Lower Big Thompson Watershed Phase 2 Hydrologic Evaluation

Post September 2013 Flood Event

Prepared for:

Colorado Department of Transportation
Region 4 Flood Recovery Office

Prepared by:

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With Support from:

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We hereby affirm that this report and hydrologic analysis for the Lower Big Thompson 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.

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 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 Big Thompson watershed from the confluence with Buckhorn Creek near the mouth of the canyon to the confluence with 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 the Big Thompson River was the infamous 1976 Big Thompson Flood. In 1981, the regulatory flow rates documented by the Federal Emergency Management Agency (FEMA) in the 2013 Flood Insurance Study (FIS) for Larimer County became effective. The effective peak discharges were developed based on gage records evaluated by the U.S. Army Corps of Engineers (USACE) in 1971. In Weld County, the Preliminary 2013 FIS includes effective peak discharges based on a 1974 USACE study which evaluated a combination of gage records and discharge-probability relationships developed using unit hydrographs.

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 Big Thompson 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
Big Thompson at Wilson Avenue	22,000	22,300	1%
Big Thompson at Hwy 287	22,000	21,300	- 3%
Big Thompson at I-25	19,600	19,800	1%
Big Thompson above Little Thompson Confluence	17,700	17,300	- 2%
Big Thompson at South Platte River	24,900	27,500	10%
Little Thompson River at Big Thompson River	18,000	16,200	- 10%
Dry Creek (South) at Big Thompson River	2,450	2,580	5%

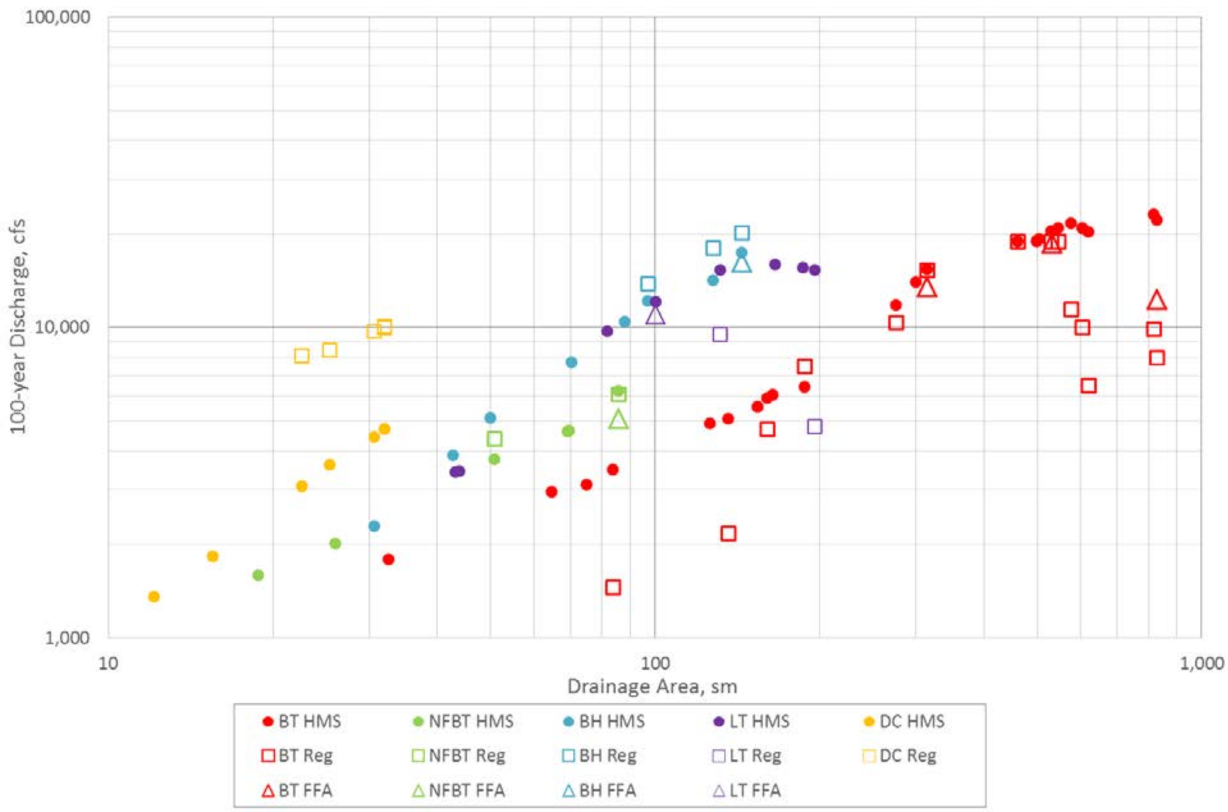
Loss parameters in the rainfall-runoff model were then individually adjusted using a runoff to rainfall ratio for each basin to provide an overall best fit with the estimated September peak discharges based on the peak 24 hours of the September rainfall rather than the entire multi-day storm. This was 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.

The model includes a level of flood attenuation in Boyd Lake, Carter Lake, Flatiron Reservoir and Pinewood Reservoir by assuming the reservoirs are full prior to the start of the storm and routing the inflow through the overflow spillways. 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 peak discharges on the main stem and large tributaries.

The predictive model peak discharges for the various return periods were compared to the results of an updated flood frequency analysis for the Big Thompson River, as well as to current regulatory discharges. This information is shown in Figure ES-1 and Table ES-2 for the 100-year event. The figure, including legend abbreviations, is discussed in detail on page 30; however, several observations can be made:

1. Compared to the modeled discharges, more scatter is associated with the current regulatory discharges, particularly on the Big Thompson River.
2. The current regulatory discharges on the Big Thompson River downstream of I-25 appear low relative to the overall trend.
3. The current regulatory discharges for Dry Creek (South) appear high relative to regulatory discharges for adjacent watersheds and relative to the predictive model results.

Figure ES-1. Comparison of 100-year Discharges in the Big Thompson Watershed



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 the Big Thompson River. 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.

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

Location	Current Regulatory Discharge (cfs)	Modeled Discharge (cfs)	Percent Difference
Big Thompson at Buckhorn Creek	19,000	18,900	- 1%
Big Thompson at Railroad Avenue	19,000	20,400	7%
Big Thompson downstream of County Road 9E	19,000	21,000	11%
Big Thompson at Interstate 25	11,500	21,800	90%
Big Thompson upstream of County Road 15 ½	10,000	21,000	110%
Big Thompson above Little Thompson Confluence	6,500	20,300	212%
Big Thompson below Little Thompson Confluence	9,900	23,200	134%
Big Thompson at South Platte River	8,000	22,200	178%
Little Thompson River at Big Thompson River	4,800	15,400	221%
Dry Creek (South) below Pinewood Reservoir	300	400	33%
Dry Creek (South) at Skinner Gulch Confluence	2,920	1,060	- 64%
Dry Creek (South) below Flatiron Reservoir	8,130	3,070	- 62%
Dry Creek (South) at County Road 29	8,430	3,610	- 57%
Dry Creek (South) at County Road 23E	9,720	4,440	- 54%
Dry Creek (South) at Big Thompson River	10,090	4,710	- 53%

Since the rainfall/runoff model results are more consistent in terms of peak discharge translation downstream than the current regulatory flows in Big Thompson downstream of I-25 and because there does not appear to be any justification for the sharp drop in regulatory peak discharges between County Road 9E and Interstate 25, it is recommended that the model results be considered for adoption as the updated regulatory peak discharges along the Big Thompson River.

It should be noted that this study was focused on peak discharge estimation in the Big Thompson River 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. Dry Creek (South) is an example where there is a significant difference between the predictive peak discharges and the current regulatory discharges. However, since the 100-yr unit discharges for the current regulatory values on Dry Creek (South) are high (315 cfs/mi^2) relative to predictive model unit discharges (150 cfs/mi^2) and to unit discharges in adjacent watersheds as shown on Figure ES-1 and because the 2013 flood was less than a 10-year storm when compared to the current regulatory values, it is recommended that this area be considered for a more focused study and re-evaluation of regulatory peak discharges.

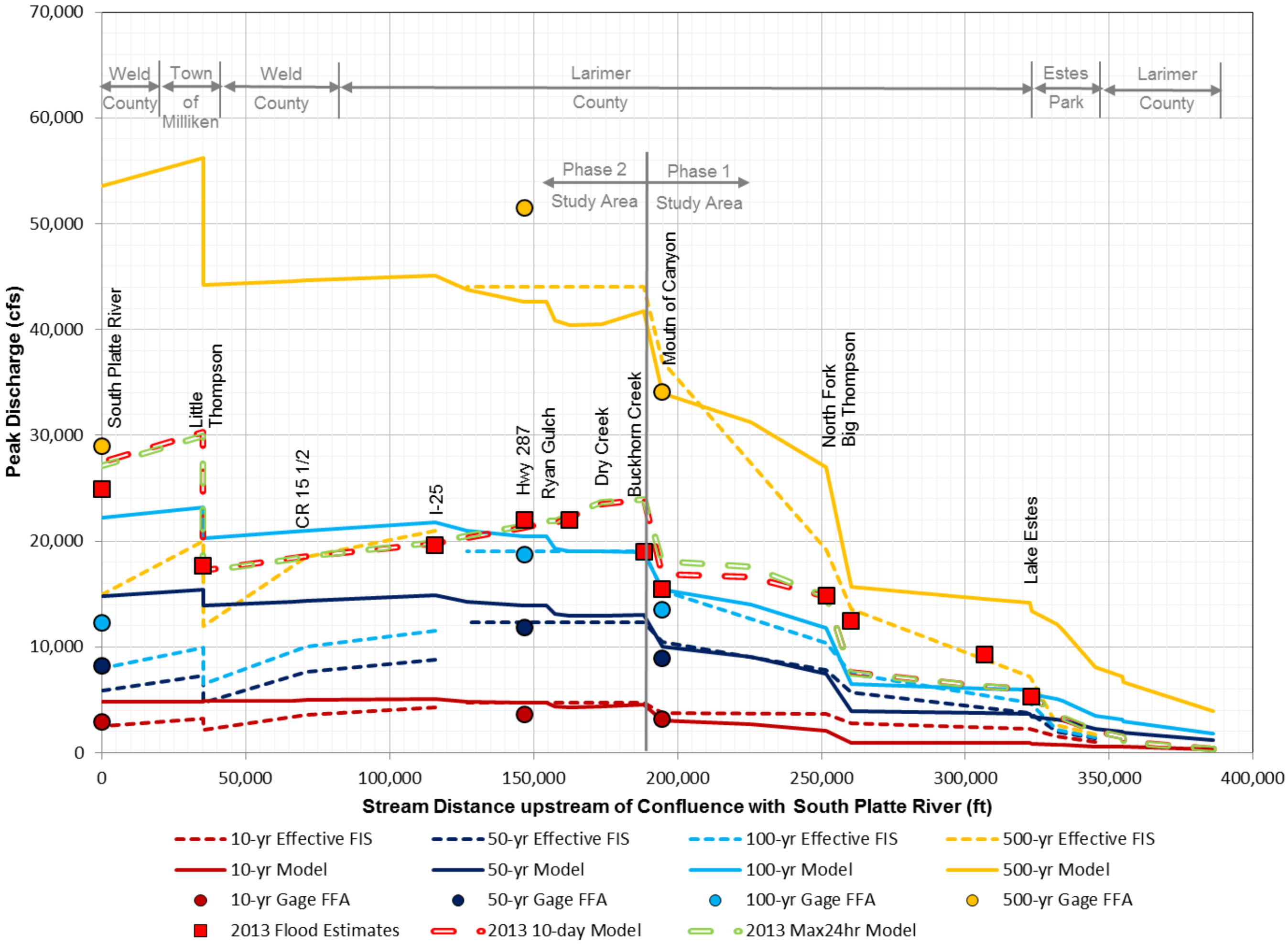
Based on the predictive model discharges for the return periods analyzed, as shown in Table ES-3 below, the peak discharge observed along the Big Thompson River during the September 2013 flood event was approximately a 1 percent annual chance peak discharge (100-year storm) upstream of the confluence with the Little Thompson River. The Little Thompson River also experienced a 1 percent annual chance peak discharge based on the report prepared by CH2M Hill. The combined peak discharge downstream of the confluence was between a 1 percent annual chance peak discharge and a 0.2 percent annual chance peak discharge, or between a 100-year and 500-year storm. It makes sense that the flood downstream of the confluence was slightly higher than a 100-year event since it is rare for both rivers to experience a 100-year storm simultaneously.

Table ES-3. Estimate of September 2013 Peak Discharge Recurrence Interval

Location	Drainage Area (mi^2)	Estimated Discharge (cfs)	Annual Chance Peak Discharge (cfs)					Estimated Recurrence Interval (yr)
			10%	4%	2%	1%	0.2%	
Big Thompson at Buckhorn Creek	461	19,000	4,530	8,580	13,000	18,900	41,800	~ 100
Big Thompson at Wilson Road	499	22,000	4,320	8,370	12,900	19,000	40,400	~ 100
Big Thompson at Highway 287	531	22,000	4,700	8,980	13,900	20,400	42,600	~ 100
Big Thompson at Interstate 25	577	19,600	5,090	9,530	14,900	21,800	45,100	~ 100
BT upstream of Little Thompson Confluence	620	17,700	4,900	9,050	13,900	20,300	44,200	~ 100
BT at Confluence with South Platte River	829	24,900	4,770	8,970	14,800	22,200	53,600	~ 100
Little Thompson above Confluence with BT	196	18,000	4,480	7,160	10,500	15,400	31,400	~ 100
Dry Creek (South) at Golf Course	32	2,450	1,390	2,400	3,460	4,710	8,540	~ 25

Figure ES-2 provides a summary of the hydrologic evaluation in the form of peak discharge profiles for the Big Thompson River 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 the Big Thompson River



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 Big Thompson watershed from the confluence with Buckhorn Creek near the mouth of the canyon to the confluence with the South Platte River.

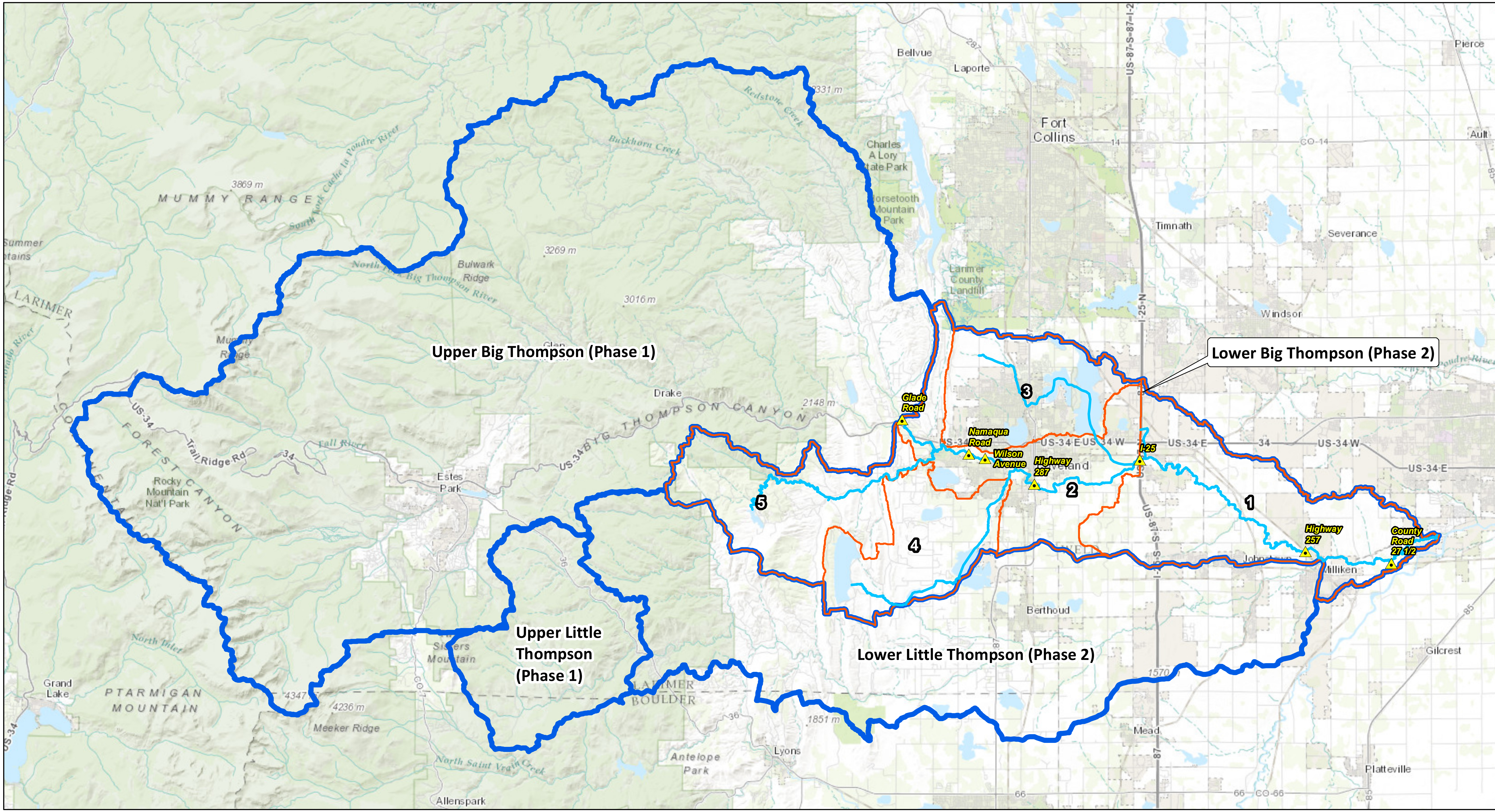
1.2 Project Area Description

The Big Thompson River originates in the Rocky Mountains, and the basin extends west to the Continental Divide with an elevation of 14,250 feet on Long's Peak. The Big Thompson River flows in an easterly direction through the southern parts of Larimer and Weld Counties passing by or through Estes Park, Drake, Loveland, Johnstown, and Milliken before reaching the South Platte River about 5 miles southwest of Greeley. Figure 1 provides an overview map of the Big Thompson watershed and shows the boundary between the Phase 1 and Phase 2 study areas. The Big Thompson River is approximately 78 miles long and the watershed encompasses a total drainage area of approximately 829 square miles. Phase 2 of the Big Thompson watershed study extends from the confluence with Buckhorn Creek (approximately 5 miles west of Loveland) to the confluence with the South Platte River, a length of approximately 36 miles with slopes ranging between 0.1 percent and 0.5 percent. Of the total 829 square mile watershed, the Phase 2 Big Thompson study area only accounts for 172 square miles. The remainder of the watershed is accounted for in the Phase 1 Big Thompson study area (461 square miles) and the Little Thompson watershed studied by CH2M Hill (196 square miles).

There are numerous small unnamed tributaries to the Big Thompson River in the Phase 2 study reach. The larger named tributaries include Dry Creek (North), Dry Creek (South), Ryan Gulch, and the Little Thompson River. A brief description of these tributaries is provided below.

Dry Creek (North) originates northwest of Loveland near County Road 19 and West 57th Street. Dry Creek (North), along with several other small unnamed tributaries in this area, drains east under Highway 287 to Horseshoe Lake which is hydraulically connected to Boyd Lake. Based on input from local personnel, these two lakes in combination with Lake Loveland, Barnes Ditch and the Greeley & Loveland Canal are all operated through coordinated actions by the Greeley & Loveland Irrigation Company. Typically there is no discharge directly to the Big Thompson River during storm events from this drainage area and irrigation system. In 2005, Boyd Lake was improved to provide regional flood control by making modifications to the spillway. Boyd Lake now provides 880 acre-feet of retention for the 100-year, 2-hour storm with zero release of stormwater onto downstream City of Loveland properties. Appendix D.1 provides the stage-storage-discharge relationship for Boyd Lake. Instead, the stormwater runoff is slowly released through a series of weirs into the Greeley & Loveland Canal and carried east under I-25. However, for storm events that exceed the available flood storage capacity in Boyd Lake, overflows through the spillway will cross over the Greeley & Loveland Canal and flow southeast toward the Big Thompson River upstream of I-25.

Dry Creek (South) originates southwest of Loveland in the foothills near Pinewood Reservoir and drains to the Big Thompson River at the Marianna Butte Golf Course. The Dry Creek (South) watershed encompasses approximately 32 square miles. Approximately 25 square miles of this watershed is located in the foothills with the remaining area located in the high plains. Dry Creek (South) has a channel length of approximately 13 miles with slopes ranging from 0.7 percent to 3.3 percent. Pinewood Reservoir and Flatiron Reservoir are both located in this watershed and were included in the hydrologic analysis. These reservoirs are not designed for flood control but do provide flood attenuation through the emergency spillways. Appendix D.1 provides the stage-storage-discharge relationship for both reservoirs. Both of these reservoirs are operated as part of the Colorado-Big Thompson Project.



