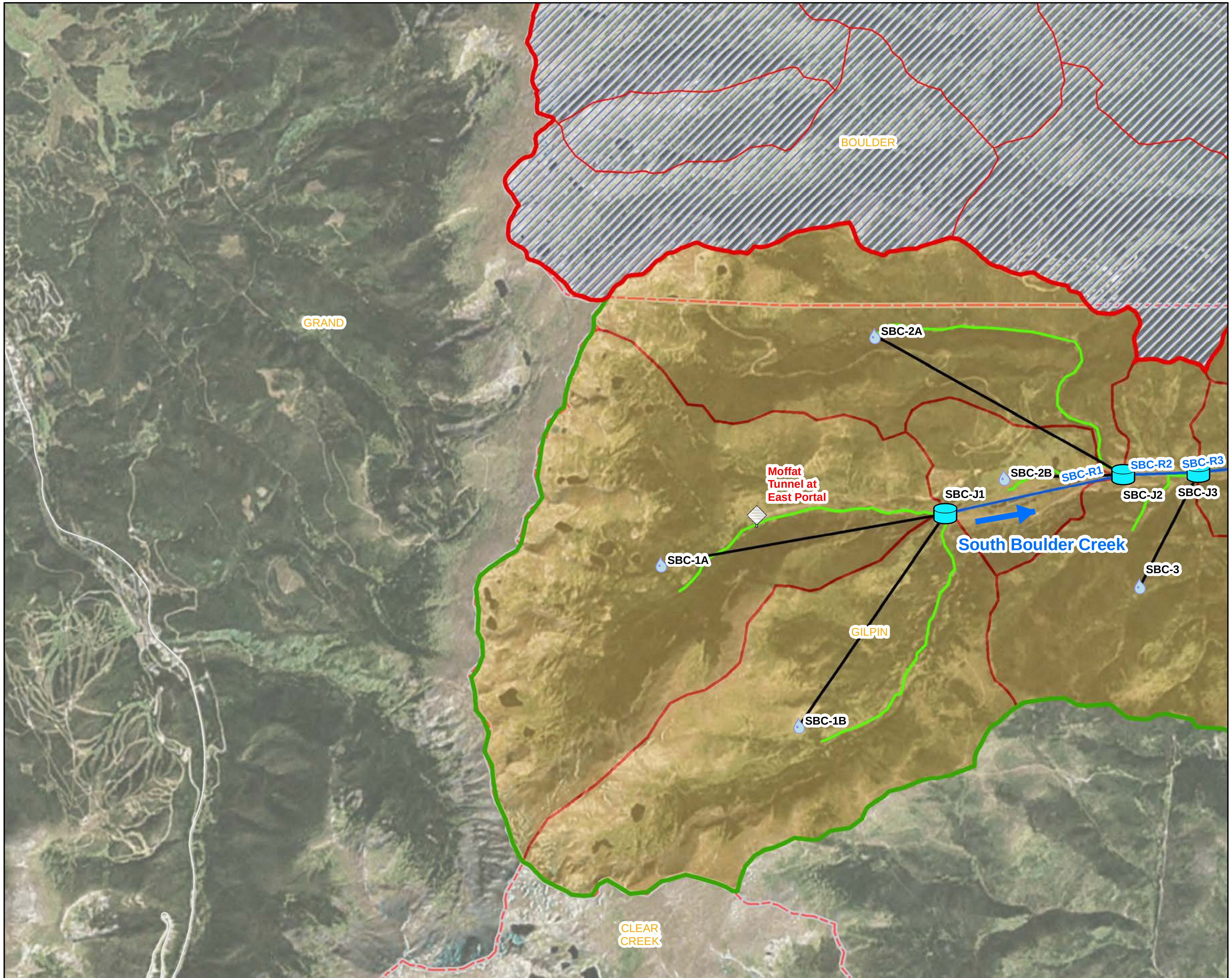

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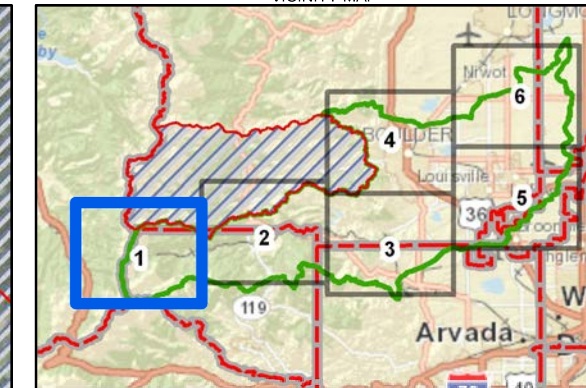


LEGEND

- Model Precipitation Gage

Subbasin Routing

- Reach Segment
- Basin Connection
- Model Junctions
- Transbasin Transfer
- Study Reach
- Phase I Model Subbasins
- Phase II Model Subbasins
- Flow Paths to Subbasin Centroid
- Phase I Model Watershed
- Phase II Model Watershed
- Counties

N

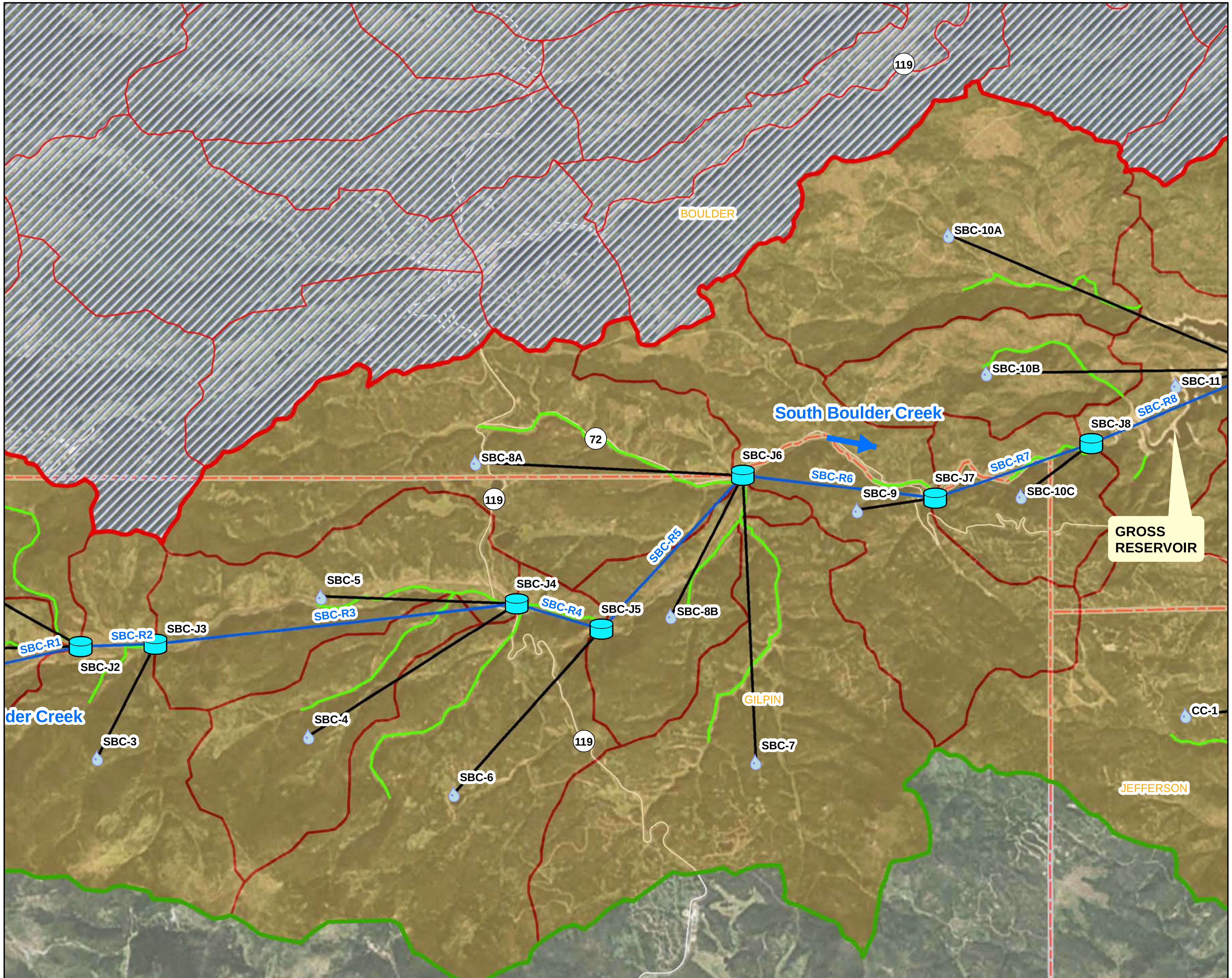
0 0.5 1 2 Miles

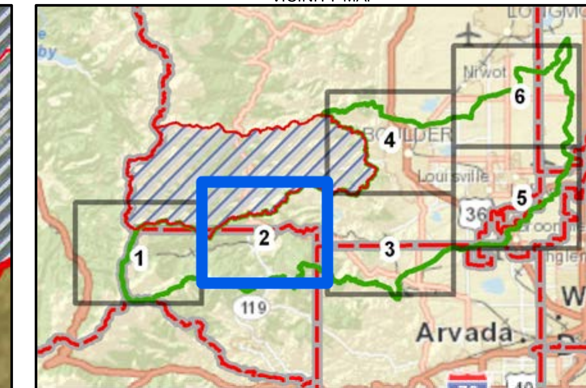
1 inch = 1 mile

FIGURE 2a
Phase II Connectivity Map - Page 1 of 6

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.





LEGEND

Model Precipitation Gage

Subbasin Routing

Reach Segment

Basin Connection

Model Junctions

Transbasin Transfer

Study Reach

Phase I Model Subbasins

Phase II Model Subbasins

Flow Paths to Subbasin Centroid

Phase I Model Watershed

Phase II Model Watershed

Counties

0 0.5 1 2 Miles

1 inch = 1 mile

FIGURE 2b
Phase II Connectivity Map - Page 2 of 6

CDOT Flood Recovery Hydrologic Evaluation

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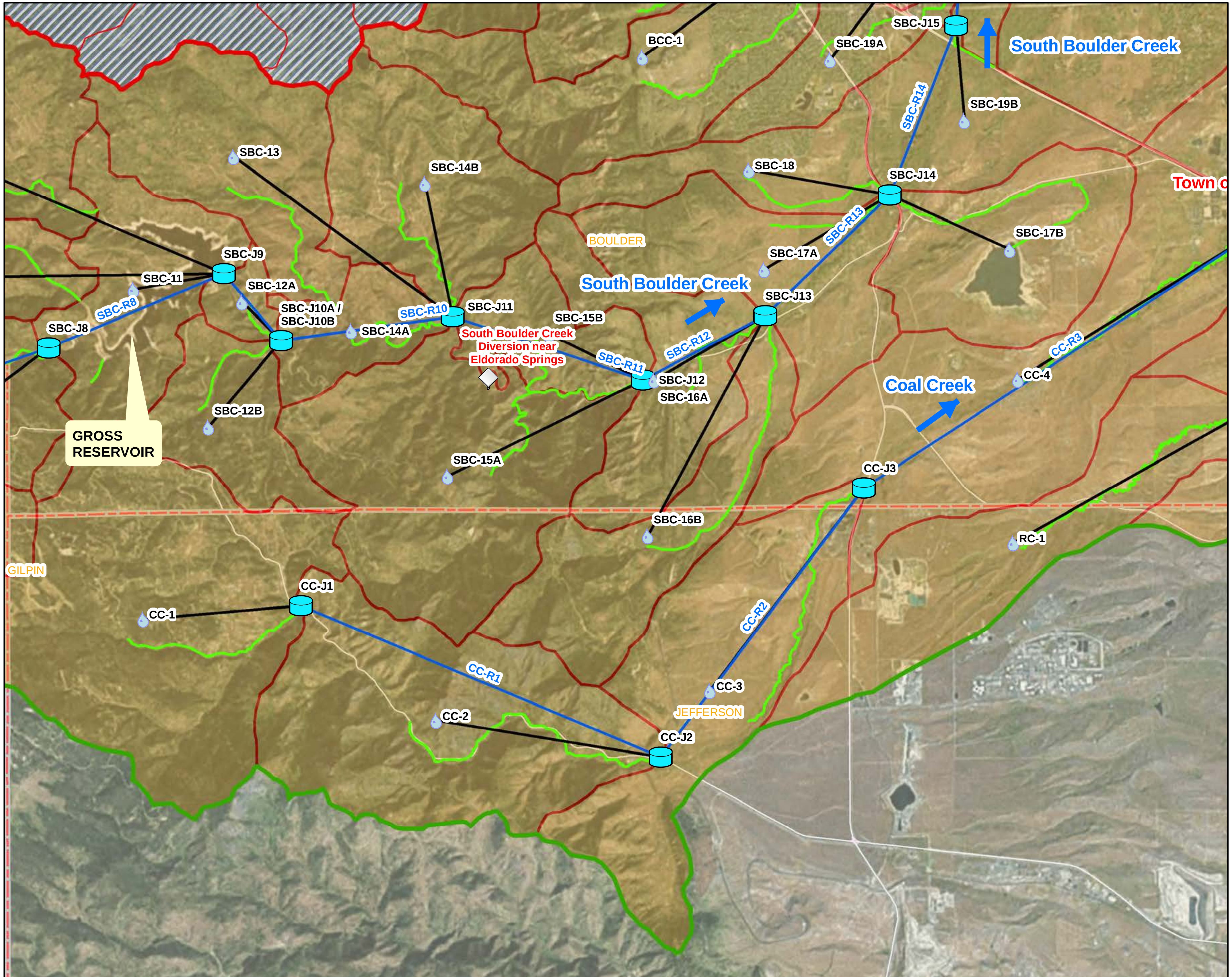

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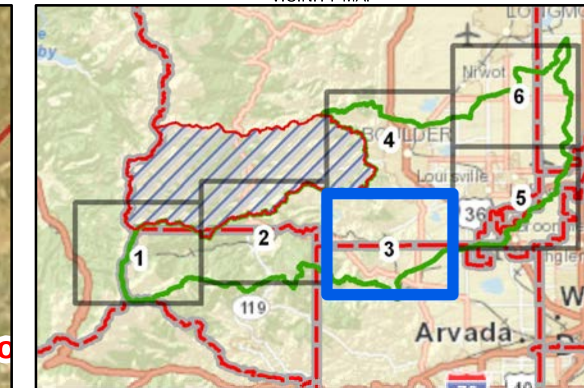


LEGEND

Model Precipitation Gage

Subbasin Routing

Reach Segment

Basin Connection

Model Junctions

Transbasin Transfer

Study Reach

Phase I Model Subbasins

Phase II Model Subbasins

Flow Paths to Subbasin Centroid

Phase I Model Watershed

Phase II Model Watershed

Counties

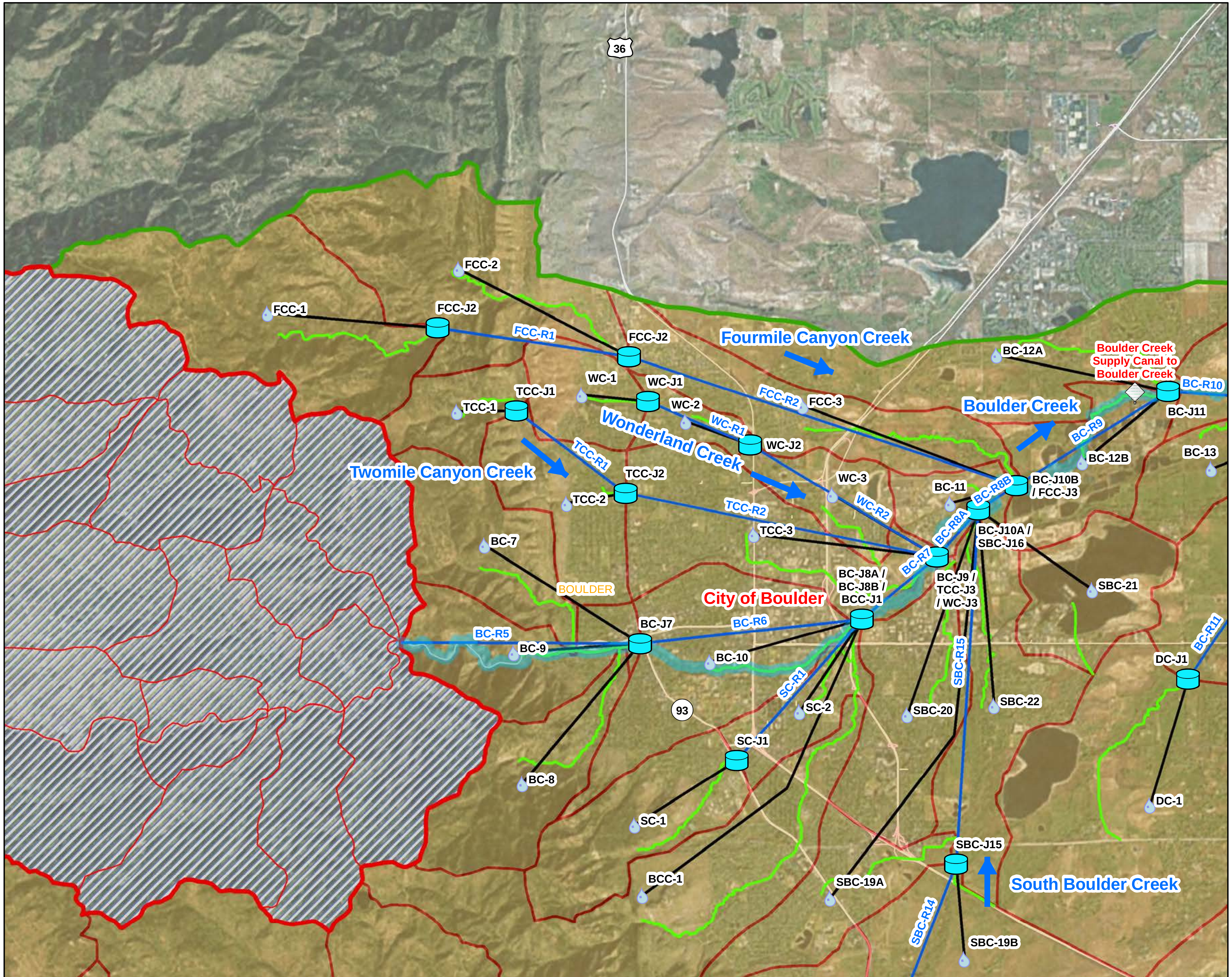
0 0.5 1 2 Miles

1 inch = 1 mile

FIGURE 2c
Phase II Connectivity Map - Page 3 of 6

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.



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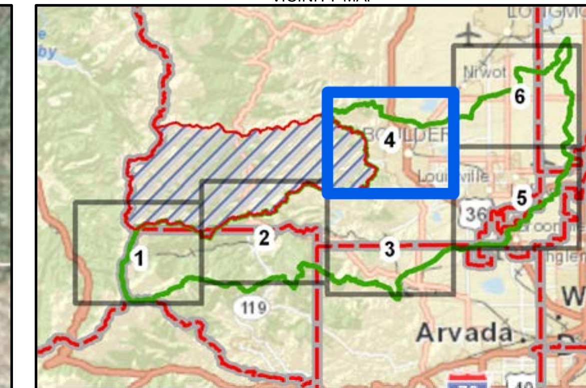


LEGEND

- Model Precipitation Gage

Subbasin Routing

- Reach Segment
- Basin Connection
- Model Junctions
- Transbasin Transfer
- Study Reach
- Phase I Model Subbasins
- Phase II Model Subbasins
- Flow Paths to Subbasin Centroid
- Phase I Model Watershed
- Phase II Model Watershed
- Counties

N

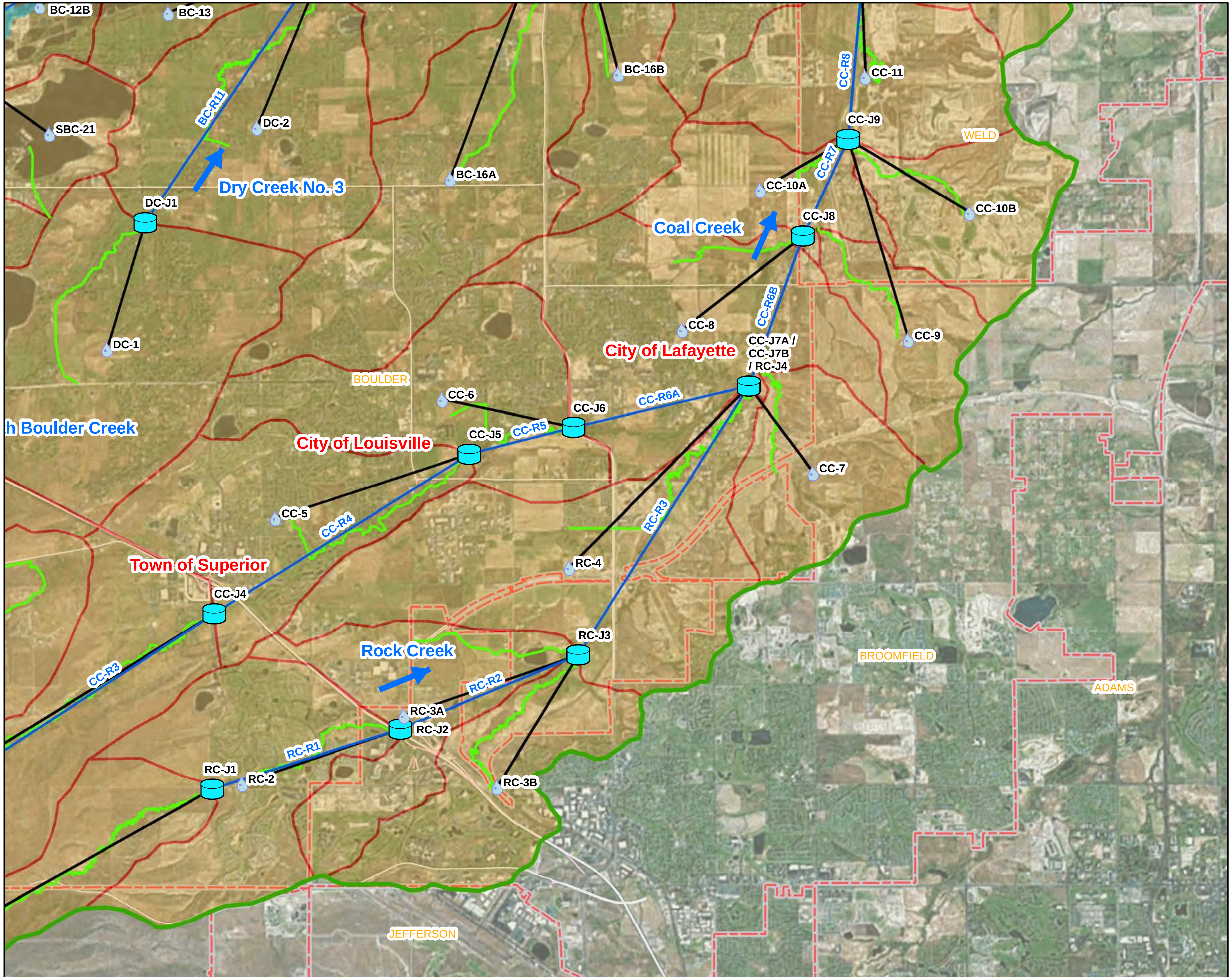
0 0.5 1 2 Miles

1 inch = 1 mile

FIGURE 2d
Phase II Connectivity Map - Page 4 of 6

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.



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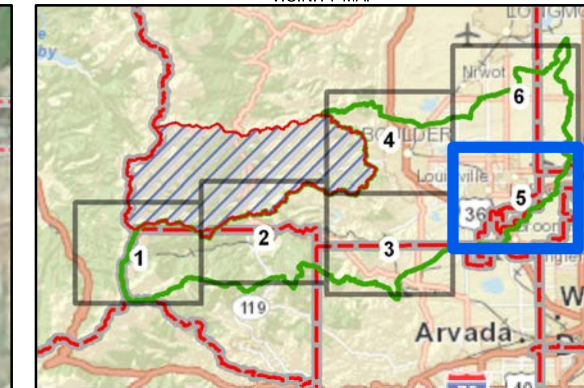


LEGEND

Model Precipitation Gage

Subbasin Routing

Reach Segment

Basin Connection

Model Junctions

Transbasin Transfer

Study Reach

Phase I Model Subbasins

Phase II Model Subbasins

Flow Paths to Subbasin Centroid

Phase I Model Watershed

Phase II Model Watershed

Counties

N

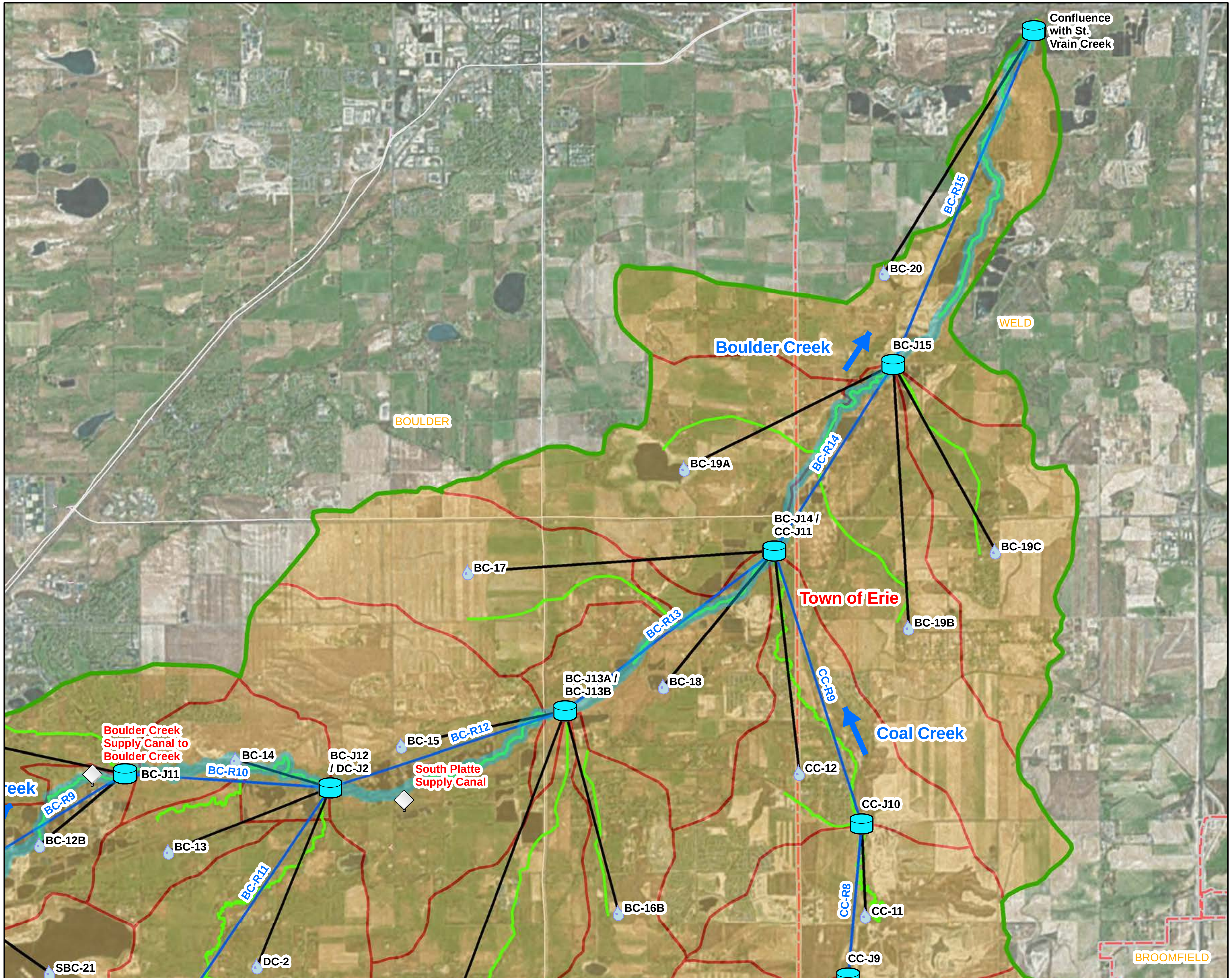
0 0.5 1 2 Miles

1 inch = 1 mile

FIGURE 2e
Phase II Connectivity Map - Page 5 of 6

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.



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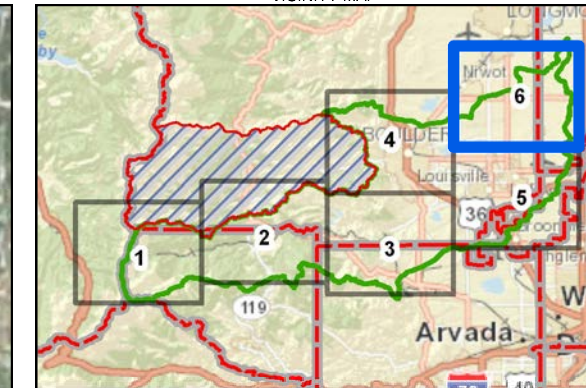


LEGEND

- Model Precipitation Gage

Subbasin Routing

- Reach Segment
- Basin Connection
- Model Junctions
- Transbasin Transfer
- Study Reach
- Phase I Model Subbasins
- Phase II Model Subbasins
- Flow Paths to Subbasin Centroid
- Phase I Model Watershed
- Phase II Model Watershed
- Counties

N

0 0.5 1 2 Miles

1 inch = 1 mile

FIGURE 2f
Phase II Connectivity Map - Page 6 of 6

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.

Figure 3 – ! ² ! 10 Day Precipitation (Phase 2)

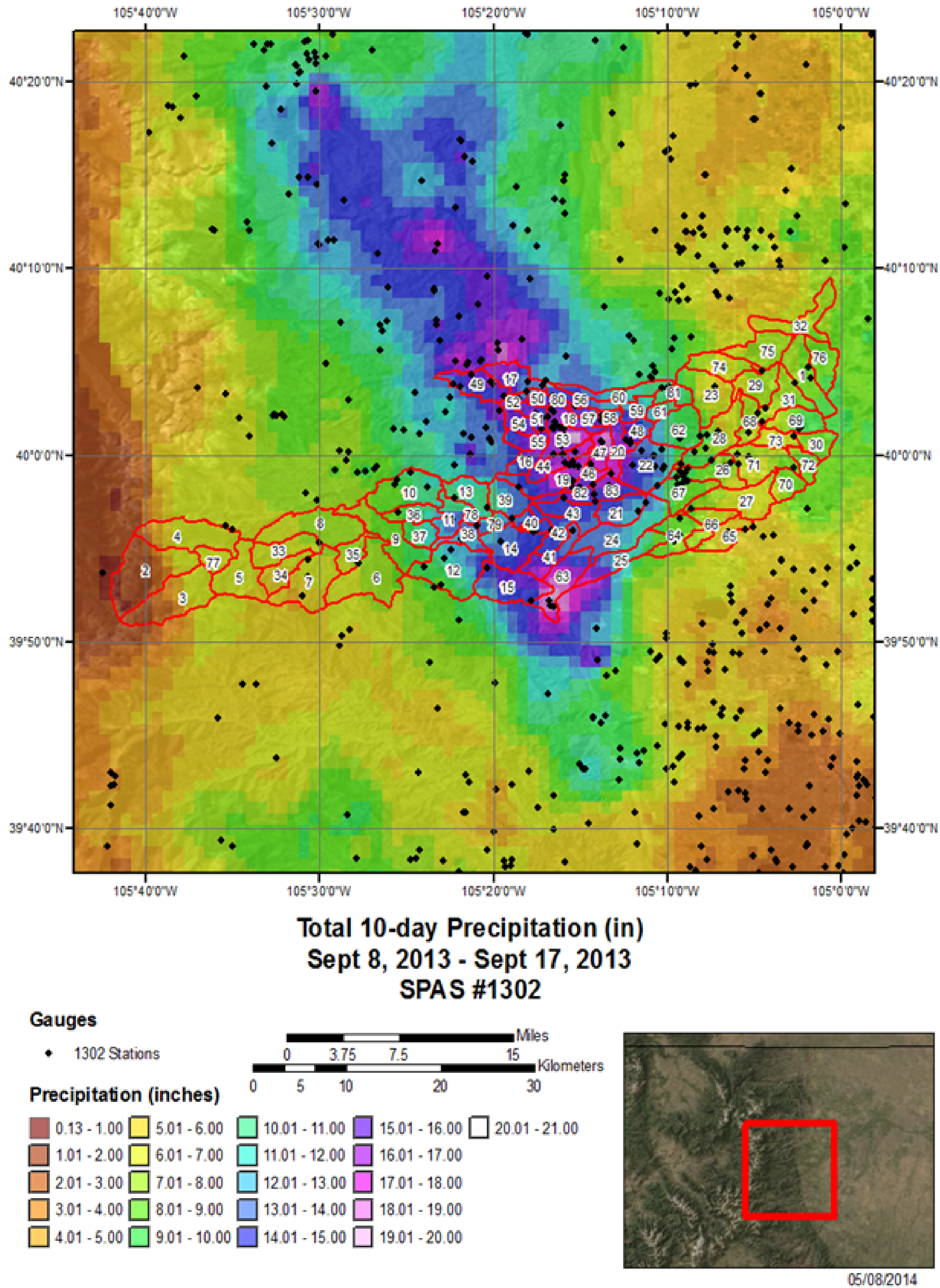


Figure 4a – Muskingum-Cunge Eight-Point Routing Cross Sections

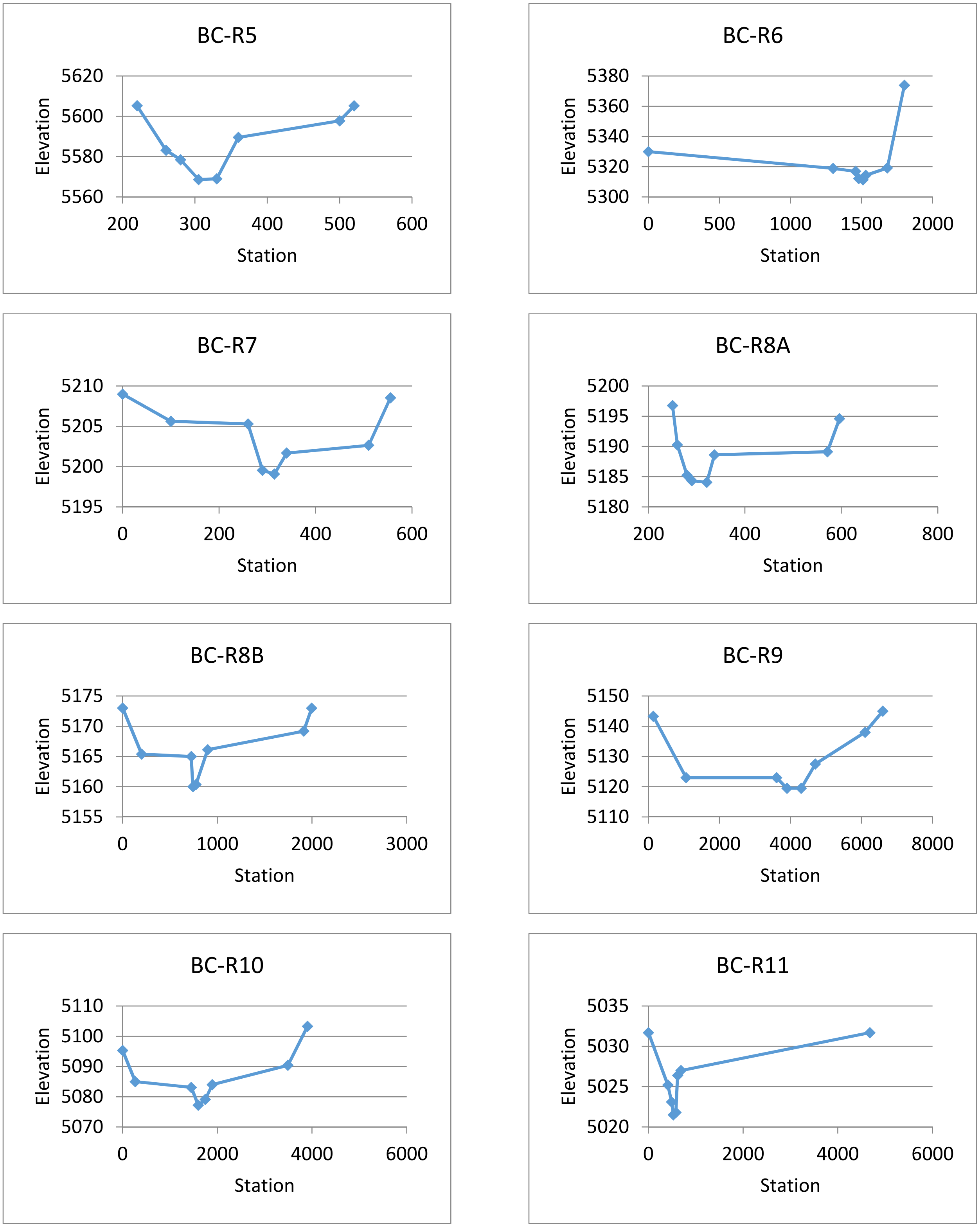
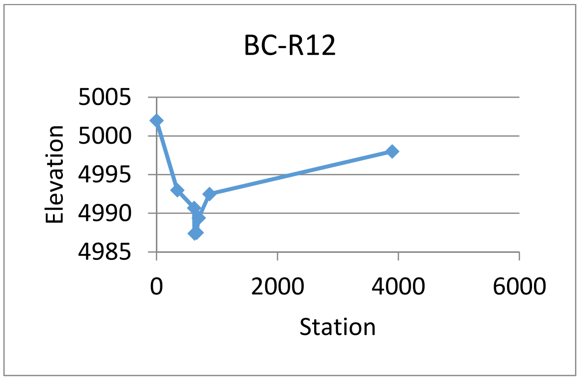
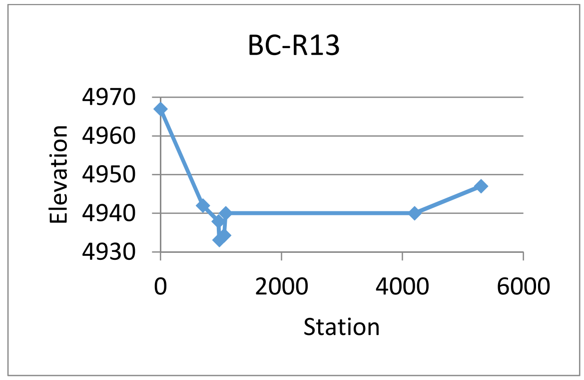