Lower Big Thompson 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
Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri

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Legend

- Investigation Sites
- Tributaries
- Lower Big Thompson Major Basins
- BigThompson Watershed

Major Basins

1. Big Thompson River (From I-25 to Confluence with South Platte River)
2. Big Thompson River (From Confluence with Buckhorn Creek to I-25)
3. Dry Creek (North) (Headwaters to Horseshoe Lake and Boyd Lake)
4. Ryan Gulch (Headwaters to Confluence with Big Thompson River)
5. Dry Creek (South) (Pinewood Reservoir to Confluence with Big Thompson River)

Ryan Gulch originates southwest of Loveland, and drains to the Big Thompson River between Taft Avenue and Railroad Avenue. Ryan Gulch has a tributary drainage area of 26 square miles (including Carter Lake). Carter Lake and its relatively small tributary drainage area (3.7 square miles) do not contribute runoff to Ryan Gulch since the lake has no emergency spillway and is capable of retaining the probable maximum precipitation (PMP) event. The St. Vrain Supply Canal serves as the outlet for Carter Lake and is capable of delivering water to the Little Thompson, St. Vrain, and Boulder watersheds. Downstream of Carter Lake, Ryan Gulch has a channel length of approximately 11 miles with slopes ranging from 0.3 percent to 1.8 percent. Ryan Gulch drains through several irrigation reservoirs including Rainbow Lake, Lonetree Reservoir, Upper Ryan Gulch Lake, and Ryan Gulch Reservoir. None of these reservoirs provide flood control or have improved spillways so they have not been included in the hydrologic analysis.

The Little Thompson River encompasses approximately 196 square miles and joins the Big Thompson River near Milliken, approximately 6.5 miles upstream of the confluence with the South Platte River. The Little Thompson River has been studied separately by CH2M Hill and is described in more detail in a separate CDOT report dated June 2015. A brief memorandum documenting the Little Thompson 10-day model calibration for the 2013 Flood is provided in Appendix E. It should be noted that the memorandum in Appendix E is focused on the 10-day Flood whereas the actual Little Thompson Hydrologic Evaluation Report is focused on a 24-hour period and the predictive storms.

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. The 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/. Street 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 Larimer 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 basin boundaries and routing reaches within the watershed.

1.5 Flood History

Unlike the September 2013 flood, historical floods on the Big Thompson River have typically been caused by intense rainfall from localized thunderstorms. These types of floods are typically characterized by high peak discharges of relatively short duration. Historical flooding has also occurred as a result of rapid spring snowmelt which typically has a longer duration. A brief summary of the Big Thompson River flood history obtained from the 2013 Larimer County Flood Insurance Study (FIS) and the Preliminary 2013 Weld County FIS is provided here. More detailed information may be obtained by referring to the appropriate FIS report.

Approximately 13 floods have occurred in Loveland on the Big Thompson River since 1864, not including the 2013 flood. These floods occurred in 1864, 1894, 1906, 1919 (8,000 cfs), 1921, 1923 (7,000 cfs), 1938, 1941, 1942, 1945 (7,600 cfs), 1949 (7,750 cfs), 1951, and 1976. All but the 1919 flood did damage to crops, homes and businesses in the Loveland area. Several of the more notable floods are described below.

The largest recorded flood on the Big Thompson River occurred from July 31st to Aug 1st, 1976. This flood was one of the worst natural disasters in the history of the State of Colorado. Intense precipitation over an approximate 60-square mile area between Lake Estes and Drake, with rainfall depths up to 12 inches, generated a flood discharge of approximately 31,200 cfs at the mouth of the canyon. This flood is known to have taken 139 lives. Property damage was estimated at $16.5 million, while hundreds of people were left homeless. Over 200 residential structures were damaged or destroyed by the flood, while nearly 1,200 land parcels were adversely affected.

On August 2nd and 3rd, 1951, intense rains over much of the Big Thompson River basin caused a dam to break on Buckhorn Creek on August 3rd. This caused severe flooding from the mouth of Buckhorn Creek to the mouth of the Big Thompson River, especially through the Loveland area. Approximately 1 mile of US Highway 34 was destroyed just west of Loveland. Irrigation works were destroyed, crop loss was heavy, and much sediment and erosion damage occurred. The lives of four people were lost and many were left homeless. Total damages from the flood were estimated at $602,000. The estimated discharge from this flood was 22,000 cfs at Loveland, larger than the 1-percent annual chance flood discharge of 19,000 cfs.

On June 4th through 7th, 1949, heavy rains in the headwaters area of the Big Thompson River basin caused a flood with a magnitude of 7,750 cfs. Although considerably lower than the effective 100-year flood discharge of 19,000 cfs, lowland areas just west of Loveland were damaged.

On June 9, 1921, the Colorado and Southern Railroad Bridge was destroyed due to heavy rains on June 2nd through 7th, 1921.

2.0 HYDROLOGIC ANALYSIS

2.1 Previous Studies

The effective Larimer County FIS was published by the Federal Emergency Management Agency (FEMA) on February 6, 2013. 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 reports are shown in Table 1.

Table 1. Select Peak Discharge Values from 2013 FIS Reports

Flooding Source and Location	Drainage Area (sq. mi.)	Peak Discharge (cfs)			
		10-yr	50-yr	100-yr	500-yr
Big Thompson River					
Confluence with South Platte River	819	2,500	5,900	8,000	15,000
Downstream from Little Thompson River	813	3,200	7,300	9,900	20,000
Upstream from Little Thompson River	613	2,200	4,700	6,500	12,000
At Larimer-Weld County Line	595	3,600	7,600	10,000	18,500
At Interstate 25	515	4,300	8,800	11,500	21,000
At County Road 9E	515	4,700	12,300	19,000	44,000
At Railroad Avenue	515	4,700	12,300	19,000	44,000
At Mouth of Canyon (Drake Gage)	314	3,800	10,500	15,300	37,000
Little Thompson River					
at Milliken	200	1,630	3,600	4,800	8,400
Dry Creek – BTR (South)					
At Confluence with Big Thompson River	33	3,020	7,470	10,100	17,100

Previous studies pertaining to the Upper Big Thompson watershed are discussed in detail in the Phase 1 Hydrologic Evaluation Report dated August 2014. This Phase 2 Report is dependent on the Phase 1 hydrology which serves as the upstream boundary condition for the Phase 2 hydrology. The remainder of this section focuses on previous studies pertaining to the Phase 2 study area.

In 1971, the USACE presented flood flow frequencies for the portion of the Big Thompson River near Loveland based on statistical analysis of USGS gage data. Those flood frequencies were verified and used for the hydraulic study by Resource Consultants, Inc., which became effective in 1981. In 2005, Ayres Associates further verified the flood flow frequencies by augmenting the stream flow data with entries from the intervening period of record. An updated flood frequency relationship was developed in accordance with criteria outlined in Bulletin 17B using a systematic record of 80 years. Comparison showed that the 1971 flood discharges were higher than those from the updated flood frequency but typically plotted within the 90% confidence interval. Therefore, the 1971 flood discharges are still the effective discharges through Loveland.

Discharge magnitudes for floods on the Big Thompson River downstream in Weld County were based upon a May 1974 USACE analysis of stream gaging data at the USGS stream gages located near Drake (Mouth of Canyon) and La Salle. Because there were no streamflow gages located between La Salle and the Larimer-Weld County Line in 1974, discharge-probability relationships were developed for the intervening drainage area using unit hydrographs, flood routing techniques and rainfall probabilities from National Weather Service TP-40. The hydrology for the Big Thompson River near the Town of Johnstown was most recently studied by Anderson Consulting Engineers, Inc. in March 2005. They completed a thorough review of discharges from the 1971 hydrologic study and the 1974 Floodplain Information Report, both completed by the USACE. However, no changes were made to the original discharge estimates from these studies.

Unfortunately, the boundary between the two USACE studies (1971 upstream and 1974 downstream) resulted in a significant drop in peak discharges between County Road 9E and Interstate 25. Although there are several gravel pits adjacent to the channel in this location, there is no physical justification for the large drop in peak discharges. The drop is most likely a direct result of the different methods used in each of the two studies.

The hydrology for Dry Creek (South) was prepared by Anderson Consulting Engineers, Inc. in December 2002. The hydrology was developed using an earlier version of HEC-HMS (most likely Version 2.2). The analysis was conducted using a 3-hour storm duration and an area reduction factor of 7.7 percent based on the size of the watershed and the time of concentration. Both Pinewood Reservoir and Flatiron Reservoir were included in the model and were assumed to be full to the crest of the emergency spillway. Stage-storage-discharge rating curves were based on information developed by the United States Bureau of Reclamation (USBR). Infiltration losses were modeled using the Curve Number method and the basin routing used the Kinematic Wave transform. Channel routing was calculated using the Muskingum-Cunge method.

2.2 September 2013 Peak Discharge Estimates

CDOT and CWCB contracted with URS to obtain peak discharge estimates within the Phase 2 portion of the Big Thompson 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 seven peak discharge estimates for the Big Thompson River in the Phase 2 study area. However, due to significant bridge overtopping, one of the estimates was discarded, leaving six reliable estimates at the following locations:

Namaqua Road, Wilson Avenue, US Highway 287, Interstate 25, US Highway 257, and County Road 27 ½. These locations are shown on Figure 1 as Investigation Sites. These estimates were supplemented with recorded data from stream flow gages and reservoir operations. Stream gages on the Big Thompson River at Glade Road, St. Louis Avenue, and at La Salle recorded the rising limb of the storm before they were washed out. This information was useful in calibrating the timing of the peak discharges. An early flood warning gage on Dry Creek at Mariana Butte Golf Course recorded the peak discharge hydrograph throughout the storm. Water surface elevations were also recorded in Pinewood Reservoir, Flatiron Reservoir, and Boyd Lake during the 2013 flood and none of these reservoirs had flood discharges through their respective spillways. The peak discharge estimates are presented on Table 4 later in the report.

2.3 Updated Flood Frequency Analyses

Flood frequency analyses (FFA) were performed to supplement the hydrologic evaluation of the Big Thompson River. 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 two stream flow gages. Figure D.1 in Appendix D shows the location of these two gages.

Big Thompson River at Loveland near St. Louis Ave.

- USGS Gage 06741510 (1948 – 1955)
- CDWR Gage BIGLOVCO (1979 – 2012)

Big Thompson River at La Salle near Mouth

- USGS Gage 06744000 (1915 & 1927 – 1981)
- CDWR Gage BIGLASCO (1991 – 2012)

The Big Thompson River gage record at Loveland has 42 annual peak flows over a 66 year period of record. The earliest is from 1948 and the latest is from 2013. Gaps in the record exist between 1955 and 1979 including the 1976 Flood. The 1976 and 2013 floods were added to the data record with peak flows of 6,000 cfs and 22,000 cfs, respectively. The 1976 peak flow was estimated independently by Ayres Associates based on aerial photos from August 1, 1976 and the floodplain model developed by Ayres in 2005. 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 7,750 cfs in 1949. The 1976 estimate is the fourth largest peak flow in the record.

The Big Thompson River gage at La Salle has 77 annual peak flows over a 99 year period of record. The earliest is from 1915 and the latest is from 2012. Gaps in the record exist between 1915 and 1927 and between 1981 and 1991. The 2013 flood was added to the data record with a peak flow of 24,900 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 followed by a peak flow of 6,710 cfs in 1995. The 1976 peak flow estimate of 2,440 cfs is the eleventh largest peak flow in the record.

As noted above, the period of record at the La Salle gage is 99 years as opposed to 66 years at the Loveland gage. At both gages the 2013 flood is by far the largest recorded peak discharge. Examination of the gage records also shows that all of the highest five peak flows at the Loveland gage (in 2013, 1949, 1980, 1976, 1999) resulted in peaks ranked in the highest eleven at the La Salle gage. It is reasonable, therefore, to conclude that a major flood in Loveland would have registered a significant peak at La Salle, especially if it was greater than the 2013 flood. The main period for which flows were recorded at La Salle but not at Loveland is between 1927 and 1948. During that time period, only one peak was recorded at La Salle exceeding 2,000 cfs (a 3,000 cfs peak in 1938). The La Salle records indicate that no flood occurred during the ungaged years at Loveland that was anywhere near the magnitude of the 2013 flood. For that reason this analysis assumes that the 2013 flood was the highest peak discharge at the location of the Loveland gage in at least the last 99 years. Therefore, the flood frequency analysis at both gages reflects the 99 year period.

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 Big Thompson 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 the Big Thompson River (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 Big Thompson River

Exceedence Recurrence Interval (years)	Big Thompson at Loveland Gage (cfs)	Big Thompson at La Salle Gage (cfs)
2	760	660
5	2,000	1,683
10	3,620	2.897
50	11,800	8,260
100	18,700	12,310
200	29,200	17,980
500	51,500	28,963

Based on these FFA results, the 2013 flood was approximately a 100-year event at the Loveland gage. The FFA at the La Salle gage indicates that the 2013 flood was closer to a 500-year event, which is not unreasonable considering that the gage is located downstream of the Little Thompson River confluence and both rivers experienced considerable flooding at almost the same time. It should be noted that reliable flood-frequency relations are difficult to estimate 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 Big Thompson 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. A calibrated HEC-HMS model was developed in Phase 1 of this hydrologic analysis for the Big Thompson River upstream of Glade Road (Big Thompson River, North Fork Big Thompson River, and Buckhorn Creek). Similarly, a HEC-HMS model for the entire Little Thompson watershed was developed by CH2M Hill. The model output, in the form of discharge hydrographs, from these two tributary models was then used as input to a separate model for the lower Big Thompson watershed (Confluence with Buckhorn Creek 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 Cache la Poudre watershed to the north, the Phase 1 model to the west (extends to the Continental Divide), and the Little Thompson 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 Phase 2 watershed was divided into 25 basins ranging from 1.14 square miles to 16 square miles. Basins were manually subdivided where necessary in order to compare peak discharge estimates at investigation sites, stream gages and reservoirs with results from the hydrologic model. The eleven peak discharge estimation locations used for comparison include:

1. Big Thompson River at Namaqua Road (URS)
2. Big Thompson River at Wilson Avenue (URS)
3. Big Thompson River at US Highway 287 (URS)
4. Big Thompson River at Interstate 25 (URS/CDOT)
5. Big Thompson River at US Highway 257 (URS)
6. Big Thompson River at County Road 27 ½ (URS)
7. Dry Creek at Pinewood Reservoir (CDWR)

8. Dry Creek at Flatiron Reservoir (CDWR)
9. Dry Creek at Marianna Butte Golf Course Gage (City of Loveland)
10. Ryan Gulch at Carter Lake (CDWR)
11. Boyd Lake Emergency Spillway (Greeley & Loveland Irrigation Company)

2.4.3 Basin Characterization

The basin characteristics of the Lower Big Thompson watershed (Phase 2) consist mainly of agricultural and pasture lands with developed urban areas around Loveland, Johnstown, and Milliken. The watershed topography generally slopes west to east with mild slopes. The individual basin slopes range from approximately 0.3 percent in the eastern plains to as steep as 8.5 percent in the headwaters of Dry Creek. Four major tributary areas join the Big Thompson River in the Phase 2 study area; Dry Creek (South), Ryan Gulch, Dry Creek (North, tributary to Boyd Lake), and the Little Thompson River. The drainage area of the Phase 2 study area is approximately 172 square miles, whereas the total Big Thompson watershed encompasses approximately 829 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.

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 7.24 inches, ranging from as low as 4.91 inches up to 11.90 inches for the individual basins. However, almost half of this rainfall fell within a 24-hour period starting around 7 P.M. (MST) on Wednesday, September 11, 2013. The average 24-hour rainfall depth for all of the basins was 3.20 inches, ranging from 1.73 inches up to 6.71 inches for the individual basins. The average 24-hour rainfall depth of 3.20 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.

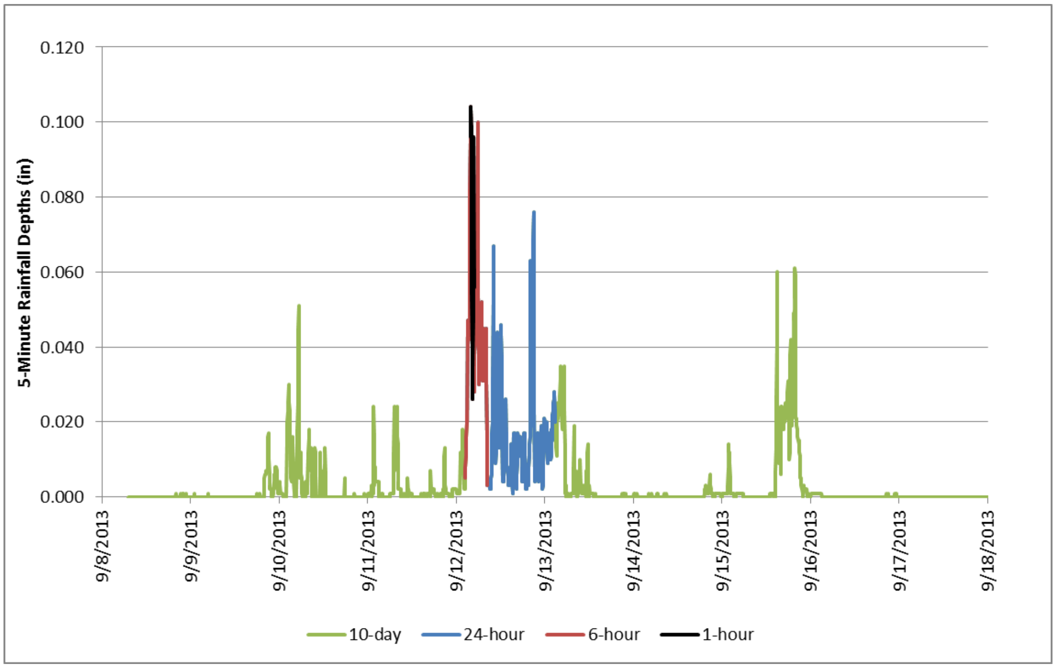
Table 3. Representative Rainfall Depths from September 2013 Flood and Associated NOAA Atlas 14 Recurrence Interval

Location	Dry Creek (South) Headwaters (LBT13C)		Ryan Gulch (LBT08)		Dry Creek (North) Headwaters (LBT16B)		Big Thompson at I-25 (LBT05B)		Big Thompson at Mouth (LBT01)	
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	11.90	1000	5.66	25	5.87	25	4.91	10 to 25	5.68	25 to 50
24-hour	6.71	200 to 500	2.09	2 to 5	2.48	5	2.02	2 to 5	2.31	5
6-hour	3.39	25 to 50	1.20	1 to 2	1.63	2 to 5	1.15	1	1.53	2 to 5
1-hour	0.88	2 to 5	0.33	< 1	0.58	< 1	0.35	< 1	0.62	< 1

Figure 2 shows a hyetograph for a basin in the headwaters area of Dry Creek (South). The incremental depths are based on a 5-minute time step. As shown in Table 3, Dry Creek (South) 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 roughly a 1000-year event, the maximum 24-hour rainfall total is between a 200-year and 500-year event, the maximum 6-hour rainfall total is between a 25-year and 50-year event, and the maximum 1-hour rainfall total is only between a 2-year and 5-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.

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.

Figure 2. September 2013 Rainfall Hyetograph for Dry Creek (South) Headwaters



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 Big Thompson model and the Phase 2 Little Thompson 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 the Big Thompson River 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 and 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 default values ranging from 0.05 to 0.15 were appropriate for the channels in the Phase 2 study area.

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 the Big Thompson River, possibly to storage reservoirs or adjacent watersheds. Even if this water eventually returned to the Big Thompson River it was most likely delayed relative to the peak observed in the Big Thompson River. One example of this form of attenuation is the flood waters which were diverted from the Big Thompson to Lake Loveland, Horseshoe Lake, and Boyd Lake via the Barnes Ditch and the Greeley & Loveland Canal.

- 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.

- Gravel pits located within the floodplain that have available storage capacity and potentially result in split flows by diverting water along historic channel alignments. The flood extents observed in these locations were very wide and the velocity of flow in the overbank areas was significantly slower than the velocities in 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 the Big Thompson River was dependent on two primary factors. The first was the inflow hydrograph from the Upper Big Thompson watershed (461 square miles) developed in Phase 1 (provided in Appendix D.3). The second factor was the major tributaries and local drainage basins within the Phase 2 study area. The Upper Big Thompson watershed experienced heavier rainfall during the 2013 flood and the discharge at the mouth of the Canyon tended to dominate the peak discharges downstream in the Big Thompson River in the Phase 2 study area. Peak discharges from the Phase 2 tributaries were generally smaller and peaked earlier than the discharge from the canyon, limiting the overlap.

Based on this knowledge, the Phase 2 tributaries were calibrated first based on available peak discharge estimates, gage records, and comparison of unit discharges with respect to rainfall depths/intensities. The model was calibrated at four reservoirs (Pinewood, Flatiron, Carter, and Boyd) located in various tributaries based on observed stage-discharge records and the knowledge that none of these reservoirs utilized the overflow spillways during the 2013 flood. Once the tributary drainage basins in Phase 2 were initially calibrated, attempts were made to calibrate the combined flows in the Big Thompson River. 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 Big Thompson 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
Big Thompson at Wilson Avenue	22,000	22,300	1%
Big Thompson at Hwy 287	22,000	21,300	- 3%
Big Thompson at I-25	19,600	19,800	1%
Big Thompson above Little Thompson Confluence	17,700	17,300	- 2%
Big Thompson at South Platte River	24,900	27,500	10%
Little Thompson River at Big Thompson River	18,000	16,200	- 10%
Dry Creek (South) at Big Thompson River	2,450	2,580	5%

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 Big Thompson 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**
LBT17	3.12 (2.52-3.80)	4.02 (3.20-5.24)	4.81 (3.72-6.32)	5.69 (4.24-7.67)	8.10 (5.56-11.6)
LBT14	2.96 (2.38-3.68)	3.82 (3.04-5.11)	4.58 (3.53-6.19)	5.44 (4.05-7.56)	7.83 (5.38-11.5)
LBT13D	2.93 (2.34-3.67)	3.79 (3.01-5.12)	4.57 (3.51-6.22)	5.44 (4.05-7.63)	7.89 (5.43-11.7)
LBT13C	2.99 (2.39-3.75)	3.87 (3.07-5.24)	4.67 (3.59-6.37)	5.56 (4.13-7.80)	8.06 (5.54-12.0)
LBT13B	3.08 (2.47-3.84)	3.98 (3.16-5.33)	4.78 (3.68-6.46)	5.68 (4.22-7.89)	8.17 (5.61-12.0)
LBT15	3.00 (2.41-3.73)	3.87 (3.08-5.16)	4.64 (3.58-6.25)	5.50 (4.10-7.61)	7.88 (5.42-11.6)
LBT13A	3.13 (2.52-3.87)	4.03 (3.21-5.35)	4.84 (3.73-6.47)	5.73 (4.27-7.87)	8.20 (5.64-11.9)
LBT12B	3.07 (2.47-3.79)	3.95 (3.15-5.23)	4.73 (3.65-6.31)	5.60 (4.18-7.67)	8.00 (5.49-11.6)
LBT12A	3.15 (2.54-3.85)	4.05 (3.23-5.30)	4.84 (3.74-6.40)	5.73 (4.27-7.76)	8.14 (5.59-11.7)
LBT07	3.01 (2.43-3.69)	3.87 (3.08-5.06)	4.62 (3.57-6.10)	5.46 (4.07-7.39)	7.74 (5.31-11.1)
LBT11	2.96 (2.39-3.67)	3.81 (3.04-5.07)	4.57 (3.53-6.13)	5.41 (4.03-7.45)	7.73 (5.31-11.3)
LBT10	2.94 (2.37-3.64)	3.78 (3.02-5.02)	4.53 (3.50-6.05)	5.36 (4.00-7.35)	7.64 (5.25-11.1)
LBT09	2.95 (2.39-3.65)	3.79 (3.02-5.01)	4.54 (3.51-6.04)	5.36 (4.00-7.32)	7.61 (5.23-11.0)
LBT08	2.98 (2.41-3.67)	3.83 (3.05-5.04)	4.58 (3.54-6.07)	5.40 (4.03-7.35)	7.66 (5.26-11.0)
LBT06B	2.94 (2.37-3.61)	3.77 (2.99-4.93)	4.49 (3.46-5.92)	5.29 (3.94-7.16)	7.47 (5.13-10.7)
LBT06A	2.91 (2.34-3.58)	3.72 (2.95-4.87)	4.42 (3.40-5.85)	5.20 (3.86-7.05)	7.31 (5.01-10.5)
LBT05B	2.87 (2.31-3.54)	3.66 (2.90-4.81)	4.35 (3.34-5.76)	5.11 (3.80-6.94)	7.18 (4.92-10.3)
LBT05A	2.85 (2.28-3.53)	3.62 (2.85-4.76)	4.29 (3.29-5.69)	5.03 (3.72-6.84)	7.02 (4.81-10.1)
LBT16B	3.02 (2.44-3.67)	3.88 (3.09-5.04)	4.63 (3.58-6.07)	5.47 (4.08-7.36)	7.77 (5.34-11.1)
LBT16A	2.92 (2.35-3.55)	3.73 (2.97-4.85)	4.45 (3.43-5.84)	5.24 (3.90-7.05)	7.40 (5.08-10.5)
LBT05C	2.87 (2.30-3.53)	3.66 (2.90-4.79)	4.36 (3.35-5.75)	5.13 (3.81-6.93)	7.22 (4.95-10.3)
LBT04	2.85 (2.29-3.53)	3.63 (2.86-4.77)	4.31 (3.30-5.71)	5.05 (3.74-6.86)	7.06 (4.84-10.1)
LBT03	2.84 (2.28-3.52)	3.61 (2.85-4.74)	4.28 (3.28-5.66)	5.01 (3.71-6.80)	6.97 (4.78-10.0)
LBT02	2.82 (2.27-3.48)	3.57 (2.82-4.66)	4.21 (3.24-5.56)	4.92 (3.65-6.66)	6.82 (4.68-9.74)
LBT01	2.77 (2.27-3.38)	3.50 (2.80-4.52)	4.13 (3.21-5.38)	4.81 (3.60-6.44)	6.64 (4.56-9.41)

Due to the size of the Big Thompson watershed (approximately 829 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.

Figure 3. Depth-Area Reduction Factor (DARF) Curves

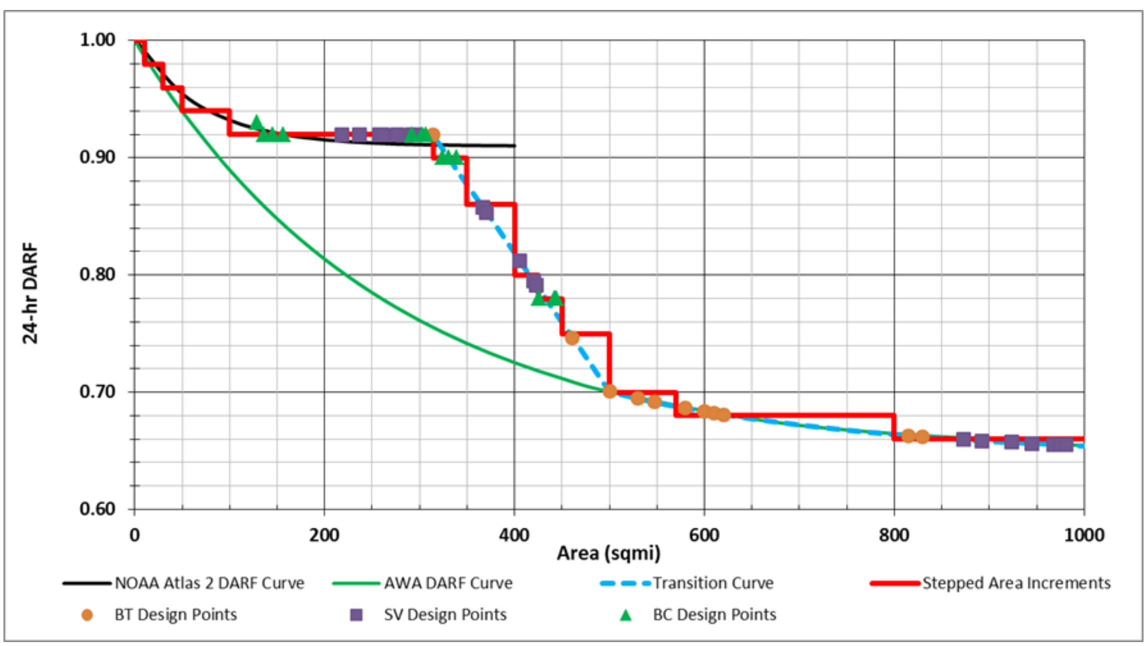


Table 6. Stepped Area Increments for DARF Application

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

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 the Little Thompson River, a 34% reduction in rainfall is applied to all basins in the model to determine the effective peak discharge at 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 the Big Thompson River. This process is described in more detail in Appendix D.2. Appendix D.4 shows the appropriate DARF value for all nodes in the model.

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 Big Thompson watershed. These critical areas included:

- Only the Phase 2 Study Area without inflows from the Upper Big Thompson Phase 1 Study Area.
- Phase 2 Study Area with inflows from Buckhorn Creek but not from the Upper Big Thompson.
- Phase 2 Study Area with inflows from Buckhorn Creek and the Upper Big Thompson Study Area up to Lake Estes (no discharge from Lake Estes).
- Phase 2 Study Area with inflows from the Upper Big Thompson Study Area up to Lake Estes (no discharge from Lake Estes or Buckhorn 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 the Big Thompson River 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. Big Thompson 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 initial 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 (7.24 inches) and the average 24-hour maximum rainfall (3.20 inches) for the Big Thompson watershed was 4.04 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. The 10-day calibrated CN values were increased from an average of 59 up to 78 for the maximum 24-hour rainfall period in order to produce results similar to the peak discharge estimates from the 2013 flood. 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 Big Thompson River (from headwaters to the South Platte River), Dry Creek (South), Ryan Gulch, and Dry Creek (North). 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 recurrence intervals.

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.

Table 7. Hydrologic Model Peak Discharge Results

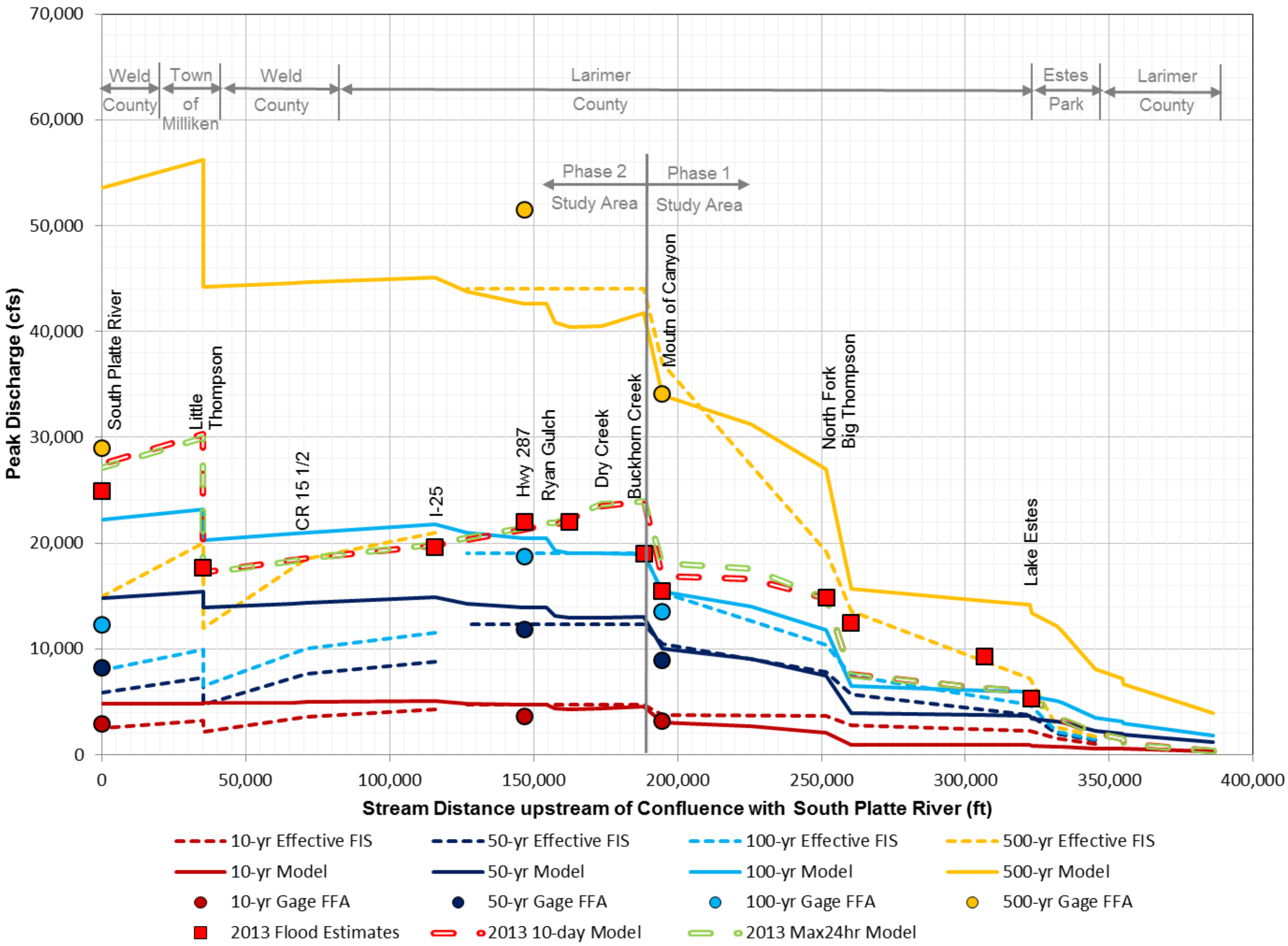
Description	Area (sq. mi.)	Estimated Peak (cfs)	2013 Flood Calibrated Model 10-day (cfs)	2013 Flood Max 24hr Period CN Calibrated (cfs)	NOAA Design Storms (CN Calib & DARF) 10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)
BT at confluence with Fern Creek in RMNP	33		330	390	350	740	1,180	1,790	3,890
BT at Confluence of Glacier Creek	65		1,020	960	560	1,180	1,920	2,960	6,650
BT at confluence with Wind River upstream of Estes Park	75		1,510	1,460	560	1,200	2,000	3,120	7,220
BT at confluence with Beaver Brook	84		2,030	2,030	610	1,330	2,220	3,480	8,090
BT at confluence with Fall River	126		3,270	3,400	790	1,790	3,060	4,900	11,600
BT at confluence with Black Canyon Creek	136		3,640	3,770	790	1,830	3,150	5,070	12,100
BT inflow to Lake Estes	154		5,410	5,340	850	1,980	3,420	5,550	13,400
Lake Estes (Olympus Dam)	154	5,330	5,330	5,330	850	1,980	3,420	5,550	13,400
BT at confluence with Dry Gulch below Lake Estes	160		6,020	6,000	920	2,140	3,680	5,940	14,200
BT at Loveland Heightes (Jarrett Estimation Point #62)	164	9,300	6,270	6,250	940	2,180	3,750	6,060	14,500
BT at Mountain Shadows Lane above Drake	188	12,500	7,570	7,530	960	2,280	3,960	6,450	15,700
Confluence of BT and NFBT at Drake	276	14,800	14,700	14,700	2,120	4,540	7,490	11,800	27,000
BT at confluence with Cedar Creek	300		16,600	17,600	2,690	5,580	9,050	14,000	31,300
BT at Mouth of Canyon (Jarrett Estimation Point #66)	314	15,500	16,900	18,100	3,040	6,250	10,100	15,400	34,000
Upper Big Thompson Model Inflow Hydrographs	461	19,000	24,000	24,000	4,530	8,580	13,000	18,900	41,800
BT at Rossum Drive (Dry Creek Confl.)	499		23,500	23,700	4,330	8,380	12,900	19,000	40,500
BT at Wilson Avenue	499	22,000	22,300	22,400	4,320	8,370	12,900	19,000	40,400
BT at Taft Avenue	504		21,700	21,900	4,420	8,520	13,100	19,300	40,800
BT at Railroad Avenue	530		21,700	21,900	4,710	8,970	13,900	20,400	42,600
BT at Hwy 287 (Lincoln Ave.)	531	22,000	21,300	21,500	4,700	8,980	13,900	20,400	42,600
BT downstream of County Road 9E	547		20,300	20,500	4,770	9,170	14,300	21,000	43,800
BT at I-25	577	19,600	19,800	19,900	5,090	9,530	14,900	21,800	45,100
BT upstream of County Road 15 1/2	603		18,500	18,500	4,950	9,260	14,400	21,000	44,600
BT at County Road 15 1/2 (Road 52 & Railroad)	607		18,400	18,300	4,940	9,220	14,300	20,900	44,600
Big Thompson Upstream of Confluence with Little Thompson	620	17,700	17,300	17,200	4,900	9,050	13,900	20,300	44,200
Confluence of Big Thompson and Little Thompson	816		30,300	30,000	4,860	9,470	15,400	23,200	56,200
BT Confluence with South Platte River	829	24,900	27,500	27,200	4,770	8,970	14,800	22,200	53,600
Little Thompson Model Inflow Hydrographs	196	18,000	16,200	16,200	4,480	7,160	10,500	15,400	31,400
Pinewood Reservoir - No Discharge to Dry Creek in 2013 Flood	3	0	0	0	120	200	280	410	770
Confluence of Skinner Gulch and Mill Gulch	9		630	500	270	500	740	1,060	2,110
Cottonwood Creek at Confluence with Saddle Notch Gulch	12		950	760	330	620	950	1,350	2,750
Cottonwood Creek above Flatiron Reservoir	16		1,290	1,000	460	860	1,300	1,830	3,620
Dry Creek below Flatiron Reservoir	23		1,290	1,000	830	1,490	2,220	3,070	5,780
Dry Creek at Hogback Ridge	25		1,670	1,150	1,000	1,780	2,620	3,610	6,710
Dry Creek at CR 20	31		2,470	1,380	1,290	2,240	3,260	4,440	8,110
Dry Creek above Confluence with BT	32		2,580	1,440	1,390	2,400	3,460	4,710	8,540
Dry Creek through Golf Course	32	2,450	2,570	1,440	1,390	2,400	3,460	4,710	8,540
Carter Lake can contain PMP storm event per USBR	4	0	0	0	0	0	0	0	0
Ryan Gulch at Hertha Reservoir	7		300	190	270	470	670	920	1,640
Ryan Gulch upstream of Lonetree Reservoir	10		520	270	470	820	1,180	1,600	2,850
Ryan Gulch upstream of BT	26		1,220	510	1,330	2,250	3,170	4,240	7,380
Ryan Gulch at BT confluence	26		1,220	500	1,330	2,250	3,170	4,220	7,370
Dry Creek above Boyd Lake	14		930	820	1,440	2,020	2,540	3,120	4,720
Boyd Lake Labyrinth Spillway	27	0	0	0	1,710	2,800	3,720	4,730	7,570
Boyd Lake Overflow Path	27		0	0	1,710	2,800	3,720	4,730	7,570

Table 7 shows that calibrated model matched the peak discharge estimates within 20 percent at all observed locations in the Phase 2 Study Area. There are a few locations in the Upper Big Thompson River (Phase 1) that exceeded 20 percent differences and descriptions of those differences are provided in the Phase 1 Hydrologic Evaluation Report. There was no observed discharge from four of the modeled reservoirs (Pinewood, Flatiron, Carter, and Boyd) during the 2013 flood. However, for predictive modeling purposes the two reservoirs (Pinewood and Flatiron) on Dry Creek (South) were assumed to be full to the spillway at the beginning of the storm and had resulting peak discharges through the spillway as seen in Table 7. Boyd Lake was modeled assuming the reservoir was full to the principal spillway and that the dedicated 880 acres of flood storage between the principal spillway crest and the emergency spillway crest was utilized (depth of six inches). The peak discharge six inches above the principal spillway crest is only 250 cfs, above this depth the peak discharge increases dramatically. Carter Lake does not have an overflow spillway and is designed to contain the PMP event from the relatively small tributary drainage area around the reservoir. Therefore, Carter Lake was modeled to capture all local runoff with zero discharge to Ryan Gulch.

The peak discharge profile for the Big Thompson River is shown on Figure 4 below (larger version of Figure 4 is provided in Appendix D.6). 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, the calibrated 2013 flood model results for the 10-day rainfall period and the maximum 24-hour rainfall period are almost identical. It should be noted that the peak discharge in the Big Thompson River during the 2013 flood was driven primarily by the discharge from the upper watershed. As this peak translated downstream through Loveland the peak discharge started to attenuate since the peak discharges from the smaller Phase 2 tributaries had already moved downstream ahead of it.