Figure 4b – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)





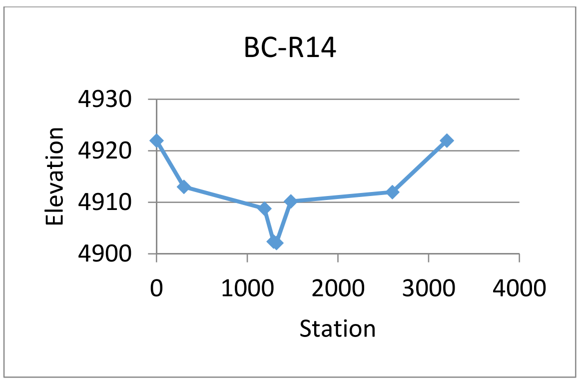


Figure 4c – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)

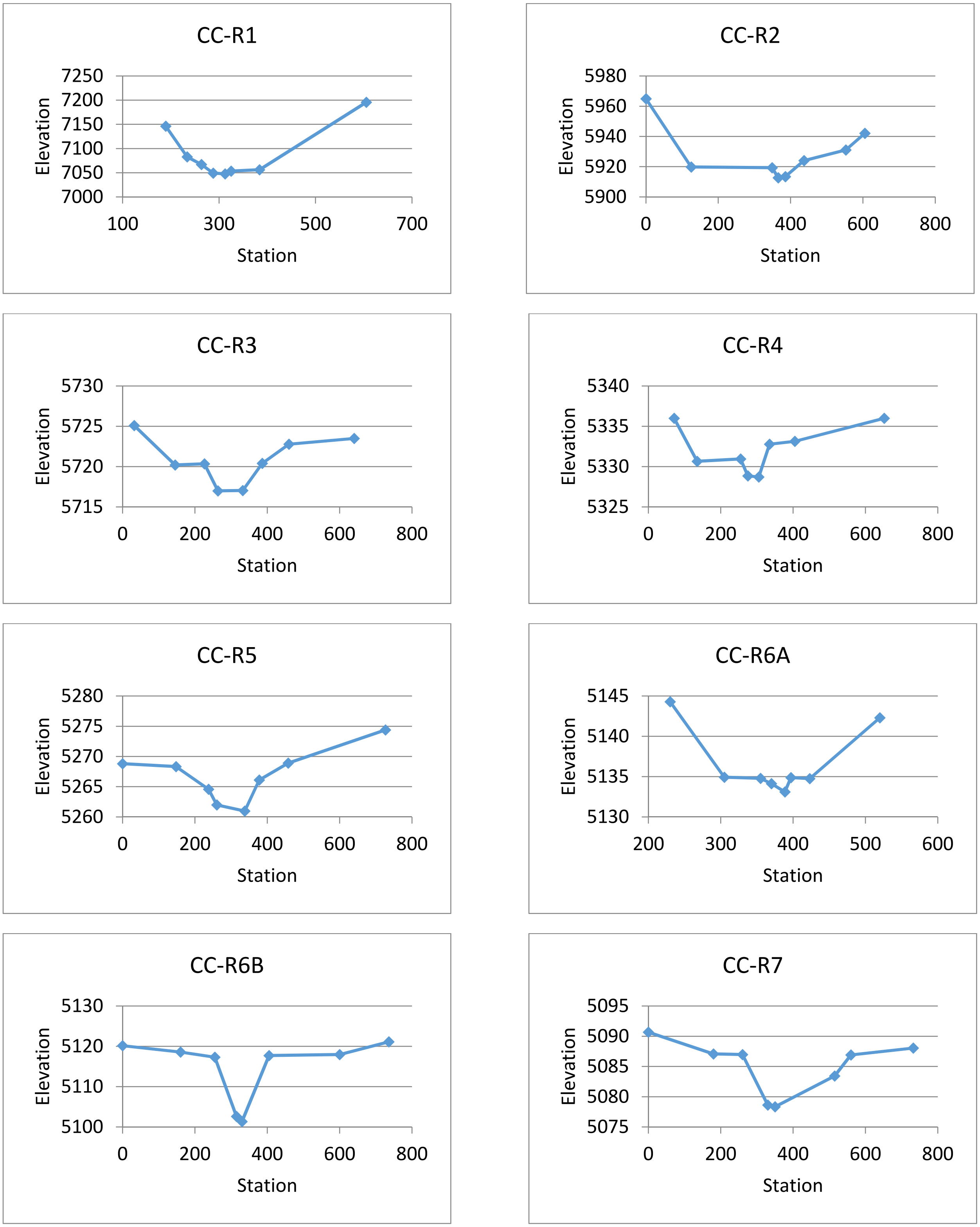
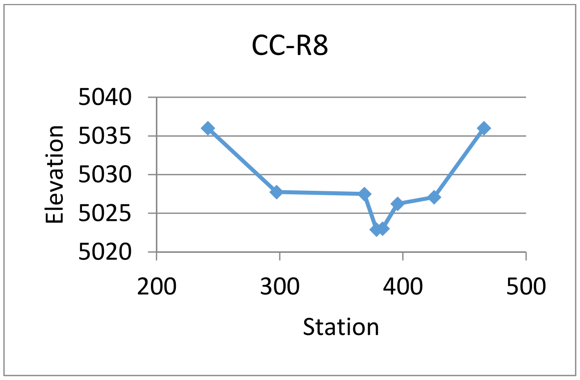
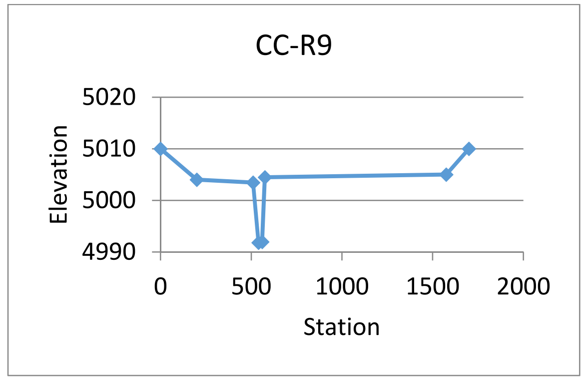
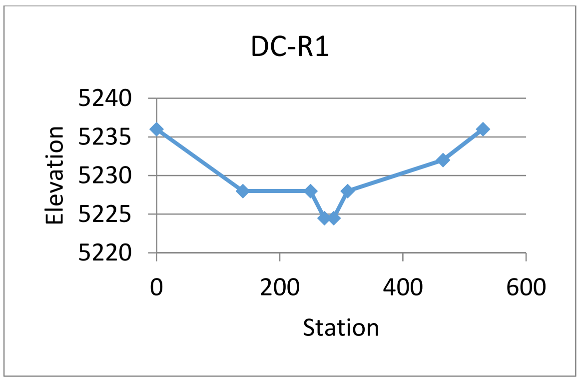
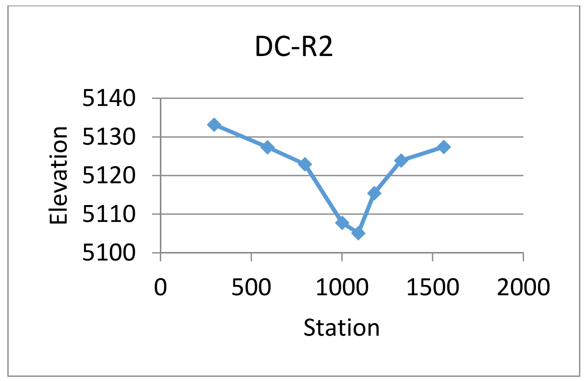


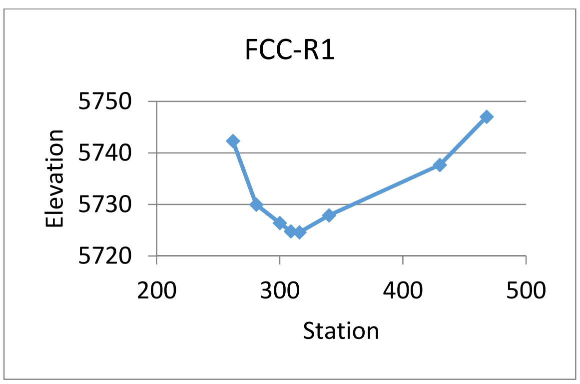
Figure 4d – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)











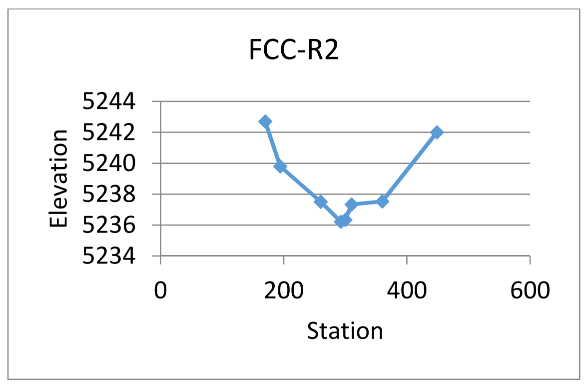
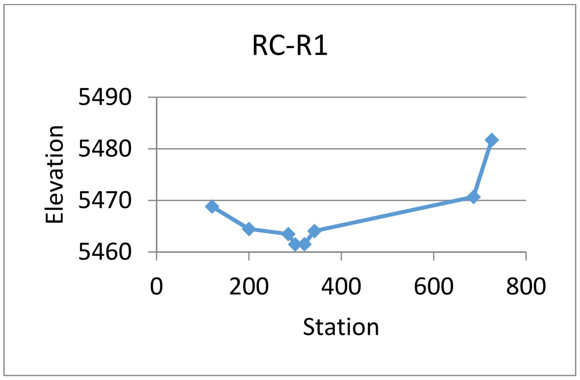
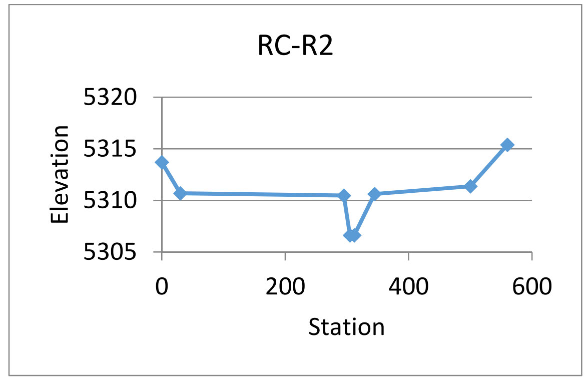


Figure 4e – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)





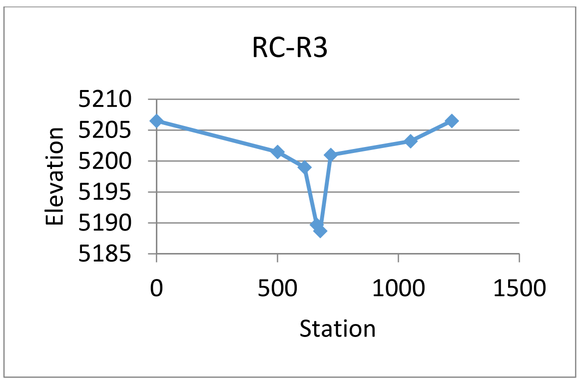
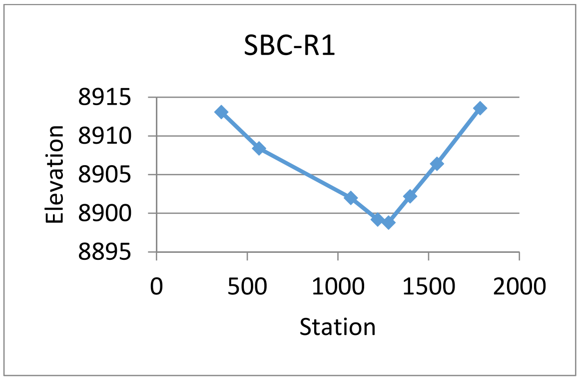
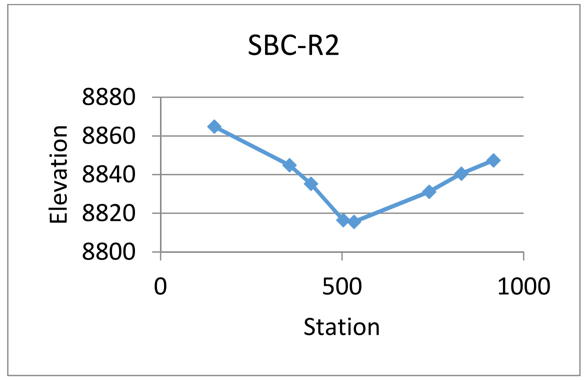
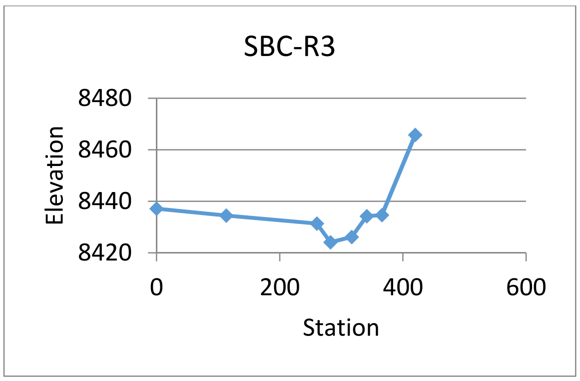
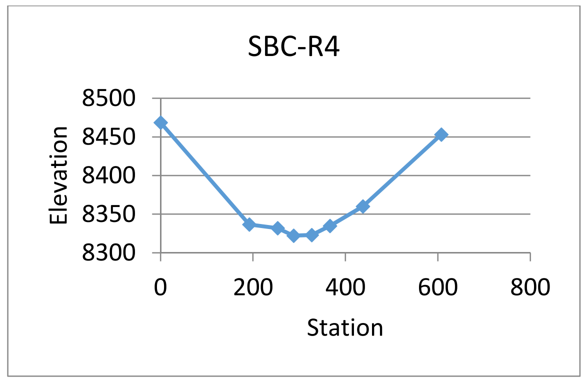


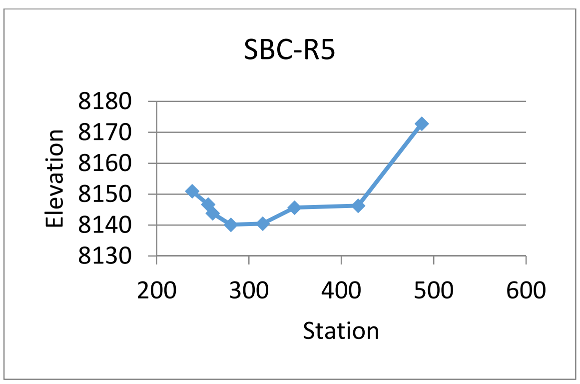
Figure 4f – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)

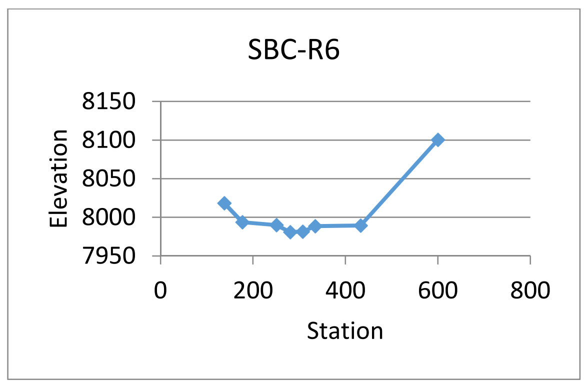


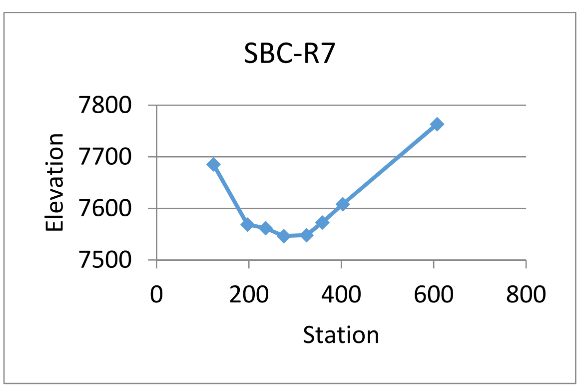






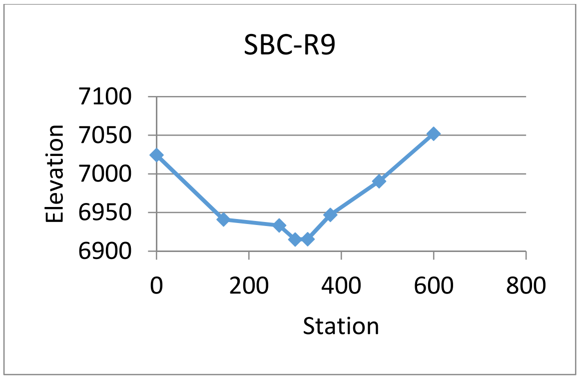


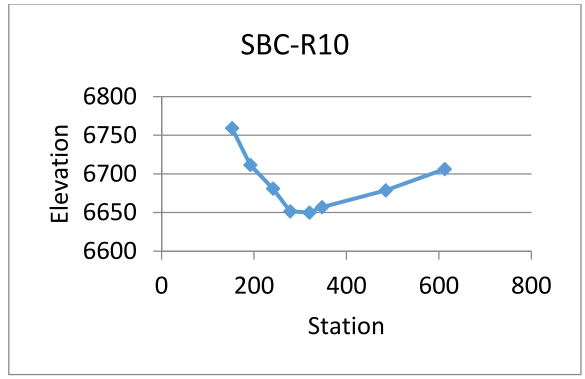


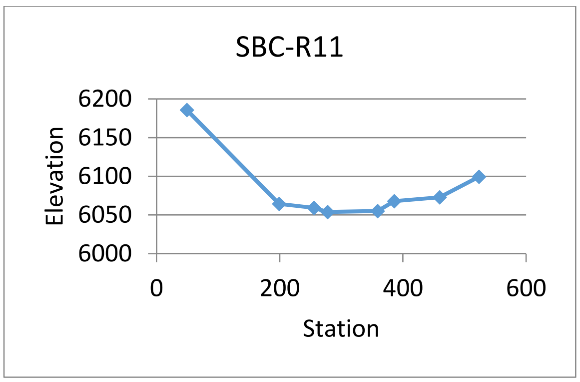


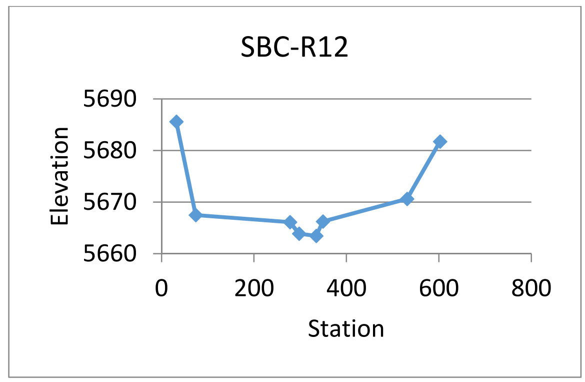
SBC-R8 intentionally omitted; Gross Reservoir modeled with Reservoir element in lieu of routing reach

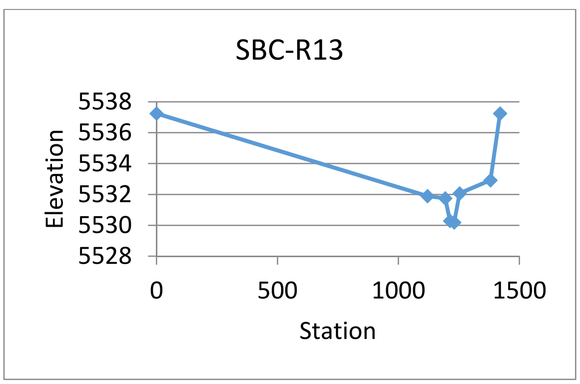
Figure 4g – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)

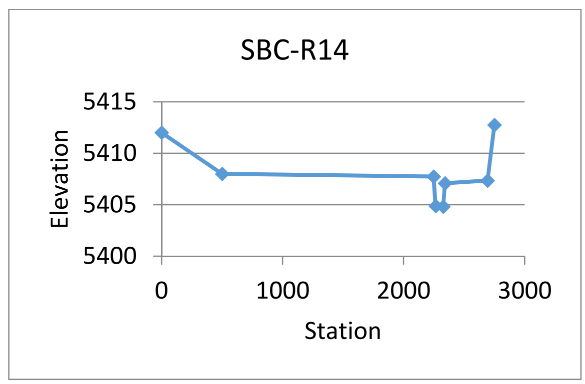












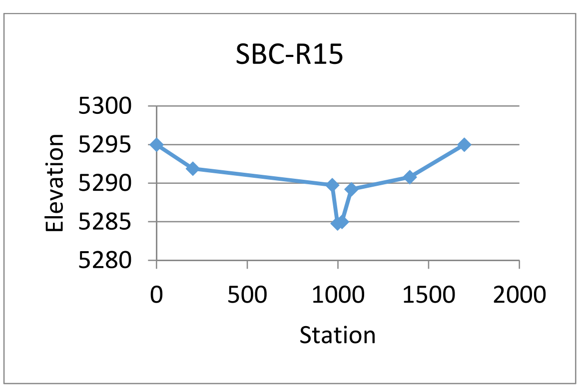
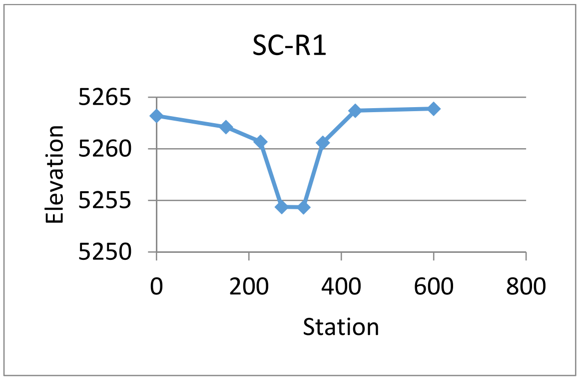
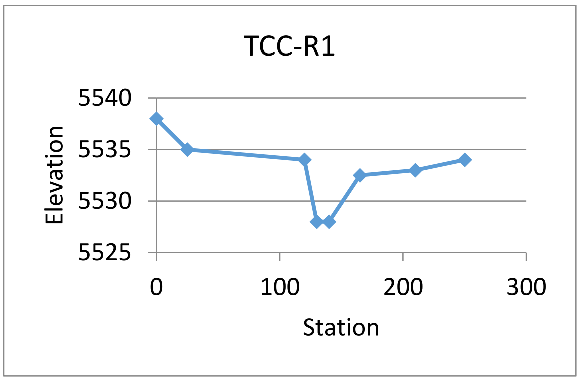
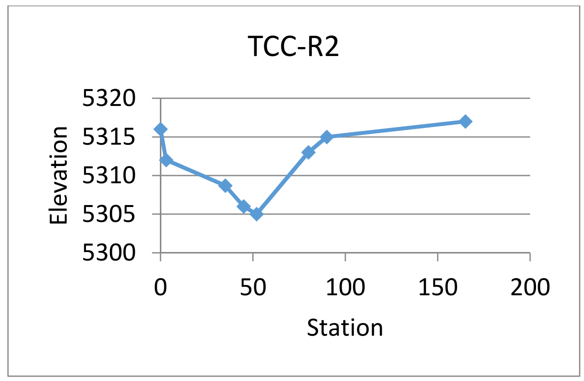
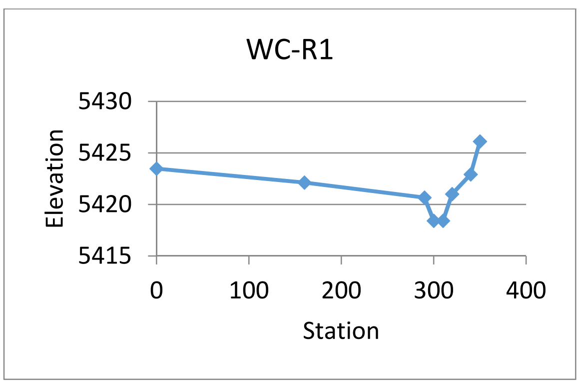


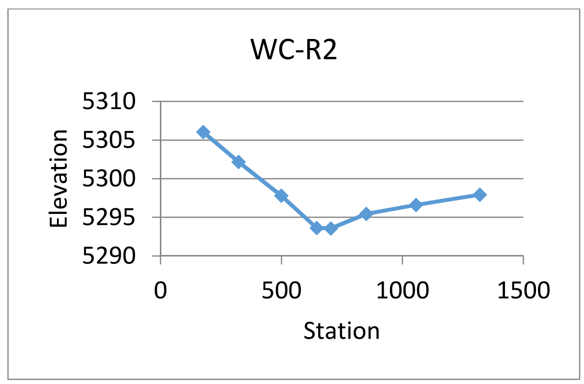
Figure 4h – Muskingum-Cunge Eight-Point Routing Cross Sections (continued)











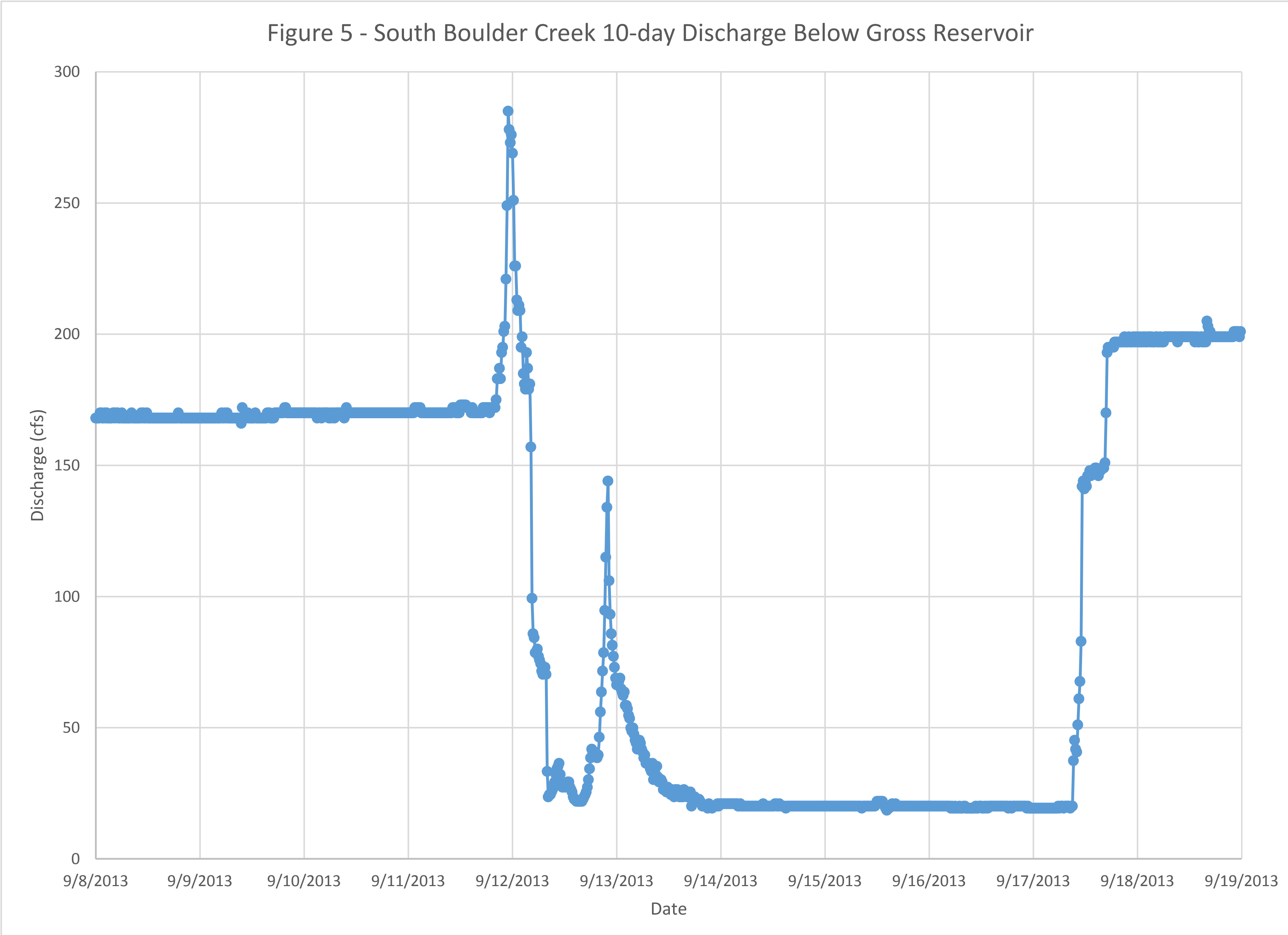


Figure 5 - South Boulder Creek 10-day Discharge Below Gross Reservoir
Discharge (cfs)
300
250
200
150
100
50
0
9/8/2013
9/9/2013
9/10/2013
9/11/2013
9/12/2013
9/13/2013
9/14/2013
9/15/2013
9/16/2013
9/17/2013
9/18/2013
9/19/2013
Date

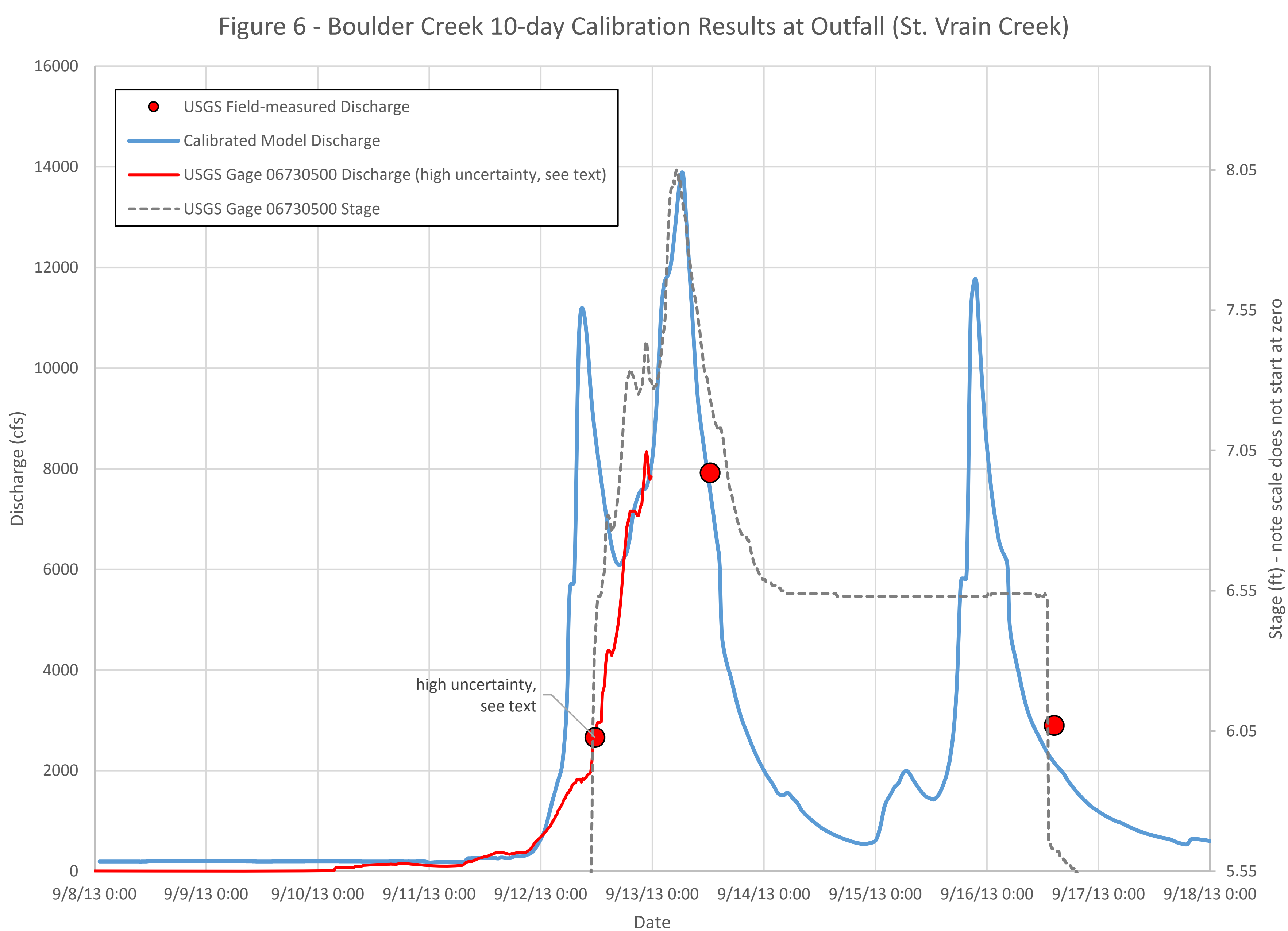


Figure 6 - Boulder Creek 10-day Calibration Results at Outfall (St. Vrain Creek)
USGS Field-measured Discharge
Calibrated Model Discharge
USGS Gage 06730500 Discharge (high uncertainty, see text)
USGS Gage 06730500 Stage
high uncertainty,
see text
Discharge (cfs)
Stage (ft) - note scale does not start at zero
Date
16000
14000
12000
10000
8000
6000
4000
2000
0
8.05
7.55
7.05
6.55
6.05
5.55
9/8/13 0:00
9/9/13 0:00
9/10/13 0:00
9/11/13 0:00
9/12/13 0:00
9/13/13 0:00
9/14/13 0:00
9/15/13 0:00
9/16/13 0:00
9/17/13 0:00
9/18/13 0:00

Appendix A
Hydrologic Analysis

Table A-1

Peak Discharge Estimates and Location for Boulder Creek Tributaries

Model Phase	Site Description	Calibration Source	Peak Discharge (cfs)
Phase 2	Bear Canyon Creek at Mohawk Drive	Jarrett, in press	1,330
Phase 2	Coal Creek downstream of Bear Creek	Jarrett, in press	1,110
Phase 2	Coal Creek near Plainview (UDFCD site)	Jarrett, in press	3,900
Phase 2	Coal Creek at US Highway 287	URS, 2015	5,000
Phase 2	Coal Creek at 120th Street	URS, 2015	3,500
Phase 2	Coal Creek at Erie Parkway	URS, 2015	6,000
Phase 2	Fourmile Canyon Creek near Sunshine, CO	USGS	1,090
Phase 2	Fourmile Canyon Creek at Pinto Drive	Jarrett, in press	1,080
Phase 2	Fourmile Canyon Creek upstream of Broadway Avenue	Jarrett, in press	1,460
Phase 2	Fourmile Canyon Creek upstream of Diagonal Highway	Jarrett, in press	2,300
Phase 1	Fourmile Creek upstream Burned Area	Jarrett, in press	490
Phase 1	Fourmile Creek downstream of Emerson Gulch	Jarrett, in press	1,070
Phase 1	Fourmile Creek near Orodell, CO	Jarrett, in press	2,300
		USGS	2,510
Phase 1	Middle Boulder Creek at Nederland, CO	CDWR	409
Phase 1	North Boulder Creek at Confluence with Middle Boulder Creek	Jarrett, in press	740
Phase 2	Rock Creek at 120th Street	URS, 2015	1,500
Phase 2	South Boulder Creek at Rollinsville, CO	Jarrett, in press	410
Phase 2	South Boulder Creek near Pinecliffe, CO	CDWR	781
Phase 2	South Boulder Creek below Gross Reservoir	CDWR	285
Phase 2	South Boulder Creek at Eldorado Springs	Jarrett, in press	2,120
Phase 2	South Boulder Creek at Highway 93	WWE, 2014	5,600 [a]
Phase 2	South Boulder Creek at South Boulder Creek Road	Jarrett, in press	3,600 [a, b]
Phase 2	Twomile Canyon Creek near North Cedar Brook Road	Jarrett, in press	1,210
Phase 2	Wonderland Creek at 15th Street	Jarrett, in press	170

cfs = cubic feet per second

[a] Possibly affected by railroad embankment breach on Doudy Draw

[b] May not account for overtopping of road and upstream flow splits to Dry Creek No. 2 and Baseline Reservoir