Figure 4. Peak Discharge Profiles for the Big Thompson River



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 two plots are at locations on Dry Creek (South) and include stage observations in Pinewood and Flatiron Reservoirs. Both of these reservoirs are operated as part of the Colorado-Big Thompson Project and the plots show a see-saw pattern resulting from inflow and release operations. The HEC-HMS model was calibrated to the observed stages in the reservoir by ignoring the daily operations and instead focusing on the rising limb of the inflow hydrograph and the total inflow volume observed.

The next two plots in Appendix D.9 show model results compared against partial records of Early Flood Warning Gages at Marianna Butte Golf Course on Dry Creek (South) and on the Big Thompson River near Glade Road. The next three plots in Appendix D.9 show the modeled hydrographs at the locations of peak discharge estimates along the Big Thompson River (Namaqua Road, Wilson Avenue, and Highway 287). The timing of the Big Thompson River peak discharge in the model was compared against the partial record at the USGS Loveland Gage and the rising limb of the hydrograph lines up almost exactly.

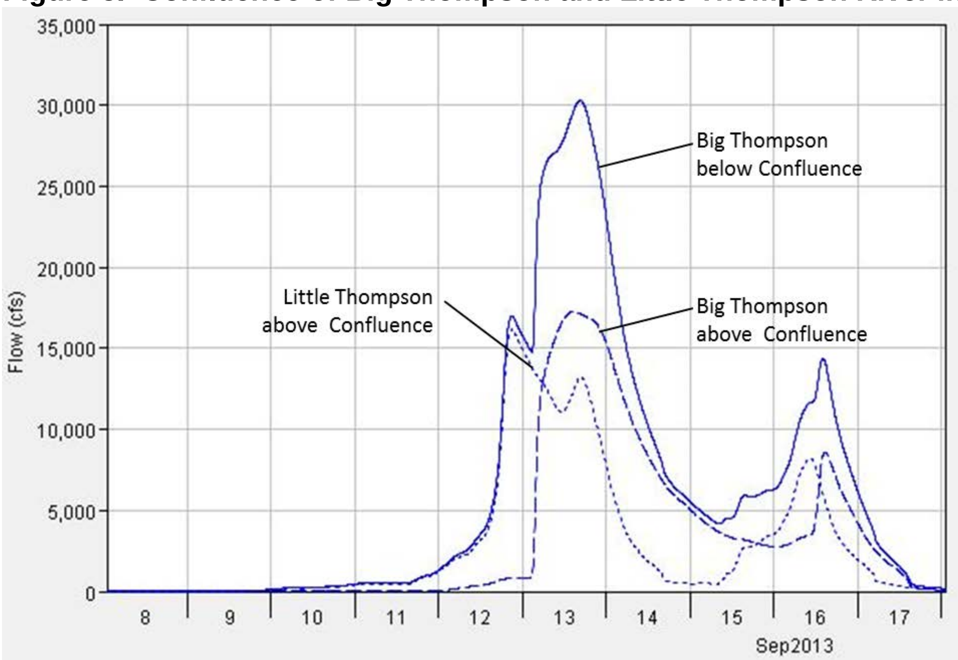
Gage records were obtained for Lake Loveland, Horseshoe Lake, and Boyd Lake from the Greeley & Loveland Irrigation Company. These lakes in combination Barnes Ditch and the Greeley & Loveland Canal are all operated through coordinated actions by the Greeley & Loveland Irrigation Company. During the 2013 flood the reservoirs were used to alleviate potential flooding through coordinated releases between the reservoirs and canals and no flood flows overtopped Boyd Lake emergency spillway. In order to simplify the hydrologic analysis of this portion of the watershed, a single storage node was incorporated into the 2013 flood model at the location of Boyd Lake using a composite stage-storage relationship to represent the combined storage observed in Horseshoe Lake and Boyd Lake during the flood. This composite reservoir was then used to calibrate the upstream drainage basins to match the observed inflow volume as shown in the Appendix D.9 plot. The HEC-HMS model shows two inflow surges representative of rainfall bursts whereas the observed stage increase in Boyd Lake was more gradual due to the coordinated reservoir operations which controlled the release into Boyd Lake from Horseshoe Lake.

Three of the last four plots in Appendix D.9 show the modeled hydrographs at the locations of peak discharge estimates along the Big Thompson River (Interstate 25, Highway 257, and County Road 27 ½). The remaining plot in Appendix D.9 compares the HEC-HMS model results against the partial gage record at La Salle downstream of the confluence with the Little Thompson River. The recorded discharge was limited at the gage but the recorded stage was more complete and the stage increases matched up well with the rising limb of the modeled hydrograph indicating that the timing of the peak discharges at the confluence of the Big Thompson and Little Thompson River were well aligned. Peaks on these two rivers overlapped considerably in the 2013 flood as indicated by the downstream peak discharge estimate and the model calibration. This is not surprising considering the relatively similar travel times and the overall storm pattern across the watershed. Figure 5 shows the overlapping hydrographs at the confluence with the Little Thompson River for the 2013 flood 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 the Big Thompson River are also shown on Figure 4. The predictive model peak discharges for the various return periods were compared to the results from the updated flood frequency analysis for the Big Thompson River as well as to current regulatory discharges. The model results compared well with the existing regulatory flows and the updated FFA upstream of Interstate 25. However, as noted in Section 2.2, the current regulatory peak discharges show a sharp drop just upstream of I-25 based on the different USACE study methodologies. In contrast, the predictive model results show a relatively constant peak discharge from I-25 downstream to the confluence with the Little Thompson. A slight attenuation of peak discharges is observed in this reach but the model also accounts for inflow from smaller tributaries which tend to balance it out.

Figure 5. Confluence of Big Thompson and Little Thompson River in 2013 Flood

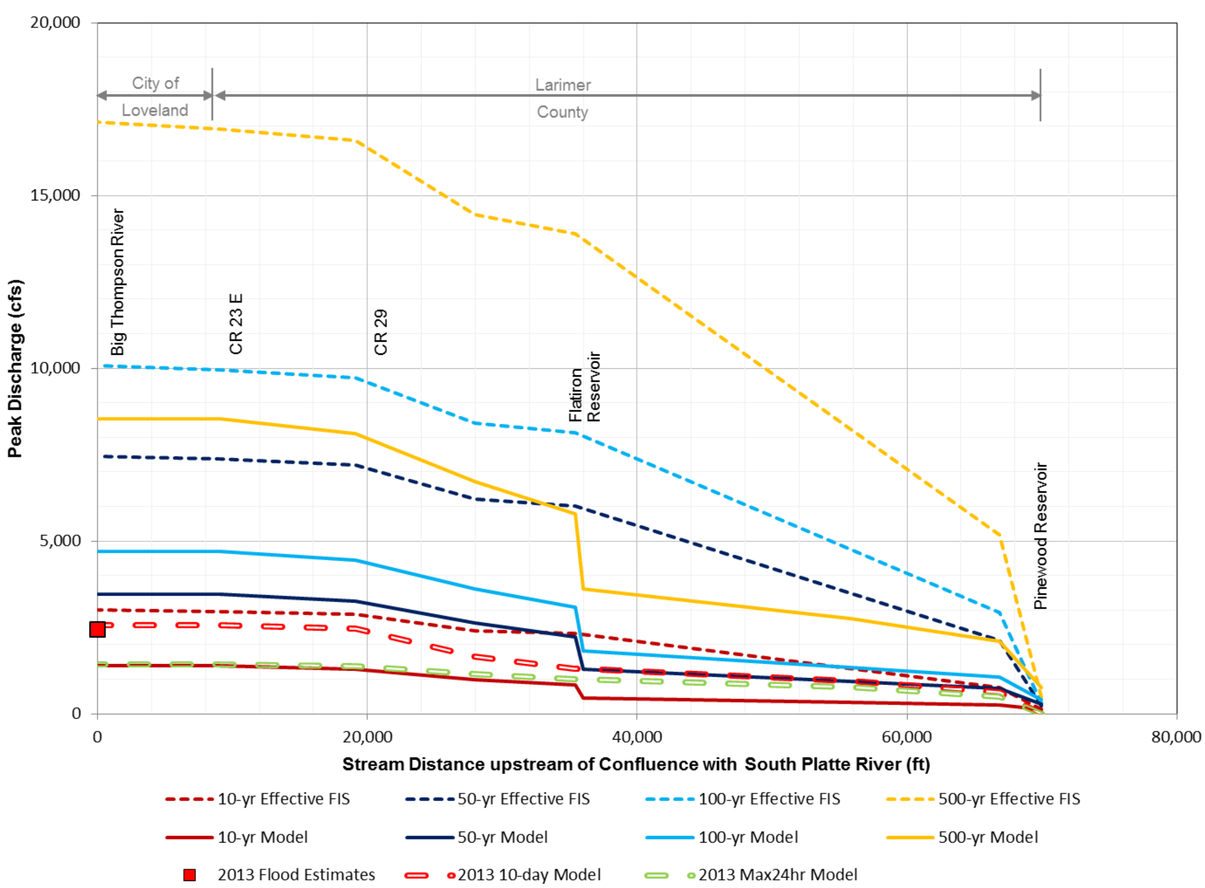


In the Phase 1 Hydrologic Evaluation Report, predictive peak discharges at the downstream end of the model (Big Thompson River confluence with Buckhorn Creek) were published as being provisional due to the fact that the tributary drainage area exceeded the limits of the NOAA Atlas 2 DARF curves. As part of the Phase 2 Study, the new AWA DARF curves were used to correct these provisional peak discharges resulting in a 100-year peak discharge of approximately 19,000 cfs on the Big Thompson River at the confluence with Buckhorn Creek. The predictive model discharges for all recurrence intervals were consistent with the current regulatory peak discharges at this location.

On the Big Thompson River at the Loveland Gage (downstream of Highway 287 at St. Louis Avenue), the predictive model results were reasonably close to the FFA results for the 50-year (+18%), 100-year (+9%) and 500-year (-17%) storms. The model results were higher than the FFA results for the 10-year (+30%) storm, which can possibly be attributed to the larger impact of irrigation systems for the smaller storms. At the La Salle Gage near the mouth of the Big Thompson River the predictive model results were considerably higher than the updated FFA results. Although the LaSalle gage has 99 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 6,710 cfs for an 829 square mile watershed (8 cfs/mi^2). The 2013 peak discharge estimate of 24,900 cfs shows that these irrigation practices though are not sufficient to control large floods, hence the assumption that their effects are ignored in the predictive model. It should be noted that even though the FFA resulted in much lower peak discharges than the predictive model, they were still considerably higher than the current regulatory values.

On Dry Creek (South), the current regulatory hydrology was prepared by Anderson Consulting Engineers, Inc. in 2002 and was also developed using HEC-HMS. The 2002 study subdivided the 32 square mile watershed into 21 basins whereas this study only modeled 8 basins. The regulatory hydrology was developed using a 3-hour storm duration with a DARF of 92.3 percent (7.7 percent reduction) as opposed to the 24-hour storm duration with a DARF of 96 percent used in this analysis. In both models Pinewood Reservoir and Flatiron Reservoir were assumed to be full to the crest of the emergency spillway prior to the start of the storm. The same stage-storage-discharge rating curves were used in each model. Infiltration losses were modeled using the Curve Number method in both models, the 2002 study used an average CN of 71 and this study used an average CN value of 75. The basin transform method used in the 2002 study was Kinematic Wave as opposed to the Snyder Unit Hydrograph. Channel routing was calculated using the Muskingum-Cunge method in both models. As shown in Figure 6 below, the peak discharge results from the 2002 study were significantly higher than the results from this model. A larger version of Figure 6 is provided in Appendix D.7.

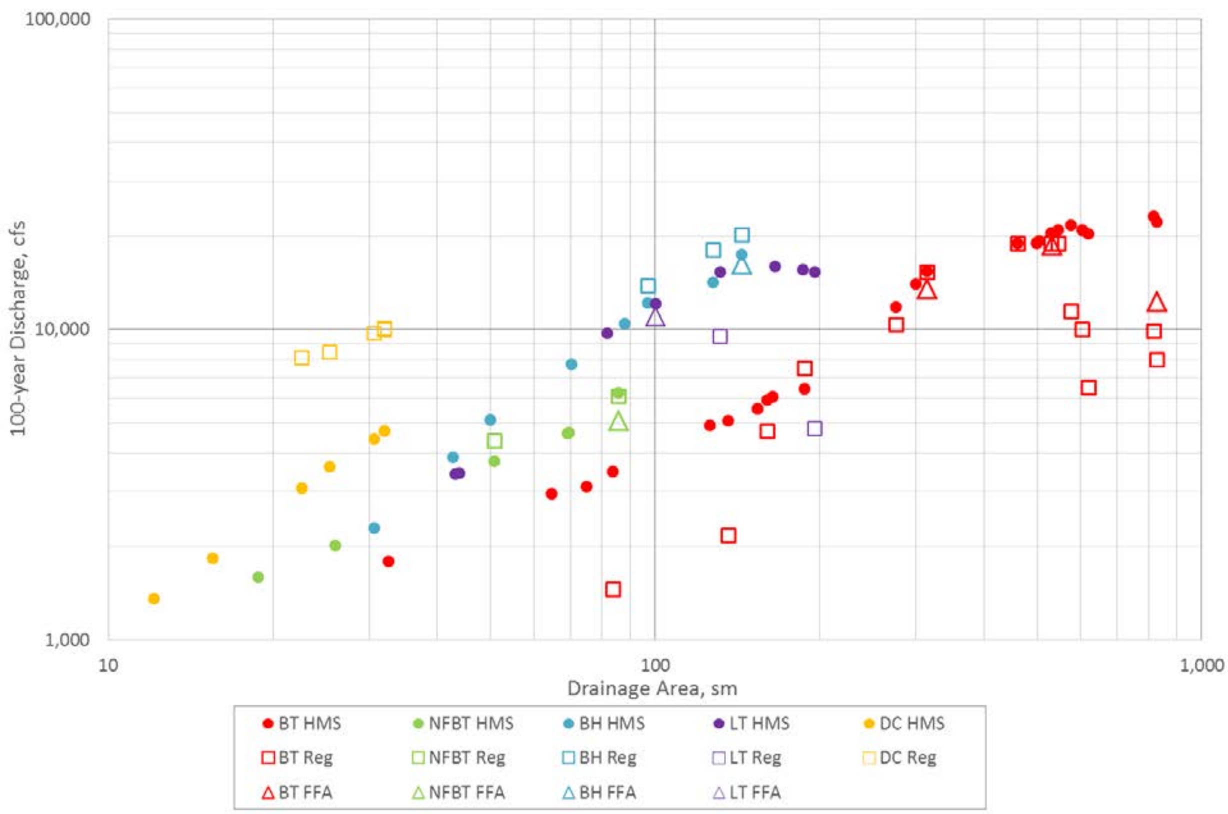
Figure 6. Peak Discharge Profiles for Dry Creek (South)



Attempts to identify the cause of the significant differences in peak discharges were made by evaluating the various inputs but no single input parameter appeared to be causing the differences. It appears the 2002 study resulted in higher peak discharges due to a combination of the shorter, more intense rainfall (3-hour vs. 24-hour), application of additional imperviousness to the curve numbers, use of the kinematic wave transform method, higher discretization of basins (21 vs. 8), and lower Manning's n values for routing. Based on the measured peak discharge of 2,448 cfs during the 2013 flood and comparison of unit discharges with adjacent watersheds as discussed below, the predictive model results from this study seem reasonable for purposes of determining peak discharges on the Big Thompson River.

The predictive peak discharges were also compared on a unit discharge basis (in cfs per square mile of watershed area) 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 Big Thompson Watershed



Watershed (color):
BT = Big Thompson River (red)
NFBT = N. Fork Big Thompson River (green)
BH = Buckhorn Creek (blue)
LT = Little Thompson River (purple)
CD = Dry Creek South (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, particularly on the Big Thompson River.
2. The current regulatory discharges on the Big Thompson River downstream of I-25 appear low relative to the overall trend.
3. The current regulatory discharges for Dry Creek (South) appear high relative to regulatory discharges for adjacent watersheds and relative to the predictive model results.

Appendix D.8 includes an additional plot of discharge versus area comparing the Big Thompson River, Little Thompson River, St. Vrain Creek, Lefthand Creek, and Boulder Creek. The trend further supports that the current regulatory discharges on the Big Thompson River downstream of I-25 are low.

4.0 CONCLUSIONS AND RECOMMENDATIONS

This report documents a hydrologic investigation of the Lower Big Thompson River (South Platte River upstream to confluence with Buckhorn Creek) 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. Based on the current regulatory discharges, the September 2013 flood ranged from less than a 10-year event to greater than a 500-year event. This wide disparity in recurrence interval highlights concerns with the current regulatory peak discharges and therefore 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. Comparison of Peak Discharge Estimates

	2013 Effective FIS Peak Discharge				Ayres 2013 Updated				2013 Flood	2013 Flood
	Approximate Location for Comparison				Flood Frequency Analysis				Estimated Peak Discharge (cfs)	Estimated Recurrence Interval (years)
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)		
BT at confluence with Buckhorn Creek	4,700	12,300	19,000	44,000					19,000	100 Year
BT at Railroad Avenue	4,700	12,300	19,000	44,000						
BT at Highway 287					3,620	11,800	18,700	51,500	22,000	100 Year
BT downstream of County Road 9E	4,700	12,300	19,000	44,000						
BT at Interstate 25	4,300	8,800	11,500	21,000					19,600	500 Year
BT upstream of County Road 15 ½	3,600	7,600	10,000	18,500						
BT upstream of Little Thompson Confluence	2,200	4,700	6,500	12,000					17,700	> 500 Year
BT below Little Thompson Confluence	3,200	7,300	9,900	20,000						
BT at Confluence with South Platte River	2,500	5,900	8,000	15,000	2,900	8,260	12,300	29,000	24,900	> 500 (FIS) < 500 (FFA)
Little Thompson above Confluence with BT	1,630	3,600	4,800	8,400					18,000	> 500 Year
Dry Creek (South) at Golf Course	3,020	7,470	10,090	17,140					2,450	< 10 Year

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 Big Thompson gages in Loveland and at La Salle are provided in Appendix B. Table 8 below shows a summary of the updated flood frequency analysis for the Big Thompson River. The flood frequency analysis results match well at the Loveland gage but are considerably higher than the current regulatory peak discharges at the La Salle gage.

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 the Upper Big Thompson River and the Little Thompson River which were developed in separate models. 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. 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 predictive model results are summarized in Table 7 and Appendix D.

Table 9 compares the predictive peak discharges from this modeling effort to current regulatory discharges for the 100-year event. Since the rainfall/runoff model results are more consistent in terms of peak discharge translation downstream than the current regulatory flows in Big Thompson downstream of I-25 and because there does not appear to be any justification for the sharp drop in regulatory peak discharges between County Road 9E and Interstate 25 resulting from the two original USACE studies, it is recommended that the model results be considered for adoption as the updated regulatory peak discharges along the Big Thompson River.

It should be noted that this study was focused on peak discharge estimation in the Big Thompson River 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. Dry Creek (South) is an example where there is a significant difference between the predictive peak discharges and the current regulatory discharges. However, since the 100-yr unit discharges for the current regulatory values on Dry Creek (South) are high (315 cfs/mi^2) relative to predictive model unit discharges (150 cfs/mi^2) and to unit discharges for adjacent watersheds as shown on Figure 7 and because the 2013 flood was less than a 10-year storm when compared to the current regulatory values, it is recommended that this area be considered for a more focused study and re-evaluation of regulatory peak discharges.

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

Location	Current Regulatory Discharge (cfs)	Modeled Discharge (cfs)	Percent Difference
Big Thompson at Buckhorn Creek	19,000	18,900	- 1%
Big Thompson at Railroad Avenue	19,000	20,400	7%
Big Thompson downstream of County Road 9E	19,000	21,000	11%
Big Thompson at Interstate 25	11,500	21,800	90%
Big Thompson upstream of County Road 15 ½	10,000	21,000	110%
Big Thompson above Little Thompson Confluence	6,500	20,300	212%
Big Thompson below Little Thompson Confluence	9,900	23,200	134%
Big Thompson at South Platte River	8,000	22,200	178%
Little Thompson River at Big Thompson River	4,800	15,400	221%
Dry Creek (South) below Pinewood Reservoir	300	400	33%
Dry Creek (South) at Skinner Gulch Confluence	2,920	1,060	- 64%
Dry Creek (South) below Flatiron Reservoir	8,130	3,070	- 62%
Dry Creek (South) at County Road 29	8,430	3,610	- 57%
Dry Creek (South) at County Road 23E	9,720	4,440	- 54%
Dry Creek (South) at Big Thompson River	10,090	4,710	- 53%

Based on the predictive model discharges for the return periods analyzed, as shown in Table 10 below, the peak discharge observed along the Big Thompson River during the September 2013 flood event was approximately a 1 percent annual chance peak discharge (100-year storm) upstream of the confluence with the Little Thompson River. The Little Thompson River also experienced a 1 percent annual chance peak discharge based on the report prepared by CH2M Hill. The combined peak discharge downstream of the confluence was between a 1 percent annual chance peak discharge and a 0.2 percent annual chance peak discharge, or between a 100-year and 500-year storm. It makes sense that the flood downstream of the confluence was slightly higher than a 100-year event since it is rare for both rivers to experience a 100-year storm simultaneously.

Based on a review and evaluation of assumptions and limitations of the various hydrologic methodologies used, 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 in the Big Thompson watershed. It is recommended that local floodplain administrators consider using the results of this hydrologic analysis to update and revise current regulatory discharges on the Big Thompson River.

Table 10. Estimate of September 2013 Peak Discharge Recurrence Interval

Location	Drainage Area (mi^2)	Estimated Discharge (cfs)	Annual Chance Peak Discharge (cfs)					Estimated Recurrence Interval (yr)
			10%	4%	2%	1%	0.2%	
Big Thompson at Buckhorn Creek	461	19,000	4,530	8,580	13,000	18,900	41,800	~ 100
Big Thompson at Wilson Road	499	22,000	4,320	8,370	12,900	19,000	40,400	~ 100
Big Thompson at Highway 287	531	22,000	4,700	8,980	13,900	20,400	42,600	~ 100
Big Thompson at Interstate 25	577	19,600	5,090	9,530	14,900	21,800	45,100	~ 100
BT upstream of Little Thompson Confluence	620	17,700	4,900	9,050	13,900	20,300	44,200	~ 100
BT at Confluence with South Platte River	829	24,900	4,770	8,970	14,800	22,200	53,600	~ 100
Little Thompson above Confluence with BT	196	18,000	4,480	7,160	10,500	15,400	31,400	~ 100
Dry Creek (South) at Golf Course	32	2,450	1,390	2,400	3,460	4,710	8,540	~ 25

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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**

DRAFT

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.

HEC-RAS Modeling:

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.

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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.

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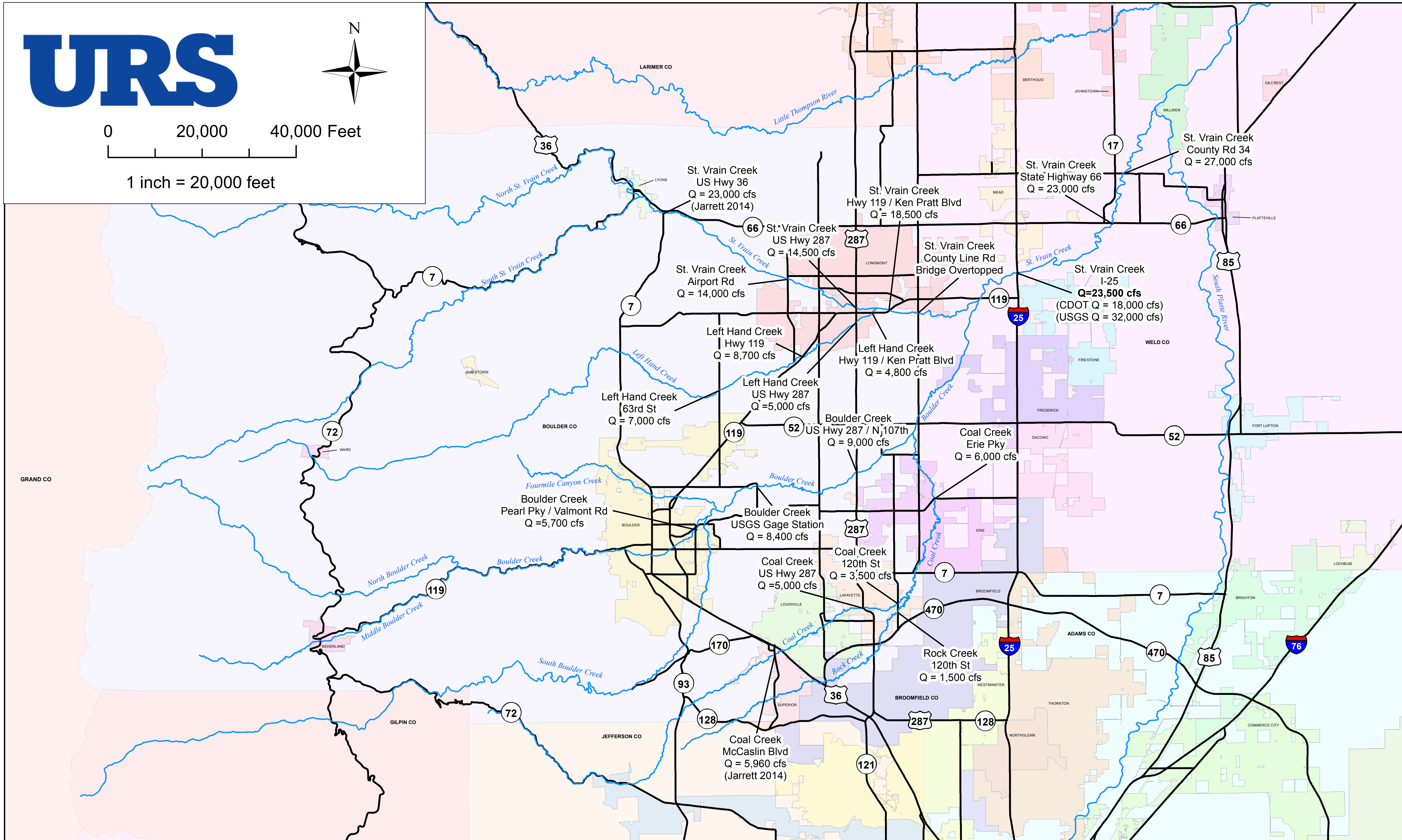
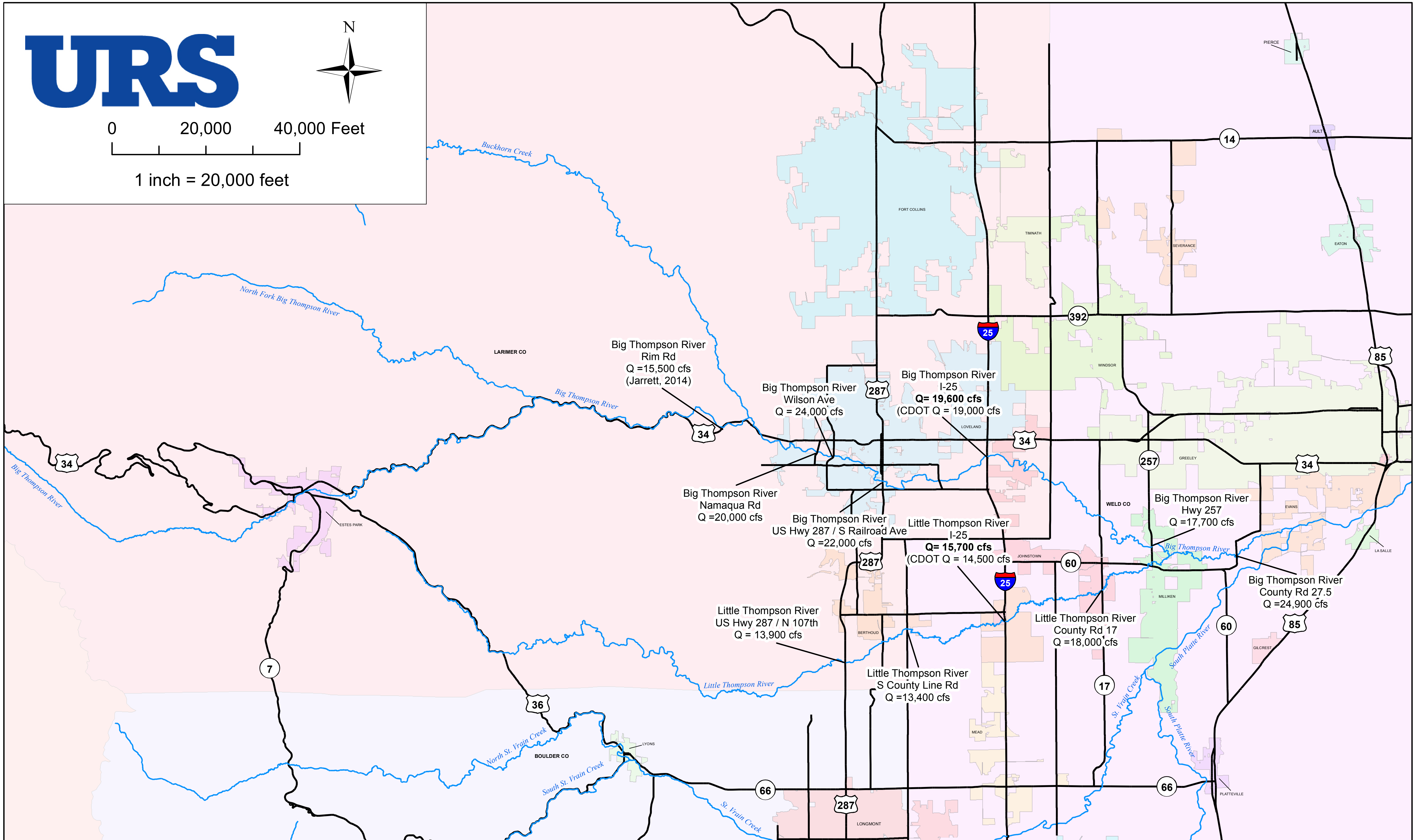
Peak Flow Discharge for the September 2013 Floods - St. Vrain Creek Drainage Area



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



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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.

Boulder Creek:

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.

Rock and Coal Creek:

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.

Lefthand Creek:

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.

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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.

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REFERENCES

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

CDOT hydraulic models

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Appendix A
HEC-RAS Results

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Little Thompson
HEC-RAS Results

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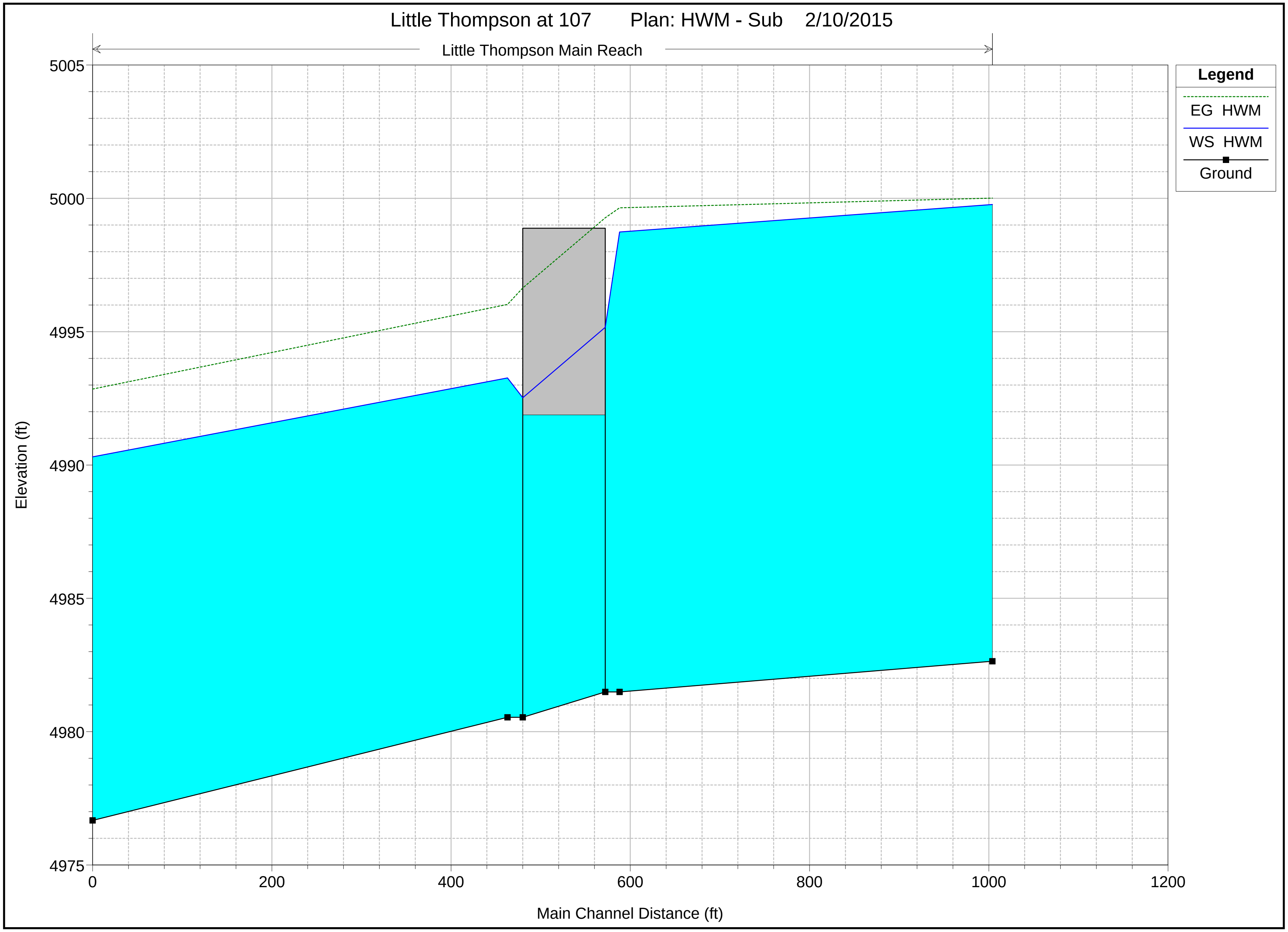
Little Thompson at 107th

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 107th

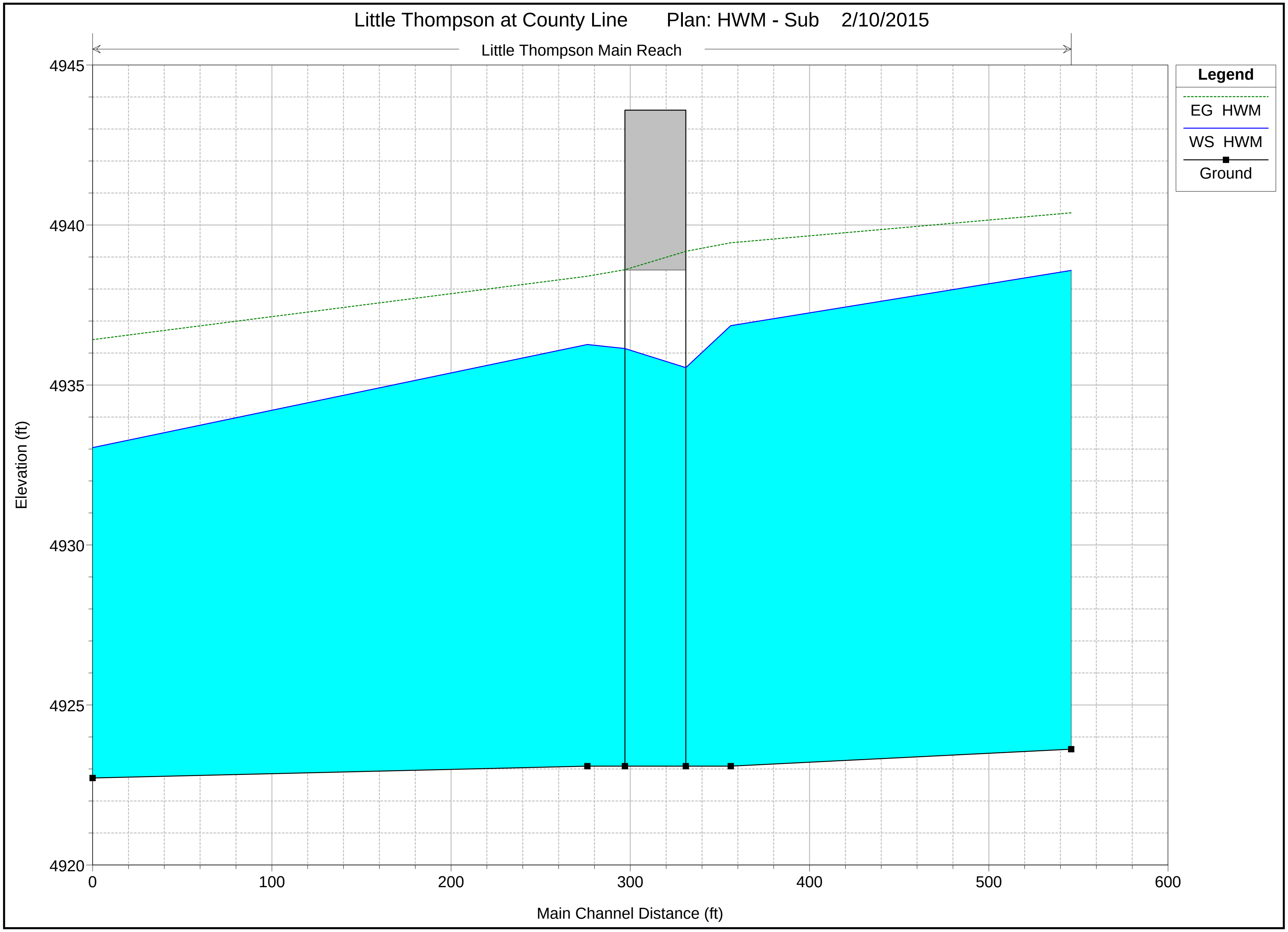


Little Thompson at S County Line

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

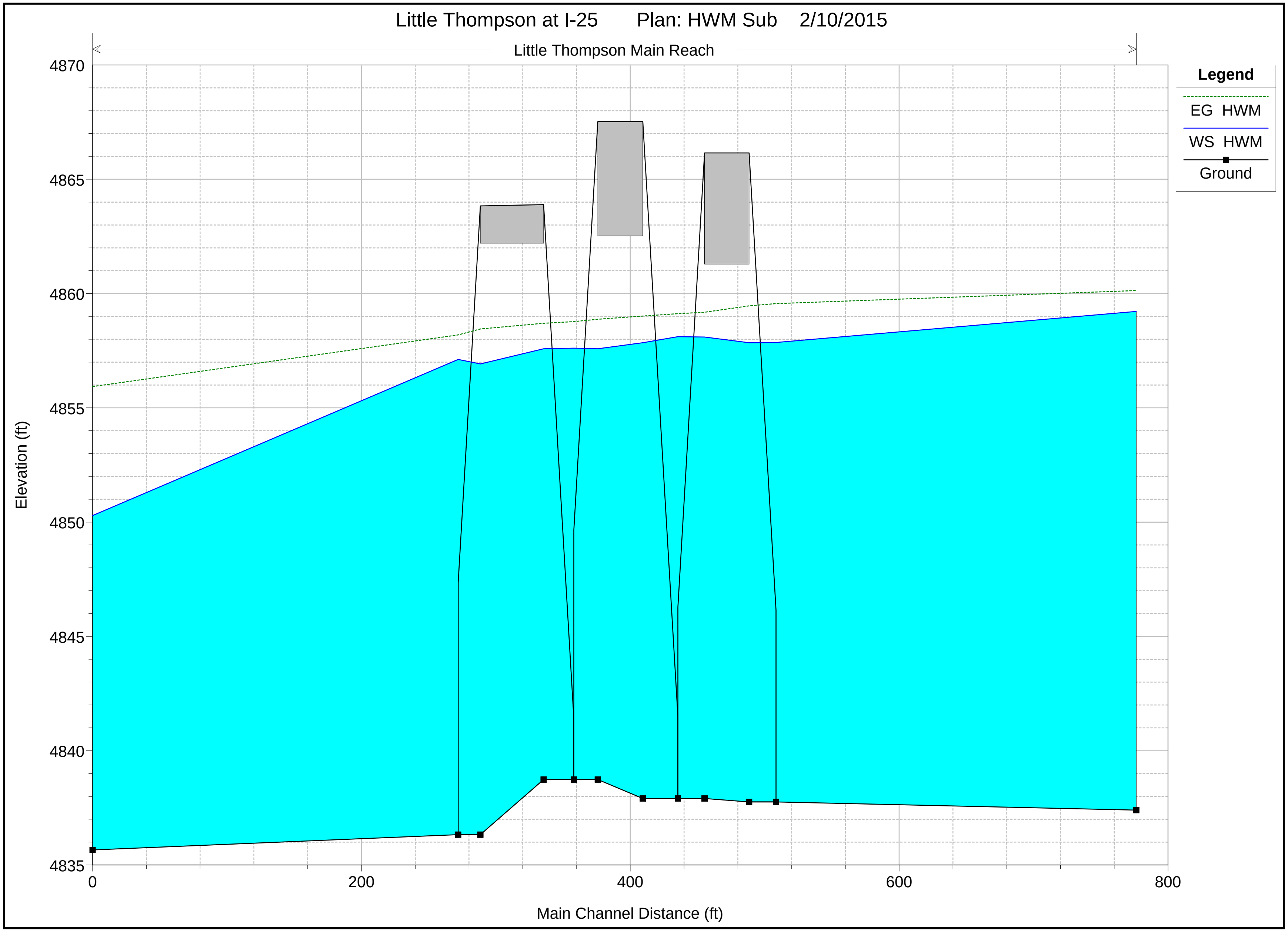
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Reach	675	HWM	13400.00	4923.62	4938.58	4936.06	4940.38	0.003837	12.09	1361.73	144.75	0.59
Main Reach	460	HWM	13400.00	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