Table A-2

Time-of-Peak Discharge Measurements and Location for Boulder Creek Tributaries

Model Phase	Site Description	Calibration Source	Time-of-Peak Discharge
Phase 2	Fourmile Canyon Creek near Sunshine, CO	UDFCD	9/12 00:50
Phase 1	Fourmile Creek near Orodell, CO	UDFCD	9/12 22:45
Phase 1	Middle Boulder Creek at Nederland, CO	CDWR	9/13 02:15 [a]
Phase 2	South Boulder Creek near Pinecliffe, CO	CDWR	9/13 04:30
Phase 2	South Boulder Creek below Gross Reservoir	CDWR	9/11 23:00
Phase 2	South Boulder Creek near Eldorado Springs, CO	UDFCD	9/12 22:00
Phase 2	South Boulder Creek at Highway 93	UDFCD	9/12 19:40 [b]
Phase 2	South Boulder Creek at South Boulder Road	UDFCD	9/12 19:25 [b]

[a] Observed peak of 409 cfs was measured from 02:00 to 02:30; HEC-HMS results report the earliest time (02:00)

[b] Possibly affected by railroad embankment breach on Doudy Draw

Table A-3: 10-day Boulder Creek Model - Subbasin Area, Curve Number, and Initial Abstraction

Basin ID	Area mi^2	CN Zone	Calibrated CN	Calibrated I_a *inches*
BC-2A	1.23	Phase 1, Boulder Canyon	47.5	2.21
BC-2B	2.51	Phase 1, Boulder Canyon	34.9	3.73
BC-3A	0.54	Phase 1, Boulder Canyon	40.2	2.97
BC-3B	0.50	Phase 1, Boulder Canyon	40.5	2.94
BC-3C	1.96	Phase 1, Boulder Canyon	35.4	3.65
BC-4	2.57	Phase 1, Boulder Canyon	48.0	2.17
BC-5A	1.72	Phase 1, Boulder Canyon	41.6	2.81
BC-5B	1.60	Phase 1, Boulder Canyon	33.6	3.95
BC-6A	0.63	Phase 1, Boulder Canyon	44.1	2.53
BC-6B	2.05	Phase 1, Boulder Canyon	33.5	3.97
BC-7	1.89	Phase 2, Foothills	34.5	3.79
BC-8	1.99	Phase 2, Foothills	29.7	4.74
BC-9	1.89	Phase 2, Foothills	40.1	2.98
BC-10	3.04	Phase 2, Plains	30.0	4.67
BC-11	0.65	Phase 2, Plains	23.0	6.70
BC-12A	3.75	Phase 2, Plains	29.3	4.82
BC-12B	1.23	Phase 2, Plains	20.2	7.88
BC-13	1.22	Phase 2, Plains	30.5	4.56
BC-14	1.72	Phase 2, Plains	19.0	8.51
BC-15	5.61	Phase 2, Plains	30.2	4.62
BC-16A	6.22	Phase 2, Plains	38.5	3.20
BC-16B	2.63	Phase 2, Plains	41.9	2.77
BC-17	3.80	Phase 2, Plains	53.5	1.74
BC-18	3.68	Phase 2, Plains	42.9	2.66
BC-19A	4.09	Phase 2, Plains	42.8	2.67
BC-19B	5.32	Phase 2, Plains	42.7	2.68
BC-19C	3.59	Phase 2, Plains	51.5	1.89
BC-20	4.65	Phase 2, Plains	41.9	2.78
BCC-1	4.84	Phase 2, Plains	21.2	7.42
CC-1	8.65	Phase 2, Foothills	41.0	2.88
CC-2	6.46	Phase 2, Foothills	38.5	3.20
CC-3	5.31	Phase 2, Plains	45.1	2.44
CC-4	7.56	Phase 2, Plains	43.3	2.62
CC-5	4.16	Phase 2, Plains	27.4	5.30
CC-6	4.37	Phase 2, Plains	41.2	2.86
CC-7	2.57	Phase 2, Plains	57.9	1.45
CC-8	5.78	Phase 2, Plains	42.0	2.76
CC-9	1.92	Phase 2, Plains	37.6	3.32
CC-10A	2.14	Phase 2, Plains	36.2	3.52
CC-10B	2.47	Phase 2, Plains	38.2	3.23
CC-11	3.81	Phase 2, Plains	46.7	2.28
CC-12	4.09	Phase 2, Plains	44.1	2.54
DC-1	7.04	Phase 2, Plains	44.3	2.52
DC-2	5.33	Phase 2, Plains	32.8	4.10
FCC-1	4.01	Phase 2, Foothills	33.9	1.95
FCC-2	3.67	Phase 2, Foothills	34.2	3.85
FCC-3	2.38	Phase 2, Plains	30.1	4.64
FMC-1	2.59	Phase 1, Fourmile Creek	27.8	5.18
FMC-2A	2.44	Phase 1, Fourmile Creek	34.9	3.73
FMC-2B	1.83	Phase 1, Fourmile Creek	32.9	4.08
FMC-3A	3.76	Phase 1, Fourmile Creek	44.5	2.49
FMC-3B	2.58	Phase 1, Fourmile Creek	32.2	4.22
FMC-4A	2.75	Phase 1, Fourmile Creek	41.3	2.84
FMC-4B	1.67	Phase 1, Fourmile Creek	53.8	1.72
FMC-4C	1.19	Phase 1, Fourmile Creek	30.9	4.47

Table A-3: 10-day Boulder Creek Model - Subbasin Area, Curve Number, and Initial Abstraction

Basin ID	Area *mi^2*	CN Zone	Calibrated CN	Calibrated I_a *inches*
FMC-5A	1.16	Phase 1, Fourmile Creek	46.8	2.28
FMC-5B	1.47	Phase 1, Fourmile Creek	37.6	3.32
FMC-6A	1.73	Phase 1, Fourmile Creek	44.2	2.53
FMC-6B	1.13	Phase 1, Fourmile Creek	48.5	2.13
MBC-1A	4.31	Phase 1, Middle Boulder Creek	57.9	1.45
MBC-1B	1.75	Phase 1, Middle Boulder Creek	51.6	1.88
MBC-2	5.79	Phase 1, Middle Boulder Creek	63.4	1.15
MBC-3A	5.74	Phase 1, Middle Boulder Creek	51.2	1.91
MBC-3B	5.76	Phase 1, Middle Boulder Creek	54.0	1.70
MBC-4	4.82	Phase 1, Middle Boulder Creek	51.2	1.91
MBC-5A	5.47	Phase 1, Middle Boulder Creek	51.3	1.90
MBC-5B	2.93	Phase 1, Middle Boulder Creek	57.4	1.48
MBC-6	2.13	Phase 1, Middle Boulder Creek	70.8	0.82
MBC-7A	1.97	Phase 1, Middle Boulder Creek	61.3	1.26
MBC-7B	3.67	Phase 1, Middle Boulder Creek	52.3	1.82
NBC-1A	3.40	Phase 1, North Boulder Creek	56.7	1.53
NBC-1B	5.31	Phase 1, North Boulder Creek	55.0	1.64
NBC-2	5.10	Phase 1, North Boulder Creek	30.3	4.59
NBC-3	4.85	Phase 1, North Boulder Creek	33.8	3.92
NBC-4	4.55	Phase 1, North Boulder Creek	31.4	4.36
NBC-5A	3.38	Phase 1, North Boulder Creek	39.2	3.11
NBC-5B	4.00	Phase 1, North Boulder Creek	39.6	3.05
NBC-5C	2.52	Phase 1, North Boulder Creek	49.3	2.06
NBC-6A	5.83	Phase 1, North Boulder Creek	44.9	2.46
NBC-6B	3.57	Phase 1, North Boulder Creek	51.1	1.92
NBC-7	2.27	Phase 1, North Boulder Creek	49.9	2.01
RC-1	4.69	Phase 2, Plains	54.2	1.69
RC-2	4.06	Phase 2, Plains	48.8	2.10
RC-3A	3.21	Phase 2, Plains	53.6	1.73
RC-3B	3.10	Phase 2, Plains	55.4	1.61
RC-4	6.60	Phase 2, Plains	56.1	1.56
SBC-1A	11.58	Phase 2, Mountains	51.0	1.92
SBC-1B	8.96	Phase 2, Mountains	45.7	2.38
SBC-2A	7.81	Phase 2, Mountains	43.6	2.59
SBC-2B	2.06	Phase 2, Mountains	31.4	4.37
SBC-3	6.08	Phase 2, Mountains	31.3	4.40
SBC-4	3.19	Phase 2, Mountains	47.3	2.23
SBC-5	4.30	Phase 2, Mountains	39.6	3.06
SBC-6	7.67	Phase 2, Mountains	34.7	3.76
SBC-7	8.87	Phase 2, Foothills	34.3	3.83
SBC-8A	8.30	Phase 2, Mountains	45.0	2.45
SBC-8B	2.58	Phase 2, Mountains	52.5	1.81
SBC-9	6.65	Phase 2, Mountains	47.2	2.24
SBC-10A	7.10	Phase 2, Mountains	49.9	2.01
SBC-10B	2.57	Phase 2, Foothills	28.8	4.95
SBC-10C	2.41	Phase 2, Foothills	37.0	3.41
SBC-11	3.16	Phase 2, Foothills	47.5	2.21
SBC-12A	0.47	Phase 2, Foothills	34.8	3.76
SBC-12B	2.37	Phase 2, Foothills	24.6	6.13
SBC-13	3.72	Phase 2, Foothills	27.8	5.20
SBC-14A	1.41	Phase 2, Foothills	25.6	5.82
SBC-14B	3.75	Phase 2, Foothills	22.1	7.07
SBC-15A	6.02	Phase 2, Foothills	26.8	5.47
SBC-15B	1.74	Phase 2, Foothills	32.3	4.19
SBC-16A	2.96	Phase 2, Foothills	20.1	7.96

Table A-3: 10-day Boulder Creek Model - Subbasin Area, Curve Number, and Initial Abstraction

Basin ID	Area *mi^2*	CN Zone	Calibrated CN	Calibrated I_a *inches*
SBC-16B	3.34	Phase 2, Plains	23.3	6.58
SBC-17A	3.60	Phase 2, Plains	25.3	5.92
SBC-17B	3.61	Phase 2, Plains	41.8	2.79
SBC-18	1.92	Phase 2, Foothills	15.5	10.87
SBC-19A	1.62	Phase 2, Plains	20.6	7.72
SBC-19B	2.49	Phase 2, Plains	31.7	4.30
SBC-20	2.29	Phase 2, Plains	26.9	5.44
SBC-21	1.68	Phase 2, Plains	72.2	18.00
SBC-22	2.52	Phase 2, Plains	19.1	8.49
SC-1	2.04	Phase 2, Plains	22.3	6.95
SC-2	0.95	Phase 2, Plains	36.3	3.51
TCC-1	0.94	Phase 2, Foothills	34.7	3.76
TCC-2	1.68	Phase 2, Plains	26.3	5.61
TCC-3	2.88	Phase 2, Plains	28.9	4.93
WC-1	1.04	Phase 2, Plains	29.2	4.84
WC-2	0.50	Phase 2, Plains	28.5	5.01
WC-3	0.96	Phase 2, Plains	35.4	3.65

Table A-4: 10-day Boulder Creek Model - Calibrated Lag Time and Peaking Factor Parameters

Basin ID	K_n	L *mi*	L_c *mi*	S *ft/mile*	T_{LAG} *hours*	C_p
BC-2A	0.15	2.23	1.26	790.4	1.55	0.20
BC-2B	0.15	2.84	1.59	911.1	1.77	0.20
BC-3A	0.15	1.66	0.99	482.7	0.59	0.20
BC-3B	0.15	1.81	1.11	474.8	0.63	0.20
BC-3C	0.15	3.45	2.04	690.6	2.14	0.20
BC-4	0.15	3.10	1.51	48.5	2.91	0.20
BC-5A	0.15	3.04	1.95	515.3	2.13	0.20
BC-5B	0.15	3.12	1.80	666.6	2.00	0.20
BC-6A	0.15	1.92	1.26	379.2	0.93	0.20
BC-6B	0.15	2.73	1.53	802.9	1.76	0.20
BC-7	0.08	3.23	1.57	373.7	1.14	0.50
BC-8	0.08	3.66	1.98	626.4	1.17	0.50
BC-9	0.08	3.01	1.24	350.9	1.04	0.50
BC-10	0.10	4.78	1.75	144.4	1.96	0.45
BC-11	0.10	1.58	0.81	130.4	1.07	0.45
BC-12A	0.12	5.81	2.00	68.0	2.97	0.40
BC-12B	0.15	2.88	1.53	23.8	3.21	0.30
BC-13	0.15	2.86	1.40	27.0	3.04	0.30
BC-14	0.15	3.34	1.57	17.6	3.57	0.30
BC-15	0.15	3.67	1.73	52.3	3.17	0.30
BC-16A	0.12	7.17	3.50	70.1	3.81	0.40
BC-16B	0.12	3.60	1.69	77.5	2.35	0.40
BC-17	0.12	4.25	2.32	83.0	2.72	0.40
BC-18	0.15	3.75	1.95	22.7	3.82	0.30
BC-19A	0.15	5.44	3.20	45.5	4.53	0.30
BC-19B	0.15	6.79	3.53	42.8	5.09	0.30
BC-19C	0.12	4.30	2.19	59.6	2.83	0.40
BC-20	0.15	7.12	3.57	31.6	5.45	0.30
BCC-1	0.08	8.00	4.43	396.6	2.14	0.50
CC-1	0.08	4.55	2.02	405.1	1.36	0.50
CC-2	0.10	5.91	3.32	291.3	2.32	0.45
CC-3	0.10	6.02	3.12	187.5	2.45	0.45
CC-4	0.12	7.01	3.52	94.8	3.60	0.40
CC-5	0.12	5.11	3.05	60.4	3.34	0.40
CC-6	0.12	4.68	2.00	94.6	2.62	0.40
CC-7	0.12	3.00	1.59	74.5	2.18	0.40
CC-8	0.12	3.19	1.39	77.5	2.12	0.40
CC-9	0.12	3.31	1.67	93.4	2.21	0.40
CC-10A	0.12	3.08	1.48	71.8	2.16	0.40
CC-10B	0.12	2.72	1.71	61.3	2.24	0.40
CC-11	0.15	4.10	2.26	34.3	3.86	0.30
CC-12	0.15	6.78	4.54	29.9	5.86	0.30
DC-1	0.12	3.79	2.26	91.7	2.56	0.40
DC-2	0.15	5.13	3.74	51.9	4.58	0.30

Table A-4: 10-day Boulder Creek Model - Calibrated Lag Time and Peaking Factor Parameters

Basin ID	K_n	L *mi*	L_c *mi*	S *ft/mile*	T_{LAG} *hours*	C_p
FCC-1	0.08	4.85	2.01	368.3	1.41	0.50
FCC-2	0.08	4.49	2.33	423.4	1.41	0.50
FCC-3	0.12	5.33	2.77	66.6	3.23	0.40
FMC-1	0.05	3.19	1.73	853.1	1.19	0.60
FMC-2A	0.05	3.22	1.99	617.4	1.33	0.60
FMC-2B	0.05	2.95	2.03	493.4	1.34	0.60
FMC-3A	0.05	4.06	2.14	340.3	0.75	0.60
FMC-3B	0.05	3.81	2.09	313.1	0.82	0.60
FMC-4A	0.05	3.57	1.83	564.0	1.35	0.60
FMC-4B	0.05	3.76	2.29	372.0	0.60	0.60
FMC-4C	0.05	3.37	2.03	274.2	0.49	0.60
FMC-5A	0.05	2.33	1.37	57.1	1.56	0.60
FMC-5B	0.05	2.38	1.66	310.0	1.26	0.60
FMC-6A	0.05	3.15	2.39	314.5	0.78	0.60
FMC-6B	0.05	2.51	1.51	358.3	0.60	0.60
MBC-1A	0.15	3.95	2.24	516.3	3.24	0.10
MBC-1B	0.15	2.87	1.75	525.1	2.68	0.10
MBC-2	0.15	3.73	1.40	592.5	2.66	0.10
MBC-3A	0.15	4.87	2.48	841.2	3.31	0.10
MBC-3B	0.15	5.96	2.79	410.3	4.14	0.10
MBC-4	0.15	3.46	1.57	324.7	2.98	0.10
MBC-5A	0.15	5.59	3.16	451.7	4.16	0.10
MBC-5B	0.15	4.06	1.97	98.3	4.12	0.10
MBC-6	0.15	0.92	0.66	239.0	1.52	0.10
MBC-7A	0.15	6.33	3.45	168.6	1.84	0.10
MBC-7B	0.15	7.08	4.24	150.1	2.08	0.10
NBC-1A	0.15	5.10	2.53	429.2	2.83	0.15
NBC-1B	0.15	4.50	2.46	477.8	2.65	0.15
NBC-2	0.15	4.48	2.19	693.7	2.39	0.15
NBC-3	0.15	3.88	2.62	398.8	2.65	0.15
NBC-4	0.15	5.73	2.90	571.6	2.94	0.15
NBC-5A	0.15	4.53	2.36	564.5	2.55	0.15
NBC-5B	0.15	4.50	2.95	370.5	2.93	0.15
NBC-5C	0.15	3.96	1.62	302.5	2.39	0.15
NBC-6A	0.15	5.07	3.46	281.7	3.36	0.15
NBC-6B	0.15	4.38	2.15	137.2	3.08	0.15
NBC-7	0.15	2.70	1.81	271.6	2.22	0.15
RC-1	0.10	7.58	4.26	121.0	3.15	0.45
RC-2	0.12	5.13	2.09	99.1	2.72	0.40
RC-3A	0.12	3.80	2.05	71.7	2.58	0.40
RC-3B	0.12	4.69	2.70	68.8	3.05	0.40
RC-4	0.15	7.15	4.07	42.3	5.44	0.30
SBC-1A	0.15	6.75	2.96	504.9	3.19	0.20
SBC-1B	0.15	6.43	2.88	366.3	3.28	0.10

Table A-4: 10-day Boulder Creek Model - Calibrated Lag Time and Peaking Factor Parameters