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

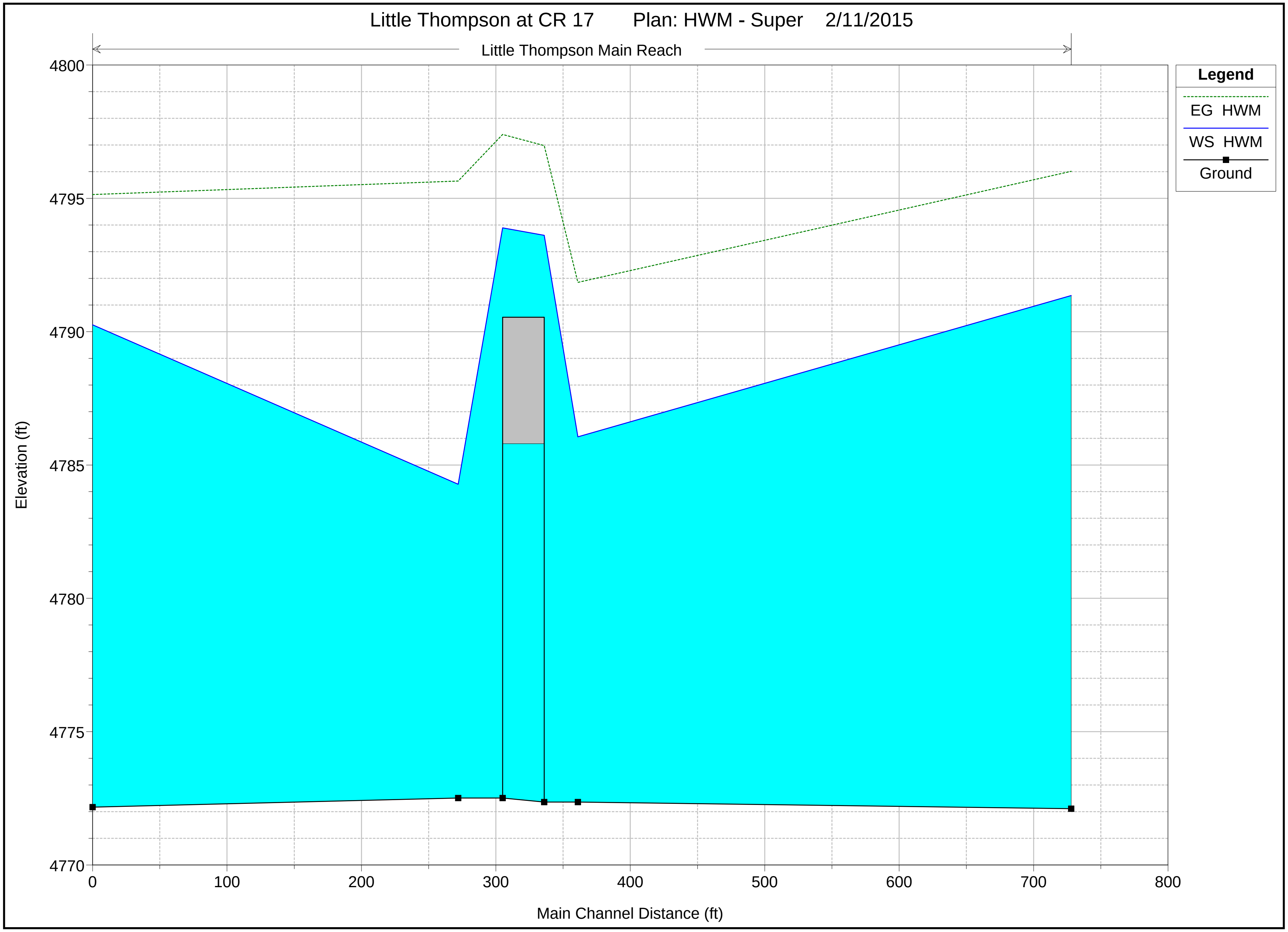
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Reach	2595.635	HWM	15700.00	4837.40	4859.22	4853.35	4860.13	0.001358	8.57	2069.01	219.51	0.35
Main Reach	2327.635	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
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4800
4795
4790
4785
4780
4775
4770
0
100
200
300
400
500
600
700
800
Main Channel Distance (ft)

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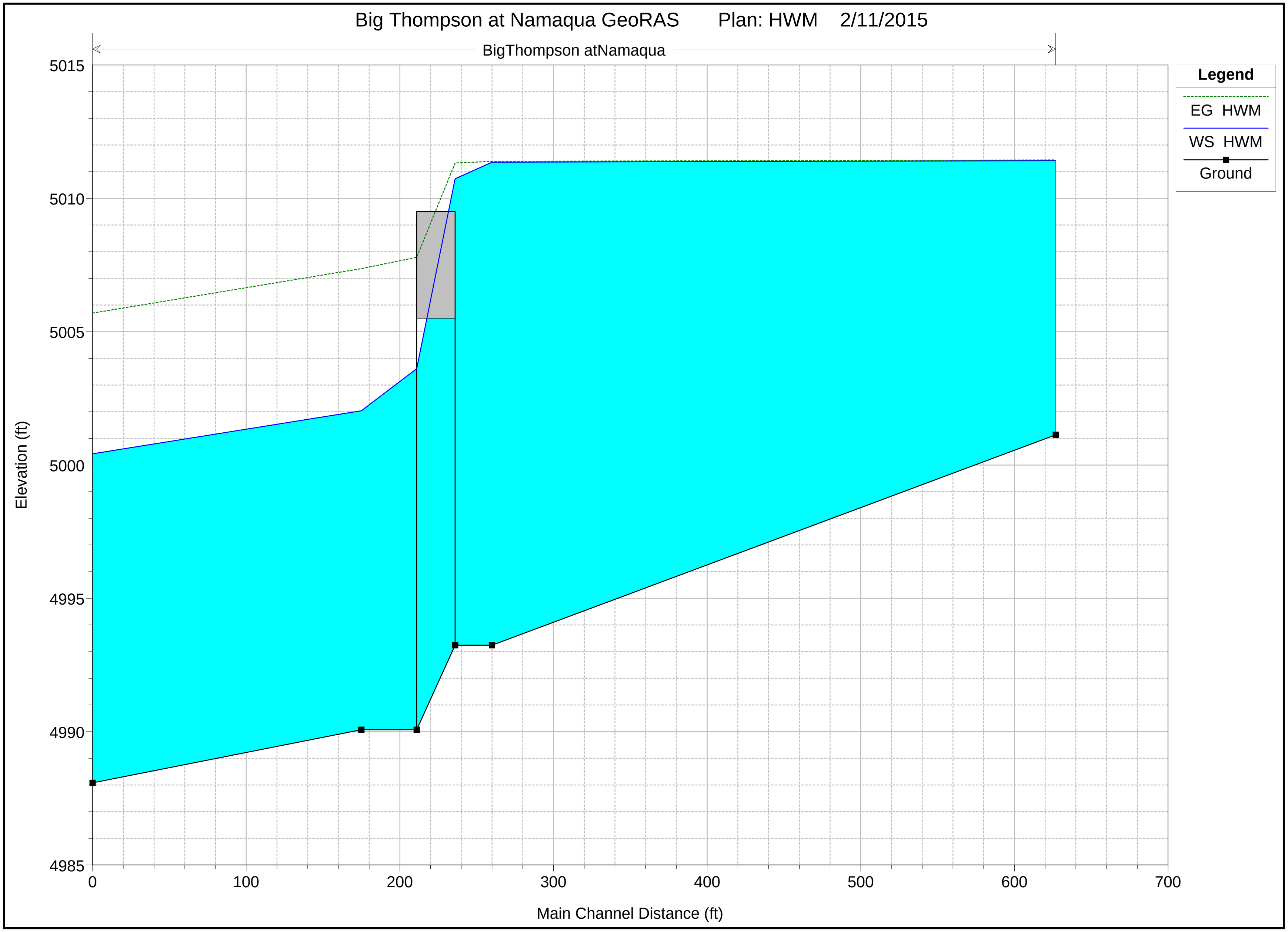
Big Thompson
HEC-RAS Results

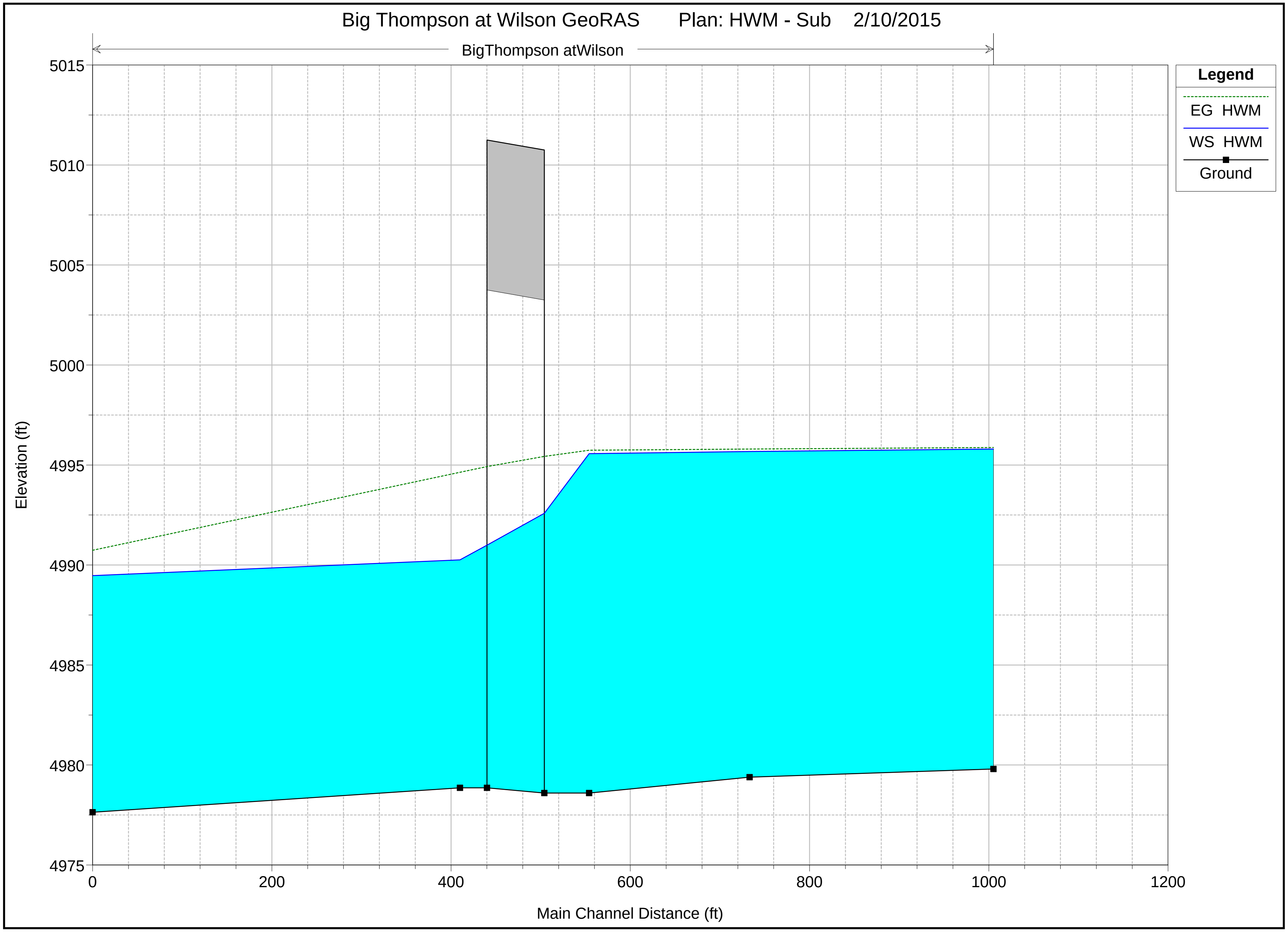
HEC-RAS Plan: HWM - Sub   River: BigThompson   Reach: atNamaqua   Profile: HWM

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)

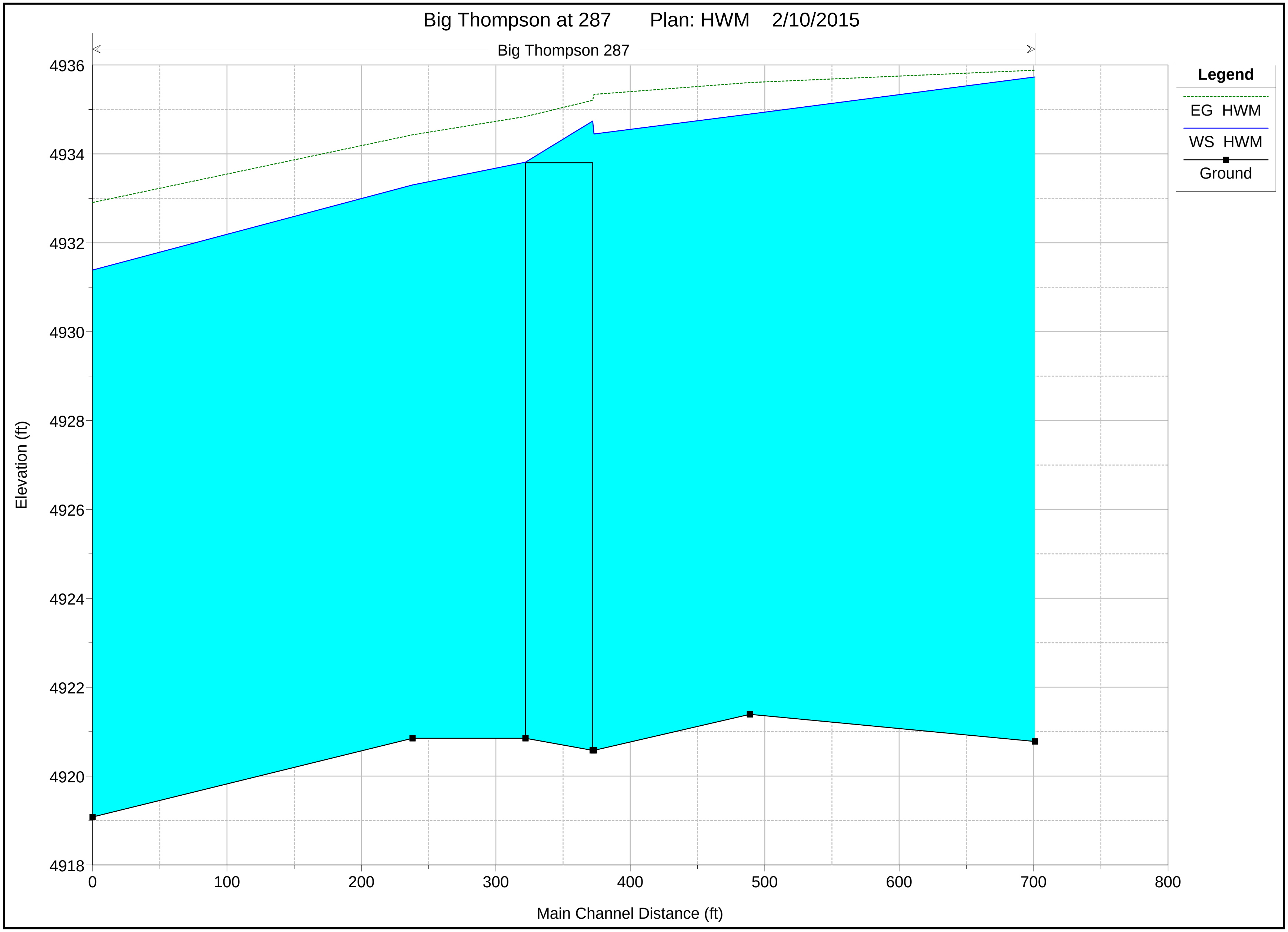
HEC-RAS Plan: HWM - Sub   River: BigThompson   Reach: atWilson   Profile: HWM

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)

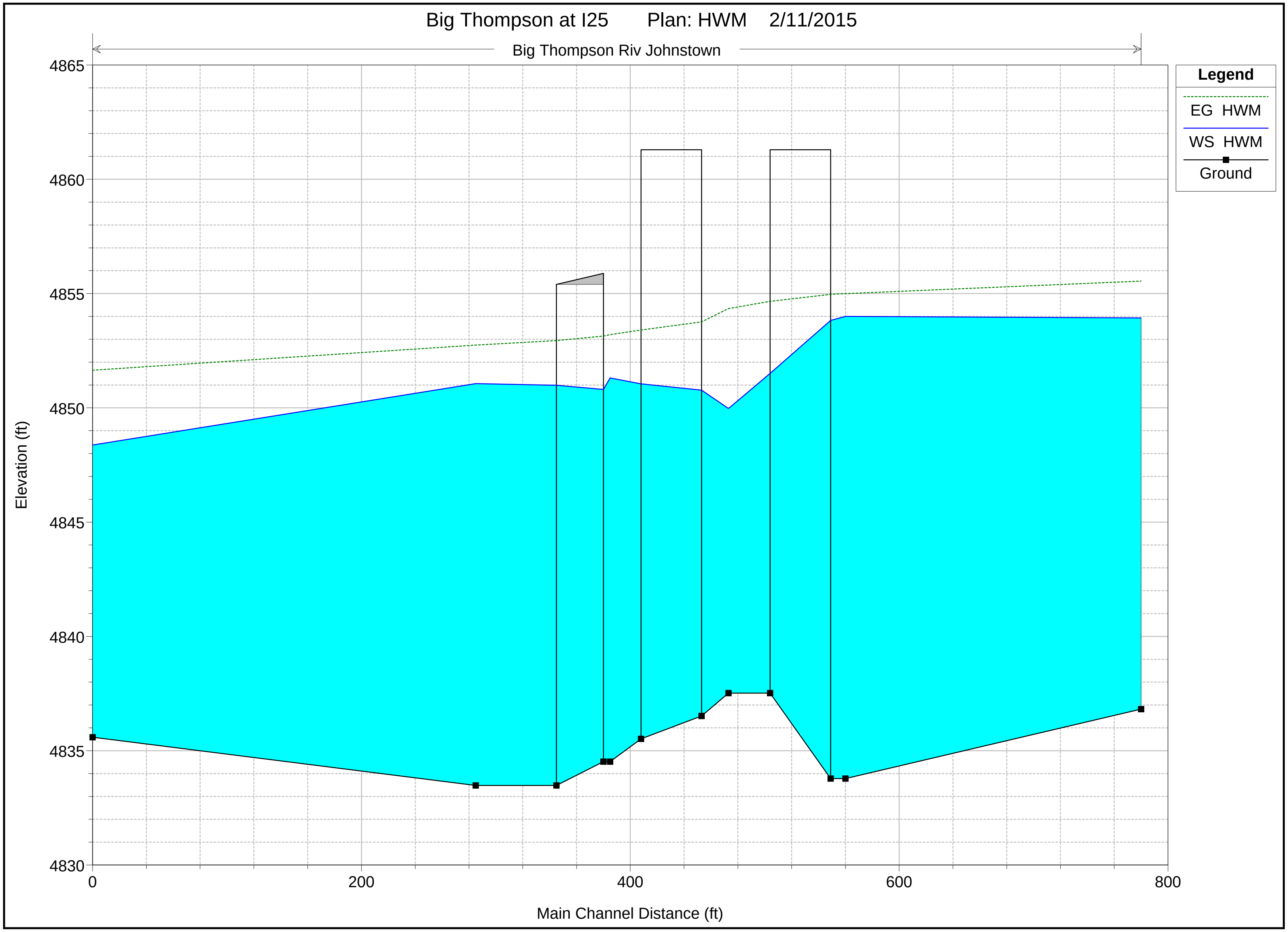
HEC-RAS Plan: HWM - Sub River: Big Thompson Reach: 287 Profile: HWM

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)

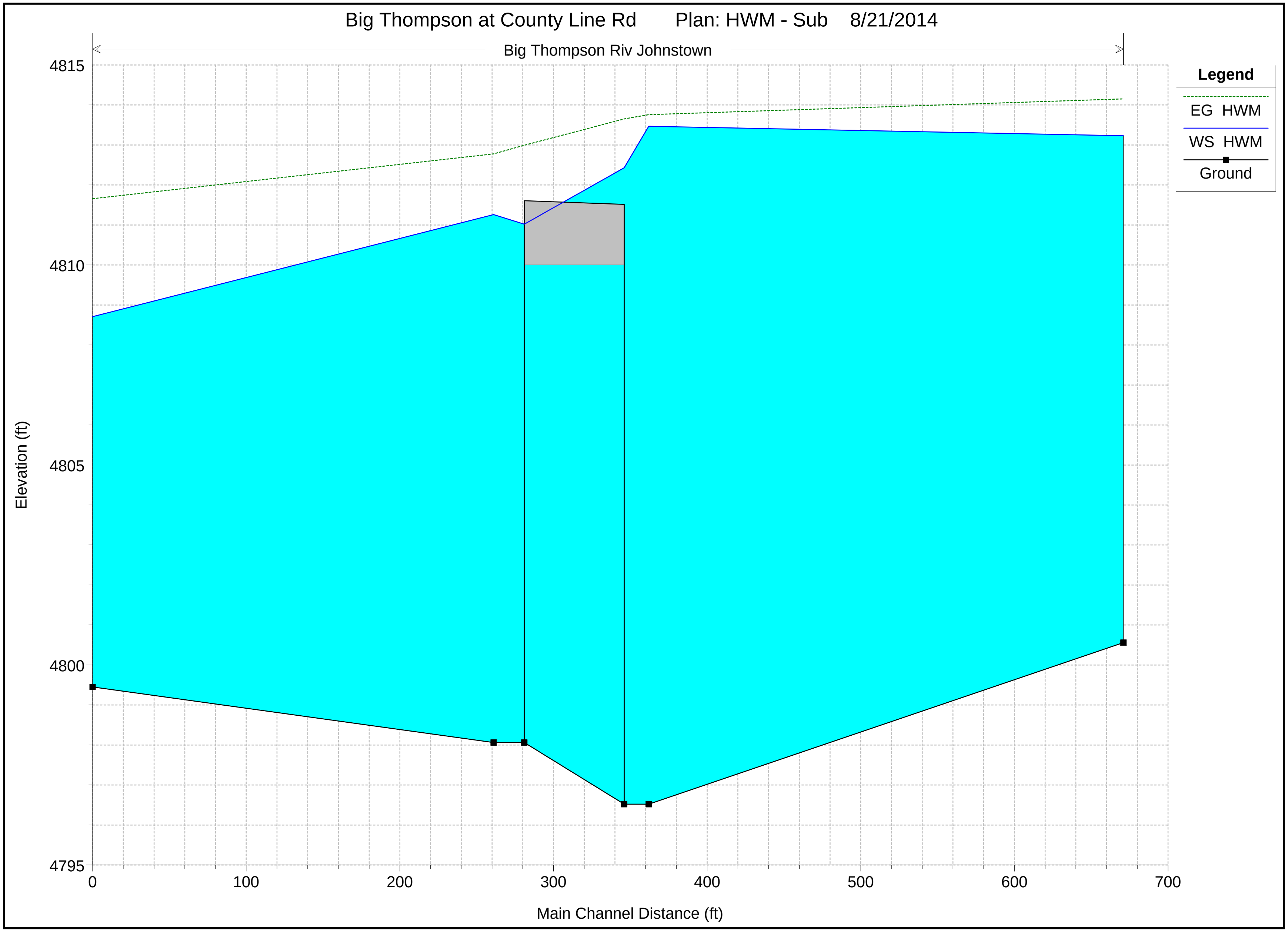
HEC-RAS Plan: HWM - Sub River: Big Thompson Riv Reach: Johnstown Profile: HWM

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 I25 Plan: HWM 2/11/2015
Big Thompson Riv Johnstown
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4865
4860
4855
4850
4845
4840
4835
4830
0
200
400
600
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	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

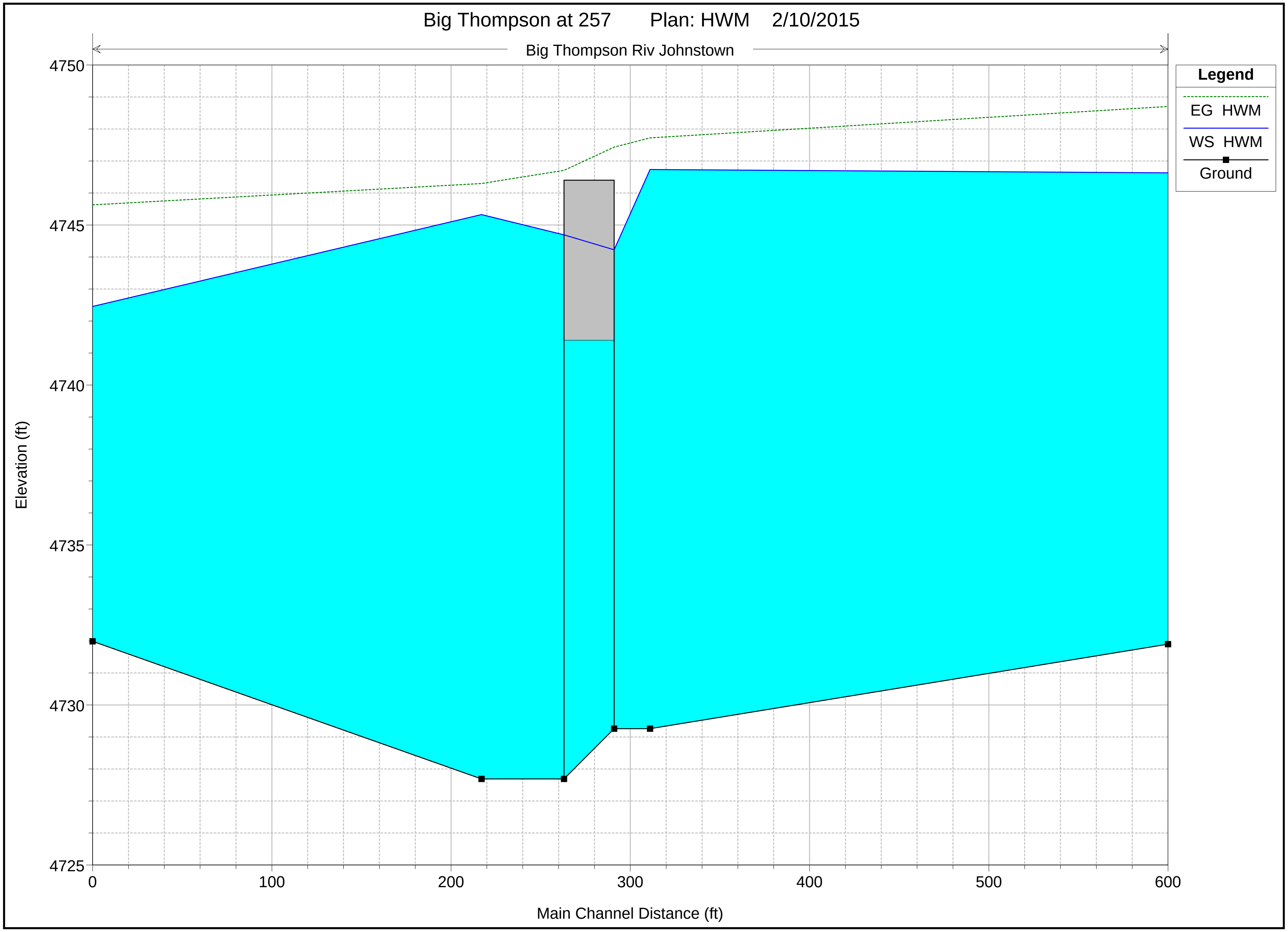


Big Thompson at County Line Rd Plan: HWM - Sub 8/21/2014
Big Thompson Riv Johnstown
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4815
4810
4805
4800
4795
0
100
200
300
400
500
600
700
Main Channel Distance (ft)

Big Thompson at 257

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

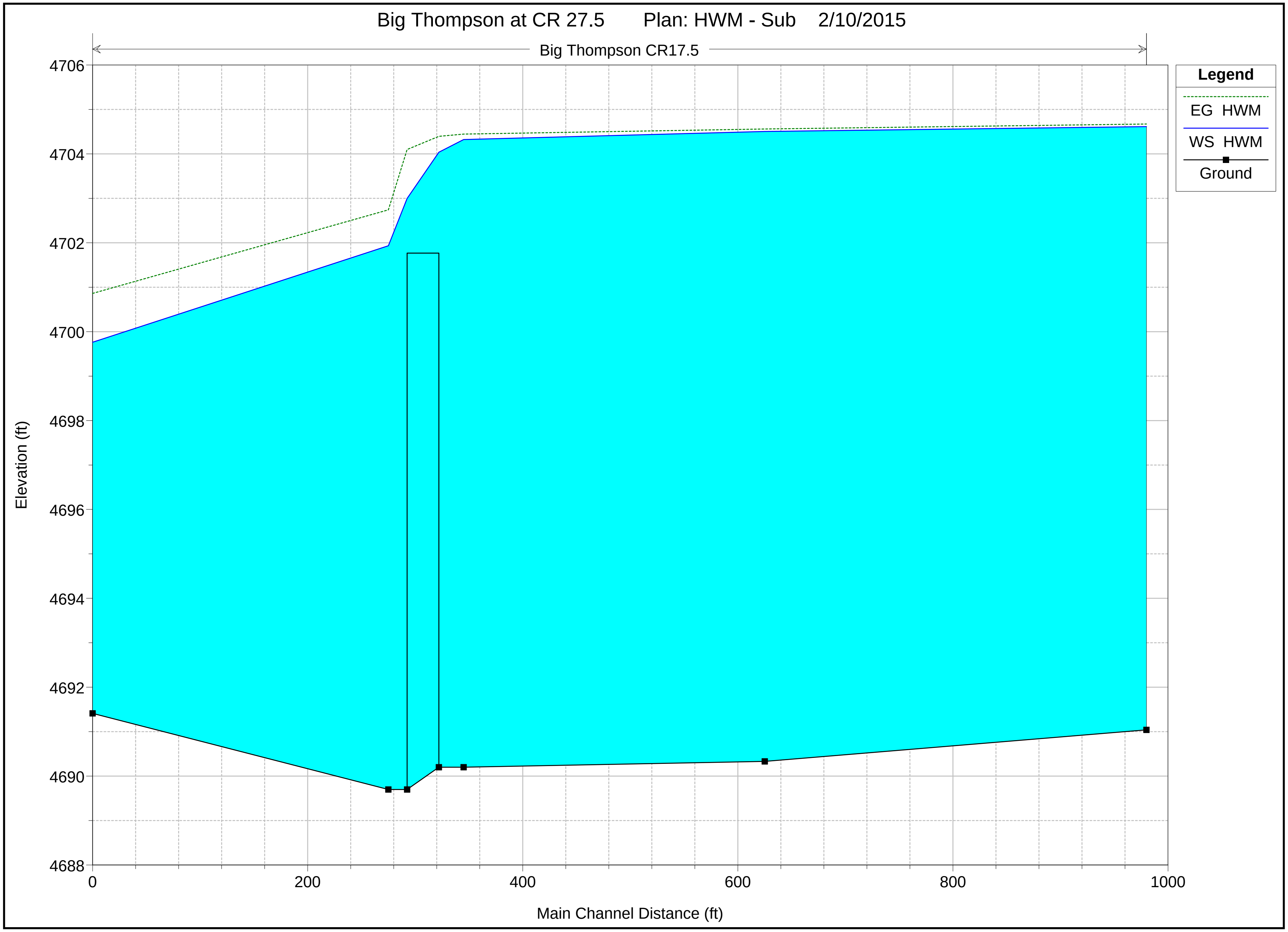
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Johnstown	1600	HWM	17700.00	4731.90	4746.63	4744.57	4748.70	0.004722	14.95	1728.32	172.67	0.72
Johnstown	1311	HWM	17700.00	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 Plan: HWM - Sub 2/10/2015
Big Thompson CR17.5
Legend
EG HWM
WS HWM
Ground
4706
4704
4702
4700
4698
4696
4694
4692
4690
4688
Elevation (ft)
0
200
400
600
800
1000
Main Channel Distance (ft)

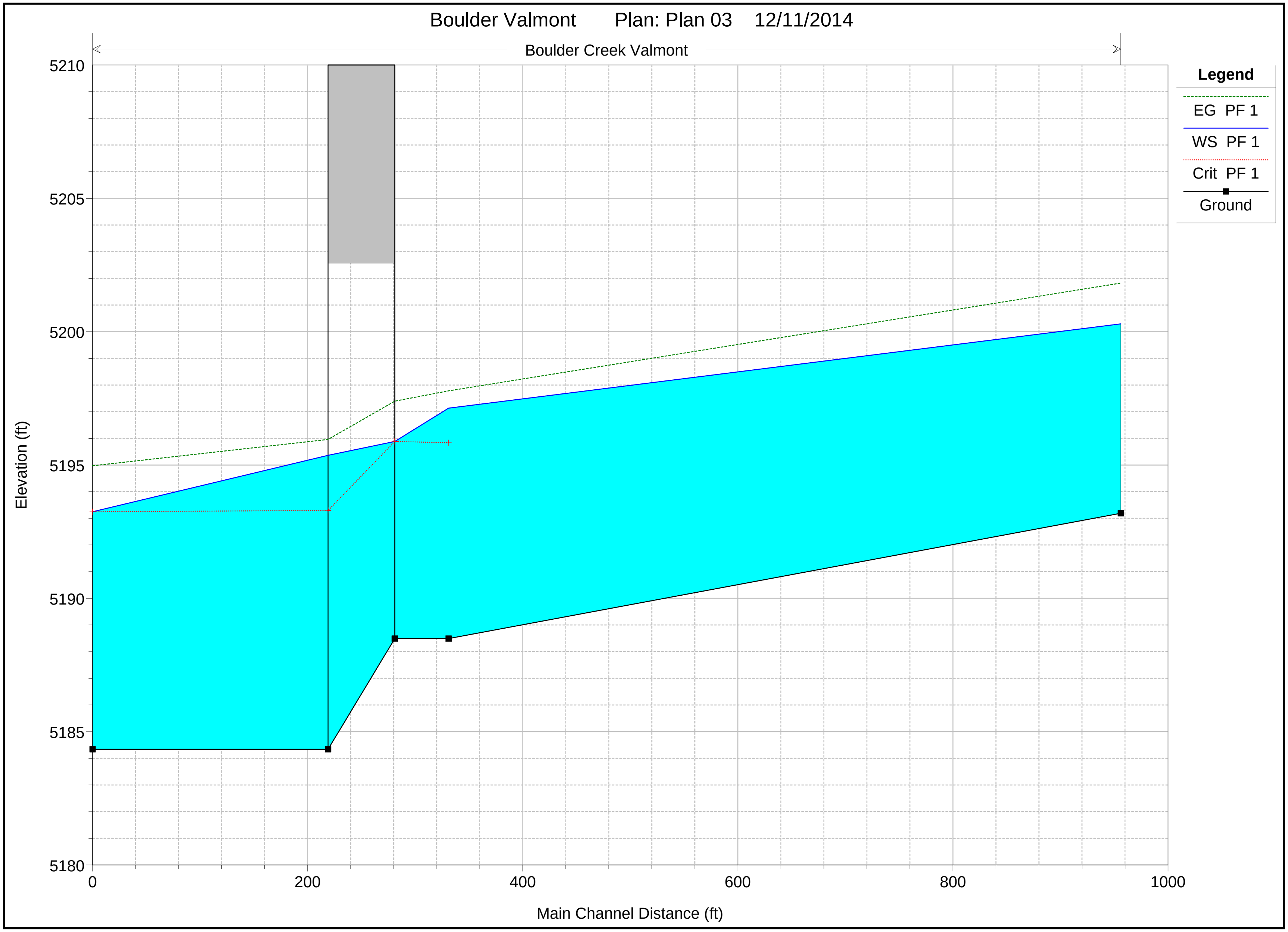
DRAFT

Boulder Creek
HEC-RAS Results

Boulder at Valmont

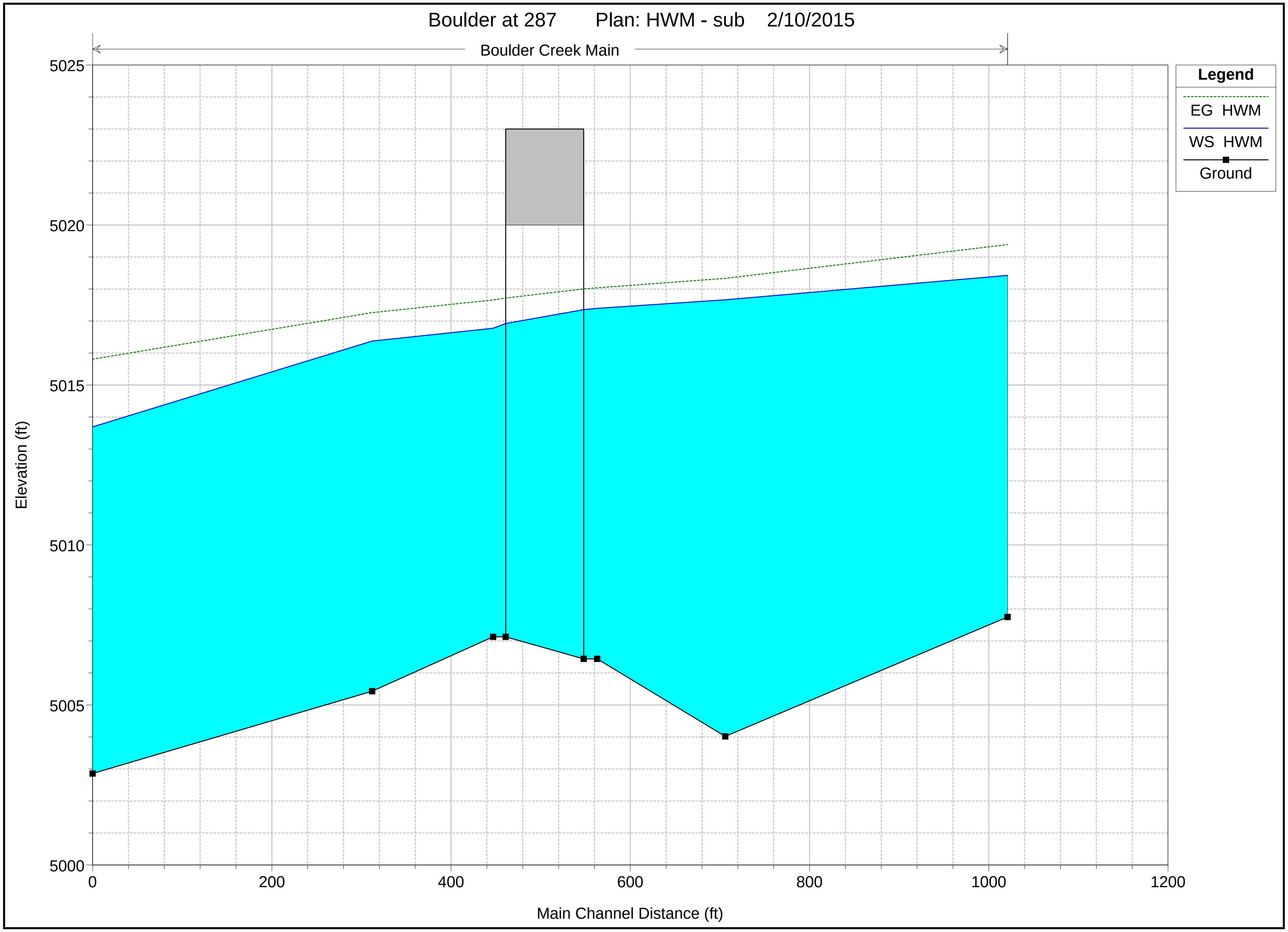
HEC-RAS Plan: Plan 03   River: Boulder Creek   Reach: Valmont   Profile: PF 1