Basin ID	K_n	L *mi*	L_c *mi*	S *ft/mile*	T_{LAG} *hours*	C_p
SBC-2A	0.15	6.86	3.87	333.2	3.75	0.10
SBC-2B	0.15	2.63	1.13	538.6	1.68	0.20
SBC-3	0.15	3.42	1.00	578.3	1.74	0.20
SBC-4	0.15	3.97	2.00	557.5	2.31	0.20
SBC-5	0.15	4.31	2.08	106.7	3.16	0.10
SBC-6	0.15	5.72	3.62	387.7	3.37	0.20
SBC-7	0.15	5.48	2.69	226.7	3.29	0.10
SBC-8A	0.15	6.57	2.96	224.2	3.61	0.10
SBC-8B	0.15	3.25	1.57	295.3	2.22	0.10
SBC-9	0.15	4.56	0.71	208.3	2.03	0.10
SBC-10A	0.15	4.67	2.09	227.4	2.87	0.10
SBC-10B	0.15	3.58	1.70	306.2	2.34	0.10
SBC-10C	0.15	3.53	1.04	404.3	1.89	0.20
SBC-11	0.08	0.71	0.24	2106.4	0.28	0.50
SBC-12A	0.08	1.24	0.51	882.8	0.49	0.50
SBC-12B	0.08	3.31	1.24	759.6	0.94	0.50
SBC-13	0.08	4.06	2.06	348.4	1.36	0.50
SBC-14A	0.10	3.34	1.67	139.9	1.73	0.45
SBC-14B	0.08	3.43	1.85	328.5	1.25	0.50
SBC-15A	0.08	5.58	2.69	541.9	1.53	0.50
SBC-15B	0.08	3.07	2.04	455.7	1.18	0.50
SBC-16A	0.08	3.67	1.41	470.6	1.10	0.50
SBC-16B	0.08	5.28	3.18	543.3	1.59	0.50
SBC-17A	0.08	3.69	1.34	363.1	1.13	0.50
SBC-17B	0.12	4.78	3.34	61.2	3.36	0.30
SBC-18	0.08	3.84	1.67	454.5	1.19	0.50
SBC-19A	0.10	3.38	1.68	143.6	1.73	0.45
SBC-19B	0.10	2.22	0.66	169.5	1.07	0.45
SBC-20	0.15	3.20	1.78	45.0	3.14	0.30
SBC-21	0.10	0.46	0.23	278.4	0.41	0.45
SBC-22	0.15	4.40	2.26	43.9	3.79	0.30
SC-1	0.08	3.19	1.51	630.8	1.03	0.50
SC-2	0.12	2.54	1.51	68.4	2.06	0.40
TCC-1	0.08	1.67	0.79	566.8	0.68	0.50
TCC-2	0.08	1.89	0.27	350.8	0.54	0.50
TCC-3	0.15	3.62	2.17	52.5	3.41	0.30
WC-1	0.08	1.29	0.68	406.0	0.63	0.50
WC-2	0.12	1.60	0.69	92.4	1.30	0.40
WC-3	0.12	1.98	0.90	62.5	1.62	0.40

Table A-5 - Gross Reservoir Release Table

Date	Reservoir Elevation (ft)	Reservoir Volume Storage (AF)	Average Outflow (cfs)
8/1/2013	7279.61	40831	183
8/2/2013	7279.3	40705	183
8/3/2013	7278.99	40579	183
8/4/2013	7278.68	40454	183
8/5/2013	7278.28	40292	183
8/6/2013	7277.9	40139	184
8/7/2013	7277.49	39975	184
8/8/2013	7277.1	39819	185
8/9/2013	7276.7	39659	180
8/10/2013	7276.39	39535	154
8/11/2013	7276.03	39392	152
8/12/2013	7275.85	39321	133
8/13/2013	7275.65	39242	139
8/14/2013	7275.48	39174	139
8/15/2013	7275.2	39064	139
8/16/2013	7274.83	38918	139
8/17/2013	7274.49	38784	134
8/18/2013	7274.12	38639	134
8/19/2013	7273.61	38440	159
8/20/2013	7273.07	38229	160
8/21/2013	7272.6	38047	160
8/22/2013	7272.14	37869	160
8/23/2013	7271.61	37664	190
8/24/2013	7271.04	37445	195
8/25/2013	7270.44	37215	194
8/26/2013	7270.03	37059	194
8/27/2013	7269.51	36861	194
8/28/2013	7268.89	36626	194
8/29/2013	7268.34	36419	193
8/30/2013	7267.76	36200	193
8/31/2013	7267.13	35965	193
9/1/2013	7266.58	35760	193
9/2/2013	7265.98	35537	193
9/3/2013	7265.33	35297	193
9/4/2013	7264.71	35068	193
9/5/2013	7264.11	34848	193
9/6/2013	7263.75	34717	169
9/7/2013	7263.35	34571	168
9/8/2013	7262.91	34411	168
9/9/2013	7262.76	34357	169
9/10/2013	7262.86	34393	170
9/11/2013	7264.21	34885	171
9/12/2013	7268.78	36584	46
9/13/2013	7272.63	38058	23
9/14/2013	7274.93	38957	20
9/15/2013	7278.05	40200	20
9/16/2013	7280.54	41210	20
9/17/2013	7281.62	41654	164
9/18/2013	7282.2	41894	201
9/19/2013	7282.35	41956	255
9/20/2013	7282.25	41914	248
9/21/2013	7282.15	41873	227
9/22/2013	7282.11	41856	218
9/23/2013	7281.91	41774	221
9/24/2013	7281.76	41712	201
9/25/2013	7281.62	41654	180
9/26/2013	7281.57	41633	150
9/27/2013	7281.72	41695	117
9/28/2013	7281.79	41724	117
9/29/2013	7281.8	41728	116
9/30/2013	7281.76	41712	115

Date	Reservoir Elevation (ft)	Reservoir Volume Storage (AF)	Average Outflow (cfs)
10/1/2013	7281.76	41712	101
10/2/2013	7281.75	41708	95
10/3/2013	7281.76	41712	92
10/4/2013	7281.81	41732	91
10/5/2013	7281.76	41712	91
10/6/2013	7281.74	41703	90
10/7/2013	7281.67	41675	90
10/8/2013	7281.57	41633	90
10/9/2013	7281.48	41596	90
10/10/2013	7281.42	41572	92
10/11/2013	7281.28	41514	93
10/12/2013	7281.18	41473	89
10/13/2013	7281.08	41432	89
10/14/2013	7280.93	41370	89
10/15/2013	7280.79	41313	88
10/16/2013	7280.64	41251	87
10/17/2013	7280.45	41173	87
10/18/2013	7280.25	41092	91
10/19/2013	7280.06	41014	91
10/20/2013	7279.86	40933	90
10/21/2013	7279.71	40871	90
10/22/2013	7279.52	40794	89
10/23/2013	7279.32	40713	89
10/24/2013	7279.13	40636	88
10/25/2013	7278.83	40514	104
10/26/2013	7278.74	40478	70
10/27/2013	7278.64	40438	66
10/28/2013	7278.63	40433	44
10/29/2013	7278.64	40438	43
10/30/2013	7278.69	40458	43
10/31/2013	7278.64	40438	43

Data provided by Denver Water

Table A-6 - Baseline Reservoir Release Table

Date	Reservoir Elevation* (ft)	Reservoir Volume Storage* (AF)	Total Inflow (cfs)	Total Outflow (cfs)
9/1/2013			9.82	14.37
9/2/2013			9.43	14.69
9/3/2013			7.07	14.40
9/4/2013			2.85	10.89
9/5/2013			3.33	9.43
9/6/2013	30.93	4436	2.21	8.47
9/7/2013			1.85	10.23
9/8/2013			1.74	10.57
9/9/2013	30.90	4436	0.73	9.14
9/10/2013	31.09	4463	0.80	3.89
9/11/2013	32.30	4826	0.00	3.65
9/12/2013			0.00	77.40
9/13/2013	34.00	5326	0.00	75.00
9/14/2013			0.00	75.00
9/15/2013			0.00	75.00
9/16/2013			0.00	75.00
9/17/2013			0.00	75.00
9/18/2013			0.00	74.00
9/19/2013			0.00	74.00
9/20/2013			0.00	74.00
9/21/2013	32.00	4740	0.00	74.00
9/22/2013			0.00	74.00
9/23/2013			0.00	74.00
9/24/2013			0.00	74.00
9/25/2013			0.00	41.00
9/26/2013	29.42	4038	0.00	41.00
9/27/2013			0.00	0.00
9/28/2013			0.00	0.00
9/29/2013			0.00	0.00
9/30/2013	29.00	3933	0.00	0.00

Provided by Baseline Reservoir Company via City of Lafeyette

Table A-7

Comparison of 10 Day Modeled Discharges to Observed Discharges on Boulder Creek Tributaries

HMS Node	Site Description	Drainage Area (mi^2)	Observed Peak Discharge (cfs)	Modeled Peak Discharge (cfs)	% Difference
BCC-1	Bear Canyon Creek at Mohawk Drive	4.56	1,330	264	-80.2%
CC-J1	Coal Creek downstream of Bear Creek	9.30	1,110	1,064	-4.1%
CC-J2	Coal Creek near Plainview (UDFCD site)	15.2	3,900	1,804	-53.7%
CC-J6	Coal Creek at US Highway 287	35.1	3,000	3,628	-27.4%
CC-J7A	Coal Creek at 120th Street	35.7	3,500	3,625	3.6%
CC-J10	Coal Creek at Erie Parkway	76.9	6,000	4,778	-20.4%
FCC-J1	Fourmile Canyon Creek near Sunshine, CO	1.80	1,090	667	-38.8%
FCC-J1	Fourmile Canyon Creek at Pinto Drive	4.10	1,080	667	-38.2%
FCC-J2	Fourmile Canyon Creek upstream of Broadway Avenue	7.68	1,460	1,102	-24.5%
FCC-J2 / FCC-J3	Fourmile Canyon Creek upstream of Diagonal Highway	10.4	2,300	1,233	-46.4%
FMC-J2 / FMC-J3	Fourmile Creek upstream Burned Area	9.0	490	370	-24.5%
FMC-J3 / FMC-J4	Fourmile Creek downstream of Emerson Gulch	14.7	1,070	885	-17.3%
FMC-J6	Fourmile Creek near Orodell, CO (Jarrett, in press)	21.4	2,300	2,284	-0.7%
FMC-J6	Fourmile Creek near Orodell, CO (USGS manual measurement)	21.4	2,510	2,284	-9.0%
MBC-J5	Middle Boulder Creek at Nederland, CO	36.2	409	409	0.0%
NBC-J7	North Boulder Creek at Confluence with Middle Boulder Creek	36.0	740	739	-0.1%
RC-J4	Rock Creek at 120th Street	21.8	1,500	1,114	-25.7%
SBC-J4	South Boulder Creek at Rollinsville, CO	45.2	410	339	-17.3%
SBC-J7	South Boulder Creek near Pinecliffe, CO	72.7	781	717	-8.2%
SBC-J10A	South Boulder Creek below Gross Reservoir	95.8	285	239	
SBC-J12	South Boulder Creek at Eldorado Springs	111	2,120	1,247	-41.2%
SBC-J14	South Boulder Creek at Highway 93	123	5,600 [a]	1,991	-64.4%
SBC-J15	South Boulder Creek at South Boulder Creek Road	134	3,600 [a, b]	2,164	-39.9%
TCC-J1	Twomile Canyon Creek near North Cedar Brook Road	1.10	1,210	184	-84.8%
WC-J1	Wonderland Creek at 15th Street	1.60	170	133	-21.8%

cfs = cubic feet per second

[a] Possibly affected by railroad embankment breach on Doudy Draw

[b] May not account for overtopping of road and upstream flow splits to Dry Creek No. 2 and Baseline Reservoir

Table A-8

Comparison of 10 Day Modeled Times-of-Peak Discharges to Observed Times-of-Peak Discharge on Boulder Creek Tributaries

HMS Node	Site Description	Observed Time-of-Peak Discharge	Modeled Time-of-Peak Discharge	Difference
CC-J2	Coal Creek near Plainview (UDFCD site)	9/12 22:30	9/12 22:25	-5 min (early)
FCC-J1	Fourmile Canyon Creek near Sunshine, CO	9/12 00:50	9/12 1:35 [a]	+45 min (late)
FMC-J6	Fourmile Creek near Orodell, CO	9/12 22:45	9/12 22:25	-20 min (early)
MBC-J5	Middle Boulder Creek at Nederland, CO	9/13 02:15 [b]	9/13 5:40	+3h 25 min (late)
SBC-J7	South Boulder Creek near Pinecliffe, CO	9/13 04:30	9/13 4:30	± 0 min
SBC-J10A	South Boulder Creek below Gross Reservoir	9/11 23:00	9/11 23:40	+40 min (late)
SBC-J12	South Boulder Creek at Eldorado Springs	9/12 22:00	9/12 22:20	+20 min (late)
SBC-J14	South Boulder Creek at Highway 93	9/12 19:40 [c]	9/12 23:30	+2h 50 min (late)
SBC-J15	South Boulder Creek at South Boulder Creek Road	9/12 19:25 [c]	9/12 23:00	+2 hr 35 min (late)

[a] Model junction is two miles downstream of gage where time-of-peak observed

[b] Observed peak of 409 cfs was measured from 02:00 to 02:30; HEC-HMS results report the earliest time (02:00)

[c] Possibly affected by railroad embankment breach on Doudy Draw

Table A-9

10-day Boulder Creek Model - Reach Routing Parameters

Reach ID	Stream Name	L *ft*	Slope *ft/ft*	$n_{channel}$	$n_{overbank}$	Invert *ft (NAVD88)*
BC-R1	Boulder Creek	11445	0.0524	0.070	0.080	6388.3
BC-R2	Boulder Creek	8193	0.0342	0.070	0.080	6123.0
BC-R3	Boulder Creek	4632	0.0173	0.070	0.080	6005.0
BC-R3A	Bummer's Gulch	13492	0.0563	0.065	0.080	6757.9
BC-R4	Boulder Creek	7796	0.0257	0.070	0.080	5785.5
BC-R5	Boulder Creek	16010	0.0235	0.040	0.055	5568.6
BC-R6	Boulder Creek	13215	0.0084	0.070	0.080	5311.4
BC-R7	Boulder Creek	6391	0.0057	0.100	0.150	5199.1
BC-R8A	Boulder Creek	4265	0.0050	0.100	0.150	5184.1
BC-R8B	Boulder Creek	3467	0.0041	0.100	0.150	5158.2
BC-R9	Boulder Creek	11716	0.0038	0.060	0.075	5124.7
BC-R10	Boulder Creek	15421	0.0032	0.040	0.045	5083.3
BC-R11	Boulder Creek	15423	0.0037	0.030	0.035	5021.5
BC-R12	Boulder Creek	14757	0.0033	0.030	0.035	4987.4
BC-R13	Boulder Creek	13902	0.0028	0.030	0.035	4933.1
BC-R14	Boulder Creek	21653	0.0030	0.030	0.030	4902.1
CC-R1	Coal Creek	26763	0.0415	0.070	0.080	7047.7
CC-R2	Coal Creek	22350	0.0272	0.035	0.055	5912.7
CC-R3	Coal Creek	30661	0.0142	0.035	0.055	5717.0
CC-R4	Coal Creek	19621	0.0102	0.035	0.055	5328.7
CC-R5	Coal Creek	8792	0.0081	0.035	0.055	5261.0
CC-R6A	Coal Creek	13656	0.0061	0.035	0.055	5133.1
CC-R6B	Coal Creek	18238	0.0022	0.035	0.055	5101.3
CC-R7	Coal Creek	11064	0.0021	0.035	0.055	5078.3
CC-R8	Coal Creek	19260	0.0021	0.035	0.055	5022.9
CC-R9	Coal Creek	19946	0.0029	0.035	0.055	4991.8
DC-R1	Dry Creek No. 3	8643	0.0052	0.040	0.055	5224.5
DC-R2	Dry Creek No. 3	25561	0.0069	0.040	0.055	5105.1
FCC-R1	Fourmile Canyon Creek	12642	0.0374	0.040	0.055	5724.6
FCC-R2	Fourmile Canyon Creek	25796	0.0141	0.040	0.055	5236.2
FMC-R1	Fourmile Creek	14925	0.0911	0.055	0.070	8581.6
FMC-R2	Fourmile Creek	14384	0.0250	0.055	0.070	7503.7
FMC-R3	Fourmile Creek	15676	0.0408	0.055	0.070	6704.8
FMC-R4	Fourmile Creek	10089	0.0436	0.080	0.100	6283.9
FMC-R5	Fourmile Creek	12495	0.0288	0.080	0.100	5889.2
MBC-R1	Jasper Creek	11643	0.0653	0.040	0.055	9260.5
MBC-R2	N. Fk. Middle Boulder Creek	21143	0.0530	0.040	0.055	9575.8
MBC-R3	Middle Boulder Creek	14376	0.0278	0.040	0.055	8638.9
MBC-R4	Middle Boulder Creek	18676	0.0223	0.040	0.055	8437.0
MBC-R6	Middle Boulder Creek	31544	0.0413	0.070	0.080	8187.5

NBC-R1	North Boulder Creek	13643	0.0645	0.070	0.080	9817.8
NBC-R2	Caribou Creek	3983	0.0854	0.070	0.080	9624.6
NBC-R3	North Boulder Creek	20694	0.0599	0.070	0.080	8557.2
NBC-R3A	Trib. to North Boulder Creek	12188	0.0328	0.070	0.080	8327.9
NBC-R4	North Boulder Creek	20713	0.0280	0.070	0.080	7791.5
NBC-R5	North Boulder Creek	12008	0.0533	0.070	0.080	7348.7
RC-R1	Rock Creek	14315	0.0118	0.035	0.060	5461.5
RC-R2	Rock Creek	17124	0.0051	0.035	0.060	5306.6
RC-R3	Rock Creek	29049	0.0046	0.035	0.060	5198.2
SBC-R1	South Boulder Creek	10007	0.0162	0.040	0.055	8898.8
SBC-R2	South Boulder Creek	4074	0.0168	0.040	0.055	8815.5
SBC-R3	South Boulder Creek	20472	0.0192	0.040	0.055	8424.1
SBC-R4	South Boulder Creek	4883	0.0130	0.040	0.055	8322.2
SBC-R5	South Boulder Creek	12963	0.0167	0.040	0.055	8140.1
SBC-R6	South Boulder Creek	12430	0.0161	0.040	0.055	7980.8
SBC-R7	South Boulder Creek	11344	0.0510	0.040	0.055	7546.1
SBC-R9	South Boulder Creek	5061	0.0363	0.040	0.055	6915.2
SBC-R10	South Boulder Creek	15864	0.0234	0.040	0.055	6649.8
SBC-R11	South Boulder Creek	19914	0.0335	0.040	0.055	6053.8
SBC-R12	South Boulder Creek	8588	0.0194	0.040	0.055	5663.4
SBC-R13	South Boulder Creek	11061	0.0122	0.100	0.150	5530.2
SBC-R14	South Boulder Creek	10760	0.0107	0.100	0.150	5404.8
SBC-R15	South Boulder Creek	21496	0.0081	0.100	0.150	5284.8
SC-R1	Skunk Canyon Creek	12399	0.0124	0.040	0.055	5254.3
TCC-R1	Twomile Canyon Creek	8747	0.0592	0.040	0.055	5528.0
TCC-R2	Twomile Canyon Creek	17950	0.0136	0.030	0.035	5305.0
WC-R1	Wonderland Creek	6752	0.0154	0.040	0.055	5418.4
WC-R2	Wonderland Creek	11960	0.0123	0.030	0.035	5293.6

Appendix F

Project Correspondence and Response to Review Comments

CDOT Review Comments on Draft Lower St. Vrain Phase 2 Report

Provided on April 6, 2015

Response to Review Comments by Jacobs Team

1. Did we decide on an appropriate number of significant figures that we'd like to include in the proposed hydrology estimates? I noticed that all the Phase 2 reports thus far have displayed hydrology estimates down to the singles value. Likely, we should be rounding to a set number of significant figures for the final results, with the exact values perhaps displayed in the appendices?
   All reported peak discharge values in the report have been rounded to three significant figures. The peak discharge values in the appendices have remained as exact values from the model output.

2. On page ES-5, 2nd paragraph - should perhaps clarify that the 100 year recurrence peak Q for St. Vrain during the flood is only for the phase 2 study area. Reference Phase 1 report for the estimated recurrence for that study area.
   Clarification was added to the report to indicate that the estimated recurrence intervals pertain to the lower reaches of the watersheds, specifically the Phase 2 study area.

3. Figure ES-2 may be a little too busy to "squeeze" into such a small space. Same note for Figures 4 and 5. It seems like perhaps these figures could have their own page? They're very good graphical representations, but can be hard to decipher if too small.
   The small versions of the figures were embedded in the report for the reader's convenience so they don't have to turn to a special section for figures or to the appendices to follow along with the discussion. Larger versions of the figures are included in Appendices D.6 and D.7. Text has been added to the relevant sections in the report to bring this to the reader's attention and to direct them to the appendices if they wish to look at the larger versions for more detail.

4. Page 27, last paragraph - include a footnote or other type of citation noting the page numbers in the Phase I report.
   Referenced Section 2.2 from Phase 1 Report which discusses the HEC-RAS analysis of the Highway 36 bridge below Lyons.

5. Table 7, Ayres values at Airport Rd for the 500 yr - is the value of 186,890 cfs correct? If so, should we consider just omitting that value from the FFA results? This does not seem like a particularly credible result.
   The value is correct based on the FFA of this gage which has a short period of record. The gage analysis results were kept in the report because the 10-year discharge estimate was reasonable and matched well with the modeled 10-year results. The report discusses these details on page 11, in the first paragraph after the bulleted gages.

CWCB Review Comments on Draft Lower St. Vrain Phase 2 Report

Provided on April 8, 2015

Response to Review Comments by Jacobs Team

1. Label Figures, extra blank page between Figure 1 & page 6.
   The figures have been labeled as Figure 1 in report and Figure D.1 in Appendix D. The blank page was removed between Figure 1 and Page 6. The blank page was originally meant for double sided printing.

2. Overall, I thought the report was fairly easy to understand and follow. Some of their table summaries were practical and easy to follow, such as Table ES-1 comparing modeled to regulatory discharges and Table ES-1 estimates of the recurrence Interval. Other Tables, such as Table 4 were not so clear and easy to understand. An HMS drainage basin map and a stream gage location map would be helpful.
   A reference to Figure D.1 in Appendix D was added to the text for Figure 4 to direct the reader to the map showing individual basin delineations. Stream gage locations were also added to Figure D.1 and the figure is referenced in Section 2.3 during the FFA discussion.

3. Appendix D needs to be updated. The figure provided is of the Big Thompson Watershed, <u>not</u> St. Vrain and there are no other basin maps provided. Appendix D.1 has ridiculously tiny text. A Stream gage location map would be helpful to reference in the section on FFA. The figures are not labeled or referenced specifically in the text. Also, I notice a lot of blank pages either after a figure or after a title page. Not sure if it's intentional or not.
   The correct St. Vrain Figure was placed in Appendix D and labeled as Figure D.1. Stream gage locations were also added to Figure D.1. The report text was updated in several locations to add references to Figure D.1. The blank pages after the figures were intentional for purposes of double sided printing. However, they have been removed from the pdf to make it easier to read the electronic version. Appendix D.1 along with Appendices D.4, D.5, D.6, and D.7 have been reformatted to 11x17 to make the text size and graphics larger and easier to read.

4. Figure ES-1 should include a legend similar to that of Figure 7. Acronyms are a pain, and they hurt slightly less than the need to comb through text to find the meaning behind them.
   The legend was intentionally not included in the Executive Summary to limit the number of pages. However, the page number in the body of the report where the legend can be found has been added to the executive summary so that the reader can more easily identify the stream name acronyms.

5. This one may sound pretty picky, and it doesn't bother me if it isn't changed, but I believe "Left Hand" Creek is traditionally two words as opposed to "lefthand".

The spelling is found both ways in various sources, however the FIS and the Floodplain Information Reports that were relied upon for development of this report both show Lefthand as one word.

6. Far too many significant figures in the final modeled discharge results. Given the input data, each modeled value should be reported to 2 significant figures (ex. 24,967 cfs should be 25,000 cfs).
   This comment was provided by several reviewers and it was agreed upon by both project teams that all reported peak discharge values in the report text would be rounded to three significant figures. The peak discharge values in the appendices have remained as exact values from the model output.

7. **Sec. 1.1:** There were numbering errors for "primary tasks…" and for the six Phase I reports.
   The errors were fixed and the numbering was restarted from one in both locations.

8. **Sec 2.4.3:** Third paragraph, second sentence explaining the Snyder Unit Hydrograph can be stricken.
   This sentence was shortened to simply state that the method requires two input parameters.

9. **Sec. 2.5.3:** pg. 25, the word "reasonableness" just seems odd. It could be stricken and the sentences wouldn't change meaning.
   The word "reasonableness" was removed from the sentence.

10. **Sec. 3.0:** Presenting the contents of the table in Appendix D.5 as a list would prevent repetition of the phrase "The expanded table in Appendix D.5 also includes…"
    The text in this section was completely reorganized to eliminate the repetition of the phrase.

Draft Lower St. Vrain Watershed Phase 2 Hydrologic Evaluation
Post September 2013 Flood Event
Prepared by Jacobs for the Colorado Department of Transportation
March 2015

<u>Review comments by Will Thomas, Michael Baker International, on behalf of FEMA</u>

Response to Review Comments by Jacobs Team

Background

Hydrology analyses were performed on St. Vrain Creek using a HEC-HMS model from Lyons, Colorado to the confluence with the South Platte River. The watershed upstream of Lyons was studied in Phase 1 of this project. The HEC-HMS model was calibrated to estimates of the September 2013 flood event made at 10 locations along St. Vrain Creek and Lefthand Creek. Once the HMS was reasonably calibrated to the September 2013 flood, NOAA Atlas 14 rainfall data were used to estimate the 10-, 4-, 2-, 1- and 0.2-percent chance flood discharges. Depth area reduction factors (DARFs) from a study by Applied Weather Associates were used to adjust the point rainfalls from NOAA Atlas 14 to be indicative of the rainfall over the respective watershed area.

The hydrologic modeling procedure that was developed and used in the Phase 1 study was used in Phase 2 that included:

- Calibrating the HMS model to the 10-day rainfall for the September 2013 flood event, and
- Adjusting the runoff curve numbers and using the maximum 24-hour rainfall for the 2013 flood to get reasonable calibration results.

Specific Comments

In hindsight, the Applied Weather Associates (AWA) study should have been conducted prior to the Phase 1 study. The DARFs from the AWA study are much more applicable to the Foothills Region than the NOAA Atlas 2 values used in the Phase 1 study. However, a reasonable adjustment procedure was developed to transition from the DARFs used in Phase 1 (NOAA Atlas 2) to those developed in the AWA study. Obviously, the funding and time are not available to update the Phase 1 studies.

The following comments are minor but are intended to improve the quality of the report.

1. Page ES-3 – The discharges in Figure ES-1 are not unit discharges. The word "unit" should be deleted.
   The word "unit" was removed from Figure ES-1 and from Figure 7.

2. Page ES-5 – Drainage areas should be added to Table ES-2 in order to better assess the reasonableness of the flood discharges.
   Drainage areas were added to Table ES-2 and Table 9.

3. Page 5 – The Phase 2 area in Figure 1 should be enlarged or shown in a separate figure to better illustrate the Phase 2 watershed area. A Figure 1 caption needs to be added to the figure. Also note that the larger Phase 2 watershed area map shown in Appendix D is actually for the LowerBig Thompson Watershed and not the Lower St. Vrain Creek watershed. This needs to be revised.

   A Figure 1 caption was added to Figure 1 in the report section. The correct figure for St. Vrain Creek was added to Appendix D as Figure D.1. The Phase 2 area is shown on Figure D.1 in more detail than on Figure 1.

3. Page 10 – In the first sentence under Section 2.2, St. Vrain Creek is omitted. It should read "… within the Phase 2 portion of the Big Thompson and St. Vrain Creek watershed following the September 2013 storm event".

   This sentence was corrected to refer to the St. Vrain watershed.

4. Page 10 – The current version of Bulletin 17B is dated March 1982. The September 1981 version of Bulletin 17B was published by the U.S. Water Resources Council and had many typographical errors.

   The date was changed to March 1982.

6. Page 11 – Rather than state the "2013 peak flow was treated as an outlier in the FFA results", it would be better to state that the "2013 peak flow was adjusted with historical information in the FFA results". Someone might get the wrong impression that this peak was omitted from the analysis.

   The language in this section was changed to state that the outlier treatment means that the peak is adjusted based on historical information.

7. Page 12 – For the FFA analysis for St. Vrain Creek at the mouth (067310 or SVCPLACO), 109 years was used for the historical adjustment period (1905 to 2013) for the September 2013 flood event. This is reasonable and 109 years should be used for the historical period in the FFA analyses for station SVLONGCO and station 06725450 (SVCBLOCO). All three stations are on the same stream and it is obvious that the September 2013 flood was the highest in at least 109 years or since 1905. It is not very informative to provide flood discharges based on 12 years of record for station SVLONGCO because they are unreasonably high. If a historical period of 109 years were used in the frequency analysis, the flood discharges would be more reasonable.
   We examined the three gages (SVC at the mouth, SVC below Longmont, and DWR SVC at Longmont) in response to the comment. We have opted not to incorporate the recommendation, which was to change the historic period to 109 years at the two upstream gages, to match the historical period at the gage at the mouth. This decision is due partly to the fact that the available gage data does not provide us with strong enough evidence to be certain that the 2013 flood was the largest peak discharge at the gage location in the last 109 years. Additionally, eight of the highest ten peaks recorded at the gage at the mouth were recorded before the other two gages existed. It seems likely that those pre-gage floods were significant in Longmont, and that any flood frequency analysis at that location intending to extend back 109 years in history would be of questionable quality if it had no peak discharge entries for those floods. The resulting analysis would likely underpredict the flood discharge for a range of recurrence probabilities.

8. Appendix B – The flood frequency curves in Appendix B using the "Ordered Distribution of Annual Peaks" do not seem very informative. The usual flood frequency graphs from HEC-SSP would be more informative and should be included in Appendix B.
   The flood frequency graphs from HEC-SSP were added to Appendix B.

The conclusions of the St. Vrain Creek hydrologic analysis are:

- The results of the current rainfall-runoff model using the 24-hour NOAA rainfall are viewed as suitable for use by CDOT in the design of permanent roadway improvements along St. Vrain Creek.
- It is recommended that the model results be considered for adoption as the updated regulatory peak discharges along St. Vrain Creek.

These are reasonable conclusions. The updated flood peak discharges from the HEC-HMS model are more reasonable estimates than the effective discharges used in previous mapping and the HMS discharges should be used in future floodplain mapping.

Will Thomas
Michael Baker International
April 1, 2015

CH2M-Hill Review Comments on Draft Lower St. Vrain Phase 2 Report

Provided on April 6, 2015

Responses to Review Comments by Jacobs Team

General Comments

1. Adding a Vicinity Map to the Executive Summary would be helpful to orient the reader.
   A reference to Figure 1 in Section 1.2 of the report was added to the Executive Summary. A link was also provided in the PDF to take the reader directly to Figure 1

2. Suggest adding bookmarks to the PDF for report sections, figures, and appendices.
   Bookmarks were added to the PDF as recommended.

3. Why is the sub-title of the report "Post September 2013 Flood Event"?
   The sub-title was a carryover from the Phase 1 Report title. It indicates that the hydrologic evaluation was initiated after the September 2013 Flood to evaluate the magnitude of the 2013 Flood and to generate updated hydrology.

4. Would recommend adding a table comparing modeled times-of-peak discharge to observed times-of-peak discharge to document the time-calibration of the model.
   Observed peak discharge timing was not available on Lower St. Vrain Creek beyond a rough estimate of the peak timing near the confluence with Boulder Creek at Highway 119. The three active stream gages on the Lower Big Thompson did not record the actual peak of the storm. However, Appendix D.9 provides several plots that compare partial stream gage records (rising/falling limbs) against the modeled hydrographs.

5. Suggest providing a table of estimated September 2013 peak discharge, modeled peak discharge, and percent difference (separate from Table 6) to both the report and the Executive Summary to clearly provide the results of the calibration process.
   Two new tables (Table ES-1 and Table 4) were added to the report to compare the percentage differences between the modeled and observed peak discharges. All other table numbers were shifted accordingly.

6. Between all four studies, are the peak discharges definitively being recommended for adoption, or are they being proposed as the "best estimate" for communities to consider adopting?
   The peak discharges are being recommended as the best estimate for adoption.

7. Numbered lists in Section 1.0 don't begin at 1
   This has been corrected.

8. It is noted that 2001 NLCD data was used; was this dataset verified against present-day land uses?
   The NLCD dataset was not verified against land use prior to generating the composite CN values. However, the composite CN values were compared to present-day land use and then adjusted if they were not representative of the current condition. Furthermore, the initial CN value was used as a calibration parameter and the initial value was adjusted up or down during the calibration process.

9. Phase 1 Hydrologic Analyses should be added as a previous study (as they are referenced as justification for the Phase 2 study later on)
   A discussion of the Phase 1 report dependency was added to section 2.1.

10. Suggest providing a map (or adding to the Vicinity Map) of the location of Phase 2 peak discharge estimates (URS and any Jarrett, stream gage, or other estimates that are referenced in the report as having been used for calibration).
    The peak discharge estimates are identified on Figure 1 and Figure D.1 as "Investigation Sites". Additional text has been added to Section 2.2 to identify the peak discharge estimate locations as Investigation Sites.

11. If not adding a table to Section 2.2, suggest adding a reference to Table 6 where peak discharge estimates are documented.
    A reference was added to Section 2.2 to direct the reader to the new Table 4 for a summary of the peak discharge estimates.

12. Suggest adding standard ASCE references (AUTHOR, year) to the report and indenting and italicizing multiple-line direct quotes from other sources.
    This recommendation was noted.

13. Section 2.4 – suggest adding a graph that shows rainfall across representative areas of the watershed and the timing of peak discharges in relation to the 24-hour calibration window (perhaps Section 2.4.5). Verify that peak 24-hour rainfall was in fact the driver of peak discharges (it was not on Boulder Creek).
    The calibration window was actually a 72-hour period driven by the maximum 24-hour period of rainfall from the 2013 Flood. In other words, one day of rain was input and the runoff was modeled for a three day period. The maximum 24-hour period of rainfall was determined by finding the maximum 24-hour rainfall depth for each of the individual basins in the watershed (performed using a macro (VBA code) developed in Excel). Based on the 24-hour time periods for each of the individual basins, a common 24-hour period that best represented the entire watershed was selected. This 24-hour period was then extracted from the 10-day rainfall record for each basin and entered into the HEC-HMS model. The model simulation was then run for a 72-hour period to make sure that any lags in the peak discharge were captured. Unlike Boulder Creek, peak discharge hydrographs were not recorded on St. Vrain Creek because all of the gages failed. However, Appendix D.9 does include partial hydrographs from the gages that agree well with the timing of the modeled peak discharges.

14. Which reservoirs were modeled and which were not? How was their impact during the September 2013 event handled? Where was information for reservoirs provided from (stage-storage, spillway rating curves, September 2013 releases)? Upon further review, much of this information is provided on page 25 for the Big Thompson report and may be more appropriate prior to presentation of calibrated model results.
    No reservoirs were modeled in the Lower St. Vrain watershed (Phase 2). This is stated at the end of the first paragraph in Section 1.2 – Project Area Description.

15. Can a map be provided that provides the location of hydrologic elements (subbasins, nodes, reaches, reservoirs)? Also would be helpful in clarifying which reservoirs were modeled and which were not.
    The report, Appendix D.1, and Figure D.1 were updated with additional information to help clarify some of comments noted above. However, a map showing nodes and reaches was not developed. The HEC-HMS model includes a background map and along with Appendix D.1 provides the necessary information.

16. How was calibration performed? Was CN adjusted for each subbasin or was it adjusted globally for a given land use?
    The CN value was adjusted for each basin individually as opposed to a global adjustment. Considerations were given to a wide variety of factors when adjusting individual CN values. These factors included the initial value generated from the land use and soil data, current land use conditions, observed downstream peak discharges, observed downstream reservoir volume changes, basin discharge relative to drainage area and 2013 precipitation depth, and unit discharges based on 100-year predictive model results.

17. Suggest re-framing the discussion on "actual flood attenuation". As currently framed, it appears a conclusion is drawn that significant attenuation occurred based on comparison of September 2013 peak discharge estimates which have an inherent uncertainty (random and systematic; both within and between different estimate methodologies) and then an additional parameter (channel losses) is added to the model to replicate that effect. Such an approach may "absorb" the uncertainty associated with September 2013 peak discharge estimates into the rainfall-runoff model. A better approach may be to present physical occurrences that had a significant impact on the magnitude of peak discharges (split flows near Hygiene), how the occurrence is not explicitly modeled by HMS (hydraulic split flow condition / berm breach), and how it was represented in HMS (channel losses). Presentation of 2013 flood extents (documenting splits flows), FIS profiles (documenting backwaters at bridges), and location of headgates and diversions with evidence of overtopping may help strengthen the discussion.
    These are all good recommendations and the text in this section has been updated to try and emphasize these points.

18. Related to the previous comment, the adopted approach in effect introduces a subjective calibration parameter to the model and creates a situation where the subjective calibration parameter can “offset” other calibration parameters. For instance, the calibrated CN for developed land uses (agricultural and urban) which dominate the watershed area downstream of where flows begin to attenuate may not truly be calibrated: the selected CNs may be too low, and offset by a low channel loss, or too high, and offset by a high channel loss, yet still achieve the same peak discharge estimate. Can further justification for the selected channel losses be provided?
    This concern was realized early on in the calibration and taken into account through an iterative calibration approach between the 2013 Flood model and the predictive model. After initial calibration of the 2013 Flood model, the resulting CN values were plugged into the predictive model. The predictive model was then used to further calibrate the individual basin CN values by comparing the 100-year unit discharges for each basin relative to basin shape, slope and land use irrespective of the routed channel flows. The revised CN values were then plugged back into the 10-day model (reverse process of runoff/rainfall ratio adjustment for 24-hour to 10-day values) and the channel loss parameter was further adjusted to match the observed peak discharges. This process was repeated 2 to 3 times in some locations to check the sensitivity of the CN values versus the channel loss values and to aid is selection of appropriate final values.

19. Discussion of AWA curve – stating that the stepped curve is conservative (assuming the AWA curve is most accurate) may address future comments on this unique analysis.
    A sentence was added to point out that the transition curve was conservative with respect to the AWA curve.

20. The 10-day and 24-hour model are first mentioned in Section 2.5.3, but are mentioned as if the reader knows which was used first and why there were two. Can this discussion be added to the overall modeling approach to help linearize the “story-line”?
    A few sentences were added to the end of Section 2.4.1 to explain the 10-day model, the 24-hour model used as a transition, and the predictive storm model. This new text also references Section 2.5.3 where the detailed process is explained.

21. When Appendix F is added, suggest including Phase 1 Responses.
    Phase 1 Responses are included along with Phase 1 report and can be taken out of context if read without having the Phase 1 report, therefore they are not included in Appendix F of the Phase 2 Report.

22. Will Jacobs be rounding to the 3-significant figures as well?
    Yes, the peak discharges presented in the report text have been rounded to three significant figures.

<u>St. Vrain Creek Comments</u>

1. Figure ES-1 provides several regulatory discharges for Boulder Creek; there are only 4 reported in the FIS (plus two on Middle Boulder Creek), the remaining are generated from the same USACE, 1977 model, but not reported in the FIS.
   It is typical for FIS reports to condense and summarize data with providing all data from the original reports.

2. Section 1.2 – provided CH2M Hill memo documents the calibration of the *10-day* calibration of Boulder Creek and should be considered independent of the 24-hour calibration that forms the basis of the *Boulder Creek Hydrologic Evaluation*. Please revise the last sentence of this paragraph to clarify.
   Clarification was added to Section 1.2.

3. It looks like the Big Thompson Workmap (Appendix D, PDF page 195) is in place of the St Vrain workmap. I would label tributaries that are discussed in the report
   The appropriate St. Vrain map has been inserted as Figure D.1. The named tributaries have been added to Figure D.1.

4. Please provide further explanation for the use of 0.15 for Manning's n.
   The default values were initially set to 0.05 for the channel and 0.10 for the overbank areas. As part of the calibration, some of the values were adjusted up to 0.15 in the Lefthand Creek tributary. An upper limit of 0.15 was placed on Manning's n for calibration purposes based on Jarrett's 1985 Report titled "Determination of Roughness Coefficients for Streams in Colorado". Review of that paper showed several channels that had Manning's n values between 0.10 and 0.15. It lists a maximum n value of 0.14 for channels not maintained, weeds and brush uncut, with a high flow stage. Other sources were also checked including the USGS Water Supply Paper 2339 "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" and Ven Te Chow (1959) Open Channel Hydraulics. These sources included overbank area Manning's n values as high as 0.20 for areas with dense areas of trees.

5. How do the 1972 USACE peak discharge estimates (which would have used a gage record that was predominantly pre-Button Rock Reservoir) compare to modeled estimates?
   The 1972 USACE 100-year peak discharge estimate at the Lyons gage (10,200 cfs) was higher than the current regulatory discharge (8,880 cfs) and the revised FFA peak discharge (8,840 cfs) but lower than the predictive model estimate (12,100 cfs). This supports the idea that Button Rock has had an impact on the gage record in Lyons and has reduced the estimated discharges.

6. Can the discussion of the “Muskingum-Cunge method, as well as several of the other HEC-HMS routing options” not providing peak flow attenuation be elaborated upon? All methods provide peak flow attenuation, so is the concern the degree of attenuation in comparison to what actually occurred?
   Many of the routing methods focus on peak translation as opposed to peak attenuation. Chapter 8 in the HEC-HMS Technical Reference Manual discusses the applicability and limitations of the different routing models available in HEC-HMS.

   - In terms of backwater effects it states that “Only the modified Puls model can simulate backwater effects, and it can do so in only the case of time-invariant downstream conditions….the effects of the backwater must be determined and included when developing the storage-discharge relationship.” As discussed in the report, we do not have the information required to develop the storage-discharge relationship for each reach.
   - In terms of floodplain storage it states that “If flood flows exceed the channel carrying capacity, water flows into overbank areas. Depending on the characteristics of the overbanks, that overbank flow can be slowed greatly, and often more ponding will occur. This can be significant in terms of translation and attenuation of a flood wave.” It goes on to state that “flood flows through extremely flat and wide floodplains may not be adequately modeled as one-dimensional flow….a two-dimensional model will better simulate the physical processes.” The two routing models it lists that can reflect overbank floodplain areas are modified Puls and Muskingum-Cunge. As noted above, we do not have the storage-discharge curves to use the modified Puls. Therefore, the Muskingum-Cunge method was selected with an 8-point cross-section.

   The Muskingum-Cunge routing method is based on a combination of the conservation of mass and the diffusion representation of the conservation of momentum. It represents attenuation of flood waves and can be used in reaches with a small slope. The relatively steep slopes along these channels limit the attenuation that can be achieved. Also, the channel slope is used as an input in place of the energy slope which is used in the calculations. It assumes the same slope for the entire cross-section, as opposed to flatter slopes in the overbanks (gravel pits, ponding areas) versus the main channel. This further limits the attenuation that can be achieved in the overbanks. Calibration attempts were made by flattening the slope, however the results showed a significant change in the peak timing as opposed to the peak magnitude. The conclusion was drawn that the degree of attenuation experienced during the 2013 Flood was not achievable using one of the reach routing methods.

7. Suggest not recommending peak discharges on smaller tributaries for following reasons:
    a. Peak discharges may be driven by different events than those analyzed (e.g., short-duration thunderstorms)
    b. The resolution of the St. Vrain Creek hydrologic model may not adequately reflect the hydrology of smaller tributaries
    c. Based on Table 6, there was no calibration information specific to smaller tributaries (only their net contribution to St. Vrain Creek, which could be achieved via any number of combinations of lag time, peaking coefficient, and CN).
    d. Language recommending a re-study for Dry Creek (South) used on page ES-4 of Big Thompson Report may be appropriate here.

    Additional language was added to the Executive Summary and Conclusion sections to clarify that the predictive peak discharges were focused on St. Vrain Creek and Lefthand Creek and that smaller tributaries should be evaluated separately with shorter, more intense storms.

8. Were CDWR gage records for any of the major diversions evaluated and incorporated into the model? For example, a gage is operated downstream of Union Reservoir.
    Yes, all USGS and CDWR gages in the St. Vrain watershed were evaluated for useful information, including the gage below Union Reservoir. The Union Reservoir gage failed during the peak of the storm and shows a constant 20 cfs discharge from the September 12th to 14th. The peak recorded at the gage during the 10-day period was only 112 cfs on the afternoon of September 15th.

9. Critical storm analysis – was a storm set up downstream of Buttonrock (i.e., part of Phase 1 basin) considered and whatever portions of Phase 2 may be "most critical"?
    An additional critical storm scenario was run to check this. The storm was assumed to cover the lower part of the St. Vrain Creek Phase 1 model below Button Rock Dam (both on North St. Vrain and South St. Vrain) which included an area of approximately 49 square miles (out of 218). The storm was also assumed to cover the lower part of the Phase 1 Lefthand Creek model at about the same north/south boundary as Button Rock Dam (includes James Creek and lower Lefthand Creek) and included an area of approximately 31 square miles (out of 58). The total drainage area contributing runoff upstream of the confluence with Boulder Creek was approximately 227 square miles as opposed to the actual 424 square miles. The DARF adjustments flattened out at 92% and never transitioned into the AWA curve. Even with the increased rainfall depths, the 100-yr peak discharges through Lyons and Longmont were considerably lower than the standard model. In Lyons, the 100-year peak discharge was only 4,800 cfs as opposed to 12,100 in the standard model. On St. Vrain Creek above Lefthand Creek, the 100-year peak discharge was only 11,700 cfs as opposed to 16,200 cfs in the standard model. On St. Vrain Creek at the confluence with Boulder Creek, the 100-year peak discharge was only 18,500 cfs as opposed to 23,200 cfs in the standard model. This scenario was included in the list of scenarios described in the report.

10. Reviewing Figure 5, it almost appears that Phase 1 of Lefthand Creek was over-calibrated and then a drastic channel loss applied to undo that over-calibration. What is the reason for the large discrepancy between the modeled discharge and estimated discharge at Old Stage Road?
    The large discrepancy at Old Stage Road is discussed in detail in the Phase 1 Report. In summary, the estimate at Old Stage Road was determined to be low based on the upstream estimates on James Creek and on Lefthand Creek above the James Creek confluence. Jamestown received some of the most intense rainfall in the area (roughly a 1,000 year depth over 24 hours). Several iterations were evaluated in Phase 1 regarding the timing of peaks at the confluence with James Creek and none of the scenarios closed the gap with the estimated peak discharge at Old Stage Road. Furthermore, the FFA results below Old Stage Road were in close agreement with the predictive model results which further supported the 2013 peak discharge from the model versus the estimate. The large channel loss applied below Highway 36 represents several things. First it reflects the significant amount of sediment deposition observed at the mouth of the canyon where the floodplain width widens abruptly, resulting in a velocity reduction and a drop in the bulking factor present in the flow. The channel loss also represents the shape of the watershed below the canyon mouth (long and narrow with very little tributary drainage area to sustain the peak discharge. Finally, several irrigation ditches and small ponds are located in this area.

11. The peak discharge below Boulder Creek appears to be a combination of Boulder Creek peaking and the receding limb of St. Vrain Creek. Given the previous discussion of applying channel losses due to flow splits, it appears that complete removal of that flow, which presumably had a slower travel time and "re-appeared" on the receding limb of the St. Vrain Creek hydrograph, may have underestimated the modeled contribution of St. Vrain Creek to peak discharges on St. Vrain Creek downstream of Boulder Creek. Do you agree with this observation or evaluate/calibrate this?
    This is an interesting observation, unfortunately there is no reliable information regarding the duration of the peaks or recorded hydrographs to confirm or deny this possibility. The timing of the return flow from splits or diversions could possibly take days to drain back into St. Vrain Creek. The observed peak discharge downstream at I-25 (23,500 cfs) was lower than the modeled peak discharge at the confluence with Boulder Creek (25,700 cfs) so it is unlikely that it was much higher.

12. Could the favorable comparison between predictive model results and FFA on Lefthand Creek be the result of the lack of a major upstream reservoir?
    The lack of a major reservoir definitely improves the reliability of the gage record, however there is a relatively short period of record for this gage, so it is unclear why the comparison of results is as close as it is.

Boulder County

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DATE: June 4, 2015

TO: Colorado Department of Transportation
Colorado Water Conservation Board

FROM: Yige Gao, Floodplain Permitting Specialist
Varda Blum, Floodplain Program Manager

SUBJECT: Phase 2 St. Vrain Watershed Hydrologic Analysis Review Comments

The Boulder County Transportation Department Floodplain Management team has reviewed the Phase 2 St. Vrain Watershed Hydrologic Analysis and has the following comments/questions.

Response to Review Comments by Jacobs Team provided below in red.

1. **Executive Summary ES-3**: "Loss parameters in the rainfall-runoff model were then uniformly adjusted to provide an overall best fit with the estimated September peak discharges based on the peak 24 hours of the September rainfall rather than the entire multi-day storm."

- Please clarify "uniformly adjusted": Based on the report, it seems like the curve numbers were adjusted for individual subbasins. **The word "uniformly" is incorrect, they were individually adjusted. The language in the report was changed to state that the loss parameters "were individually adjusted using a runoff to rainfall ratio for each basin".**

- Please clarify "peak 24 hours" in the report (Maybe in Section 2.5.3 Model Calibration): The report did not specify the maximum 24-hour calibration period, and how the calibration period was selected. Could a discussion and a representative hyetograph be provided in the report? **The maximum 24-hour period is presented in Section 2.4.5 and starts at 1 P.M. (MST) on Wednesday, September 11, 2013. Figure 2 in the same section provides a representative hyetograph which shows the 10-day, max 24-hr, max 6-hr, and max 1-hr hyetographs. The maximum 24-hour period of rainfall was determined by finding the maximum 24-hour rainfall depth for each of the individual basins in the watershed (performed using a VBA macro developed in Excel). Based on the maximum 24-hour time periods for each of the individual basins, a common 24-hour period that best represented the entire watershed was selected. This 24-hour period was then extracted from the 10-day rainfall record for each basin and entered into HEC-HMS for the Max24hr model. A brief description has also been added to Section 2.5.3 in the final report to explain this method and restate the time period.**

2. **Executive Summary ES-5** (subsection on the Boulder Creek confluence)**:** "…This indicates that the two peak discharge hydrographs overlapped but the instantaneous peak discharges were offset slightly."

- As part of the Executive Summary, in order to justify using the 24-hour storm for the predictive model, it might be helpful to briefly summarize the supporting evidence for the hydrograph overlapping in this section, such as similar travel times between Boulder Creek watershed and St. Vrain watershed upstream of Boulder Creek confluence, partial gage records during the 2013 Flood, etc. **Additional text regarding the Boulder Creek confluence timing was added to the Executive Summary to further support the use of the 24-hour storm as recommended.**

3. **<u>Section 2.2 September 2013 Peak Flow Estimates</u>**
- Just for clarification, before getting into the peak flow estimates method, could a paragraph be added to discuss how the gages functioned during the 2013 flood and how the Investigation Sites were selected? **Section 2.2 has been updated to describe how the Investigation Sites were selected and also includes a brief discussion regarding the gages that were utilized and that they washed out during the flood.**

- It might be helpful to follow up with a short discussion here on why only the peak discharges but not the peak timing was calibrated in the study. **A brief discussion was added here that although the peak discharges were not recorded, the timing of the rising limb and the observed stage information were helpful in calibrating the model.**

- It might be helpful to move the list of peak discharge estimation locations from <u>2.4.2 Basin Delineation</u> to this section. **The ten estimate locations are included in Section 2.2.**

4. **<u>Section 2.3 Updated Flood Frequency Analysis</u>**

Three out of four gages have the statement of "The 2013 peak flow was treated as an outlier in the FFA results". These statements need clarification. A record which is "treated" as a high outlier could be adjusted with historical information, or retained in the systematic record without adjustment. Although Bulletin 17B doesn't recommend that high outliers be simply dropped, if the 2013 peak flow at any gage was actually excluded from the FFA, it should also be stated explicitly. **The 2013 peak flows were not excluded from the FFA. Instead they were adjusted based on historical information. Clarification has been added to the report for each of the gages.**

5. **<u>Section 2.4.4 Hydrograph Routing:</u>** "The Manning's n values were initially set to a default of 0.05 for the channels and 0.10 for the overbank areas."
<u>Section 2.4.6 Model Calibration and Validation:</u> "…It was determined that default values ranging from 0.05 to 0.15 were appropriate for the channels in the Phase 2 Study area."

From Section 2.4.6, it was unclear whether the final Manning's n values were default values or were calibrated. Please clarify how the final n values were determined. **The text in Section 2.4.6 was updated to clarify that the Manning's n value was used as a calibration parameter to adjust travel times.**

6. **<u>Section 3 Hydrologic Model Results</u>**: "Figure 6. Confluence of St. Vrain Creek and Boulder Creek in <u>2013 Flood</u>"

The title is a bit misleading. This is not the actual gage record. Could it be clarified in the title that this is from modeling results? **The title has been changed to "Model Results at Confluence of St. Vrain and Boulder Creek in 2013 Flood". Additional language was added to the preceding paragraph to help clarify this also.**

7. **<u>Section 4.0 Conclusion and Recommendations (P34):</u>** "Table 8. Comparison of Peak Discharge Estimates"

The title might be a bit confusing. Could it be clarified to something in line with "Estimate of September 2013 Peak Discharge Recurrence Interval <u>based on Regulatory Discharges</u>", so as to be distinguished from Table 10? **The title was changed to match the recommended title.**

8. **<u>Section 4.0 Conclusion and Recommendations (P35):</u>** "The third step was to calibrate the model using the Curve Number as a calibration parameter to obtain a best fit of the model results to the peak discharge estimates. This model was calibrated to the full 10-day period."

Please clarify here or in Section 2.4.6 Model Calibration and Validation: which parameters were actually calibrated? In Section 2.4.6, the calibration parameters seem to include at least Curve Number and channel loss, if not also channel roughness. In Section 4.0, it sounds like only Curve Number was calibrated. **Section 4.0 was updated to state that the calibration included adjusting Curve Numbers, channel roughness, and channel loss.**

City of Longmont Review Comments on Draft Lower St. Vrain Phase 2 Report

Provided on June 4, 2015

Response to Review Comments by Jacobs Team

From: David Hollingsworth
To: Jim T. Wulliman; Nick Wolfrum
Cc: Steven D. Humphrey; Holly M. Linderholm; kevin.houck@state.co.us; steven.griffin@state.co.us; Derek Rapp; Schram, Heidi; Cory.Hooper@CH2M.com
Subject: RE: St. Vrain discharges
Date: Thursday, June 04, 2015 2:50:27 PM
Attachments: image001.png

Dear Kevin, Steven, and Holly,

Thank you for all of you and your team's efforts in developing new hydrology on St. Vrain Creek. At the City of Longmont we are using these values as we design improvement projects.

We do have one comment that we would like to see addressed in the hydrologic (HEC-HMS) model. Based on the community meeting that was held on May 7th it is our understanding that the Dry Creek #1 basin is modeled to discharge into St. Vrain Creek at the location of Old Dry Creek downstream of Main Street. The correct basin for Dry Creek #1 actually flows to the northeast where it crosses Sunset Street and outfalls to St. Vrain Creek upstream of the BNSF railroad bridge crossing of the Creek. **The basin delineations in the model have been updated to represent the correct outfall for Dry Creek No. 1. This change resulted in one new basin being added to the model. The model, report, and appendices have all been updated to reflect the change.**

We recognize that although this revision may not result in significant changes to the peak flow rate, the difference in outfall location is within the core of the City. It is likely that this hydrologic model will be the regulatory model to be used for decades to come. This is our opportunity to have a model that correctly represents the most significant watershed for the City. We respectfully request the hydrologic model be revised to correctly reflect Dry Creek #1 watershed and outfall location so we can regulate the commercial and downtown development along the St. Vrain through the City with confidence. We also would like this corrected so our near term bridge and channel widening projects are designed to the best possible information available.

Thank you. Sincerely, David

David Hollingsworth, P.E., CFM | Senior Civil Engineer
Storm Drainage & Floodplain Manager

City of Longmont | Public Works & Natural Resources

Weld County Review Comments on 2nd Draft of Lower St. Vrain Phase 2 Report

Provided on June 3, 2015

From:	Tom Parko Jr.
To:	Holly M. Linderholm
Cc:	Steven D. Humphrey; kevin.houck@state.co.us; Steven.Griffin@state.co.us; Diana Aungst; Jennifer Petrik; Michelle Martin
Subject:	Comments on CDOT/CWCB Watershed Reports
Date:	Thursday, June 04, 2015 9:40:03 AM

Holly,

Good morning. Below are the comments from the Weld County Department of Planning Services.

With respect to the reports for Boulder Creek, Little Thompson, Big Thompson and the St. Vrain Creek, staff has reviewed the studies and our questions were sufficiently addressed during and after the meeting on April 8th, followed by two conference calls on May 22nd and June 3rd. We have no further technical comments.

Appendix G

Digital Data (Electronic Only)