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 Valmont Plan: Plan 03 12/11/2014
Boulder Creek Valmont
5210
5205
5200
5195
5190
5185
5180
Elevation (ft)
0
200
400
600
800
1000
Main Channel Distance (ft)
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground

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

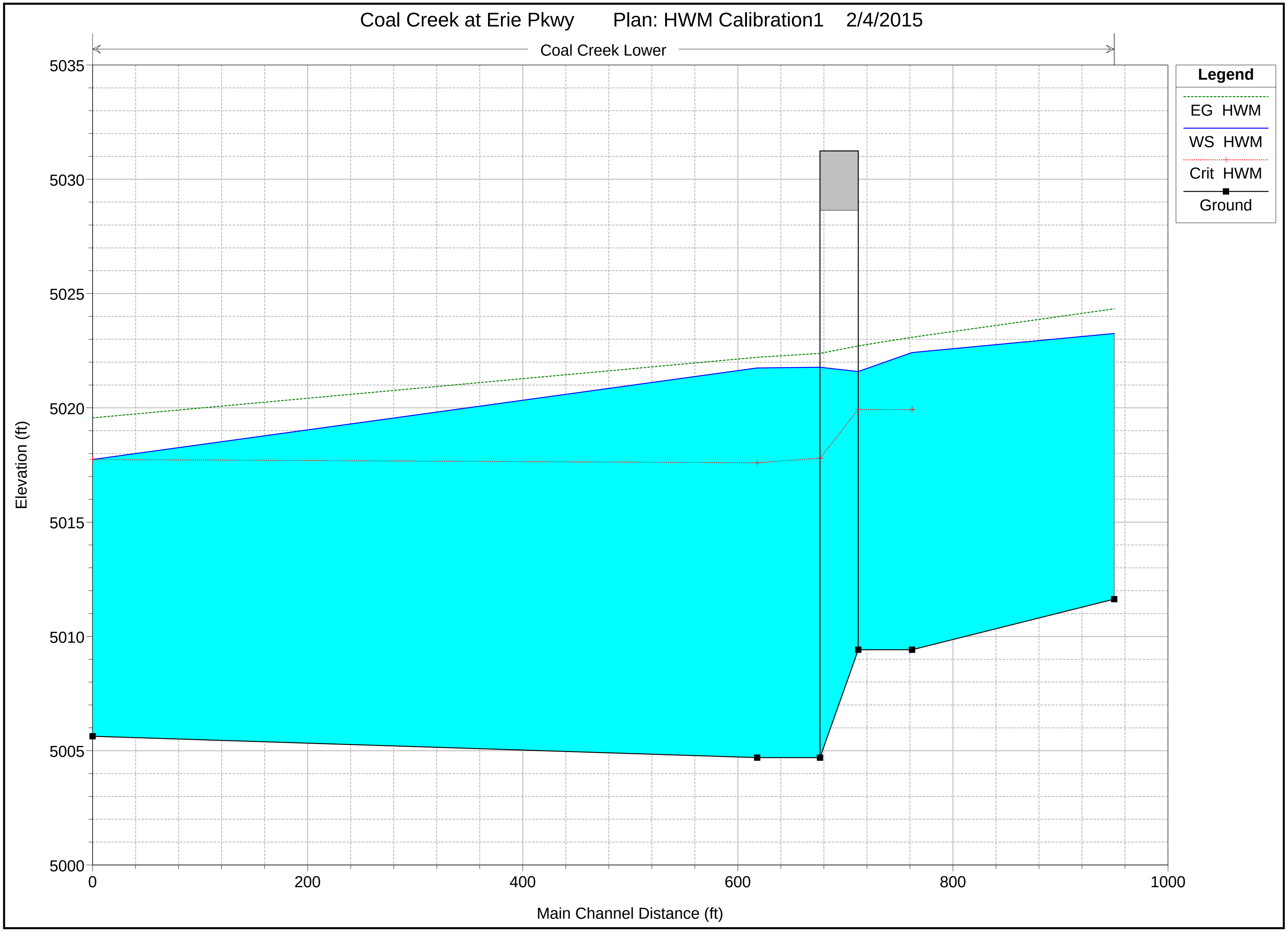
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main	2021	HWM	9000.00	5007.75	5018.43	5018.10	5019.39	0.004807	10.13	1529.85	446.05	0.61
Main	1706	HWM	9000.00	5004.02	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
5025
5020
5015
5010
5005
5000
Elevation (ft)
0
200
400
600
800
1000
1200
Main Channel Distance (ft)

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

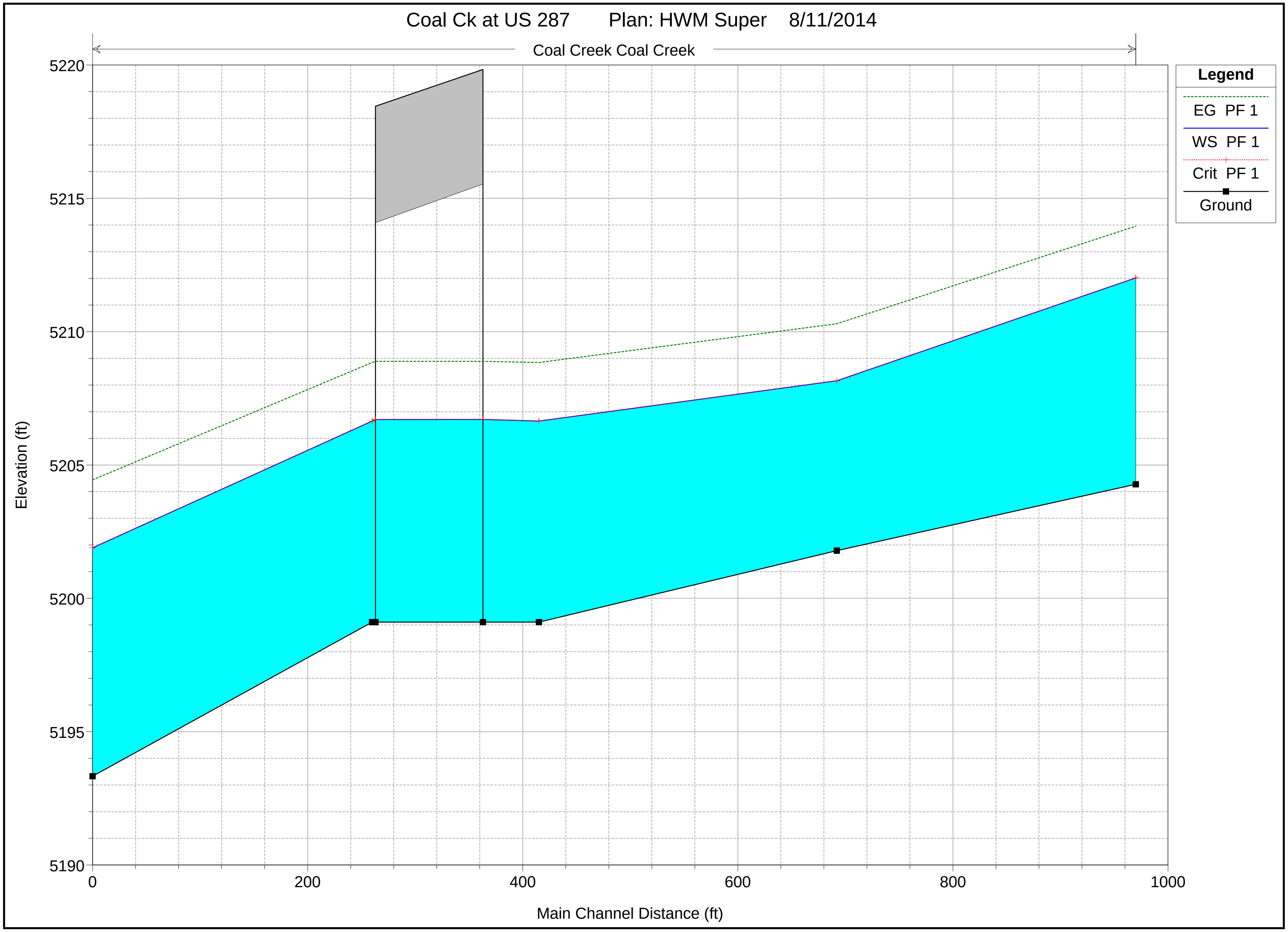
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	19370	HWM	6000.00	5011.63	5023.26		5024.33	0.008024	9.22	740.79	143.95	0.51
Lower	19182	HWM	6000.00	5009.42	5022.42	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



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

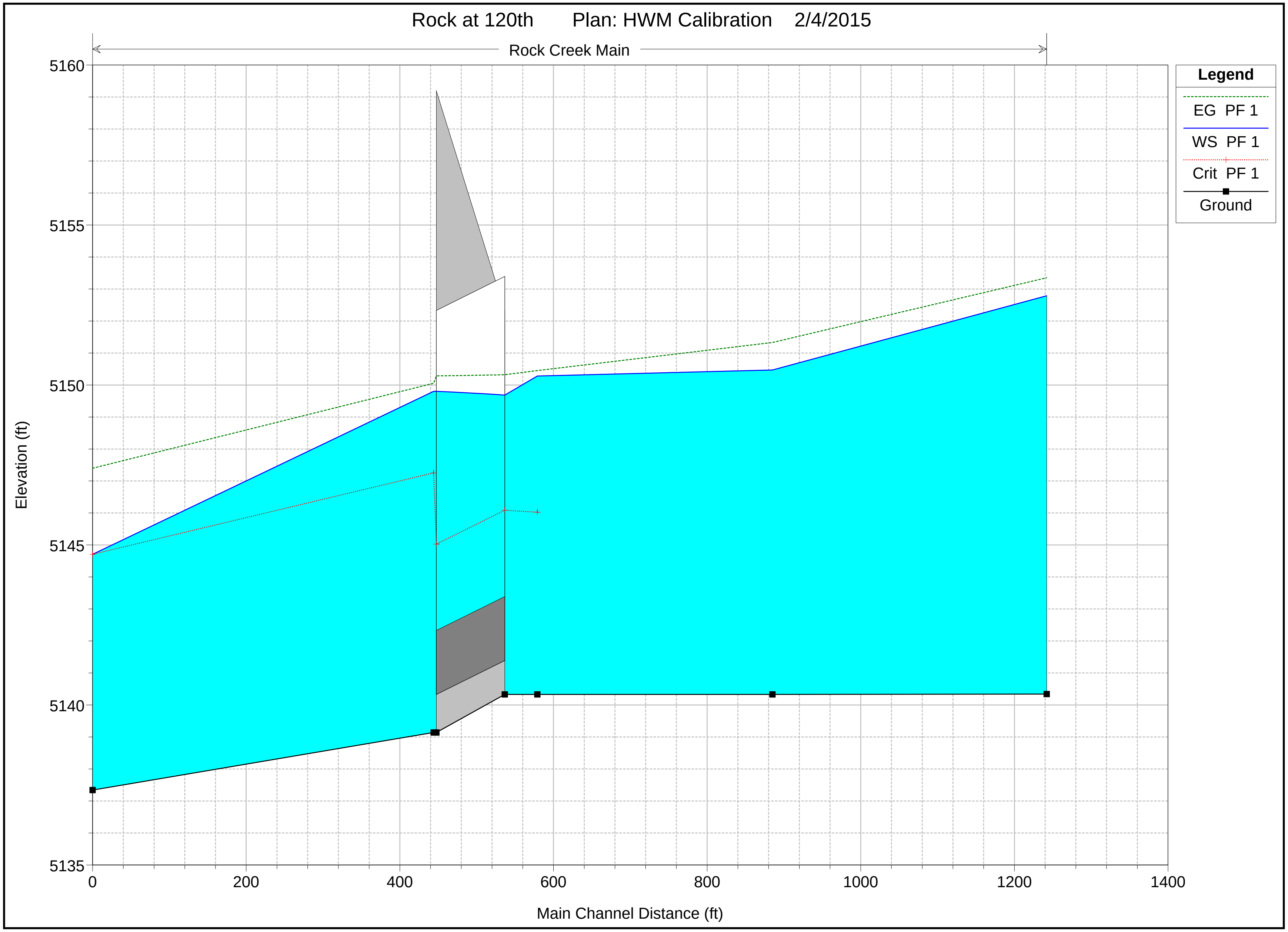

Coal Ck at US 287 Plan: HWM Super 8/11/2014
Coal Creek Coal Creek
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
5220
5215
5210
5205
5200
5195
5190
0
200
400
600
800
1000
Main Channel Distance (ft)

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

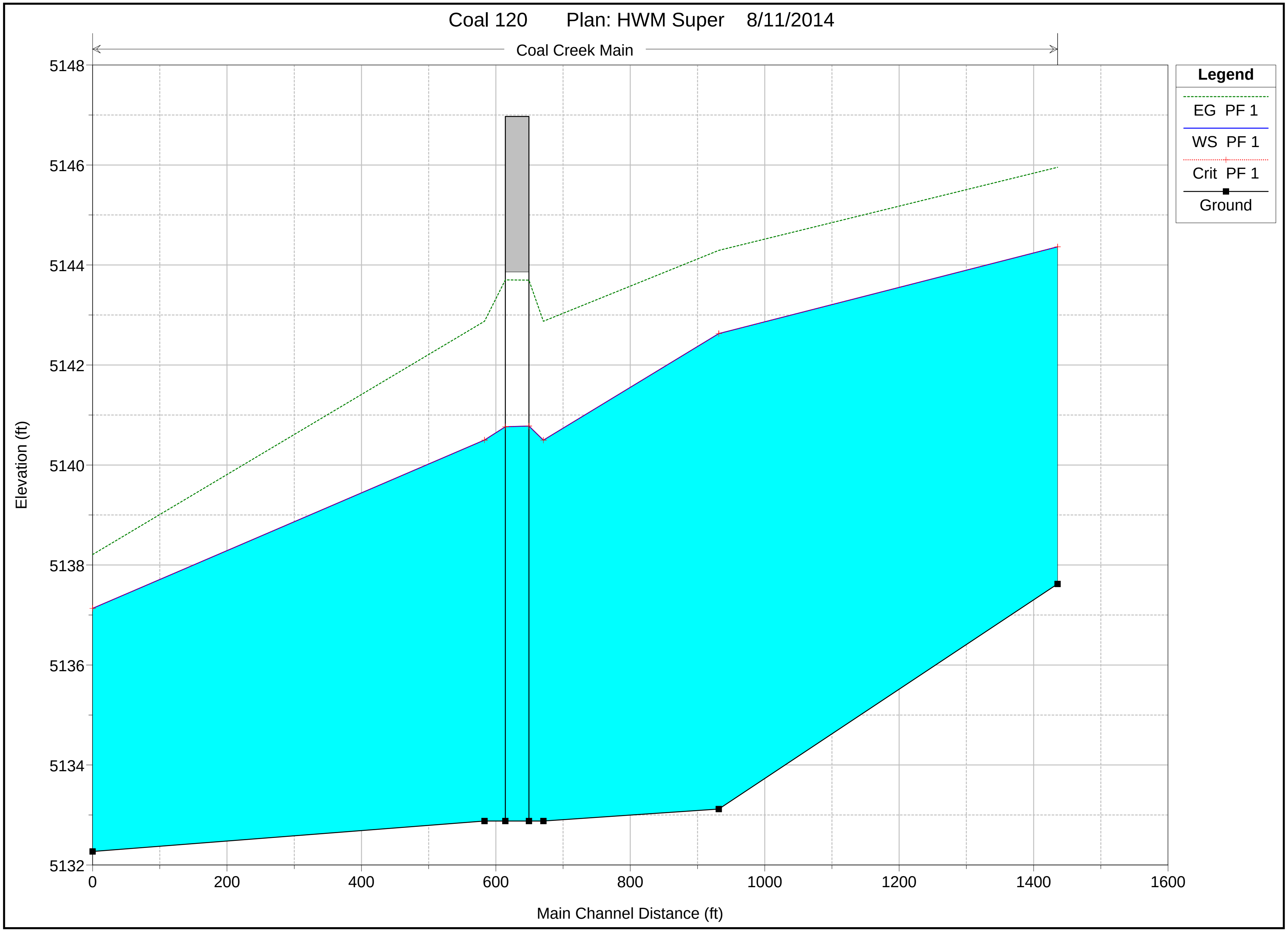


Coal at 120

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

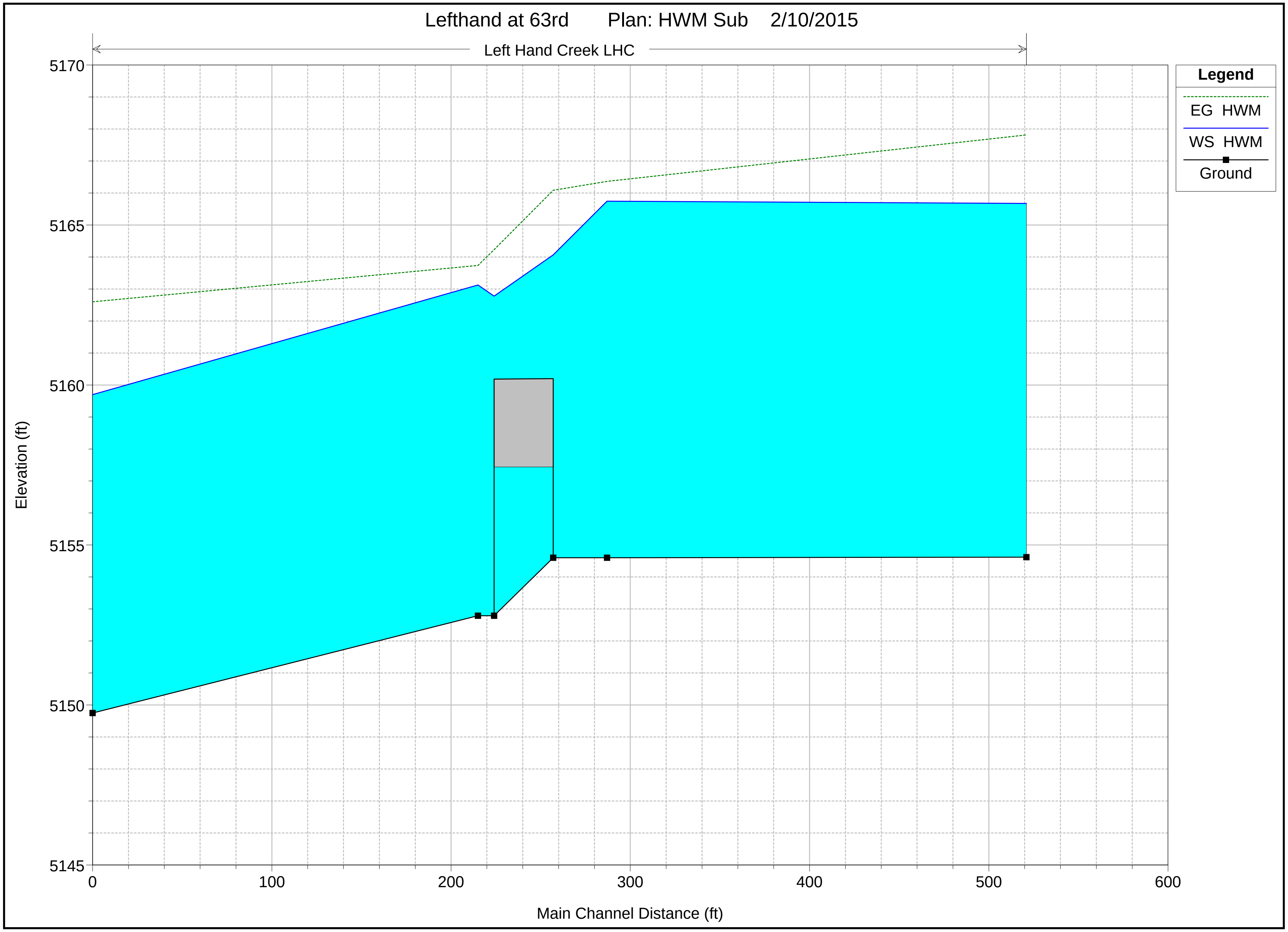
DRAFT

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

Left Hand at 63rd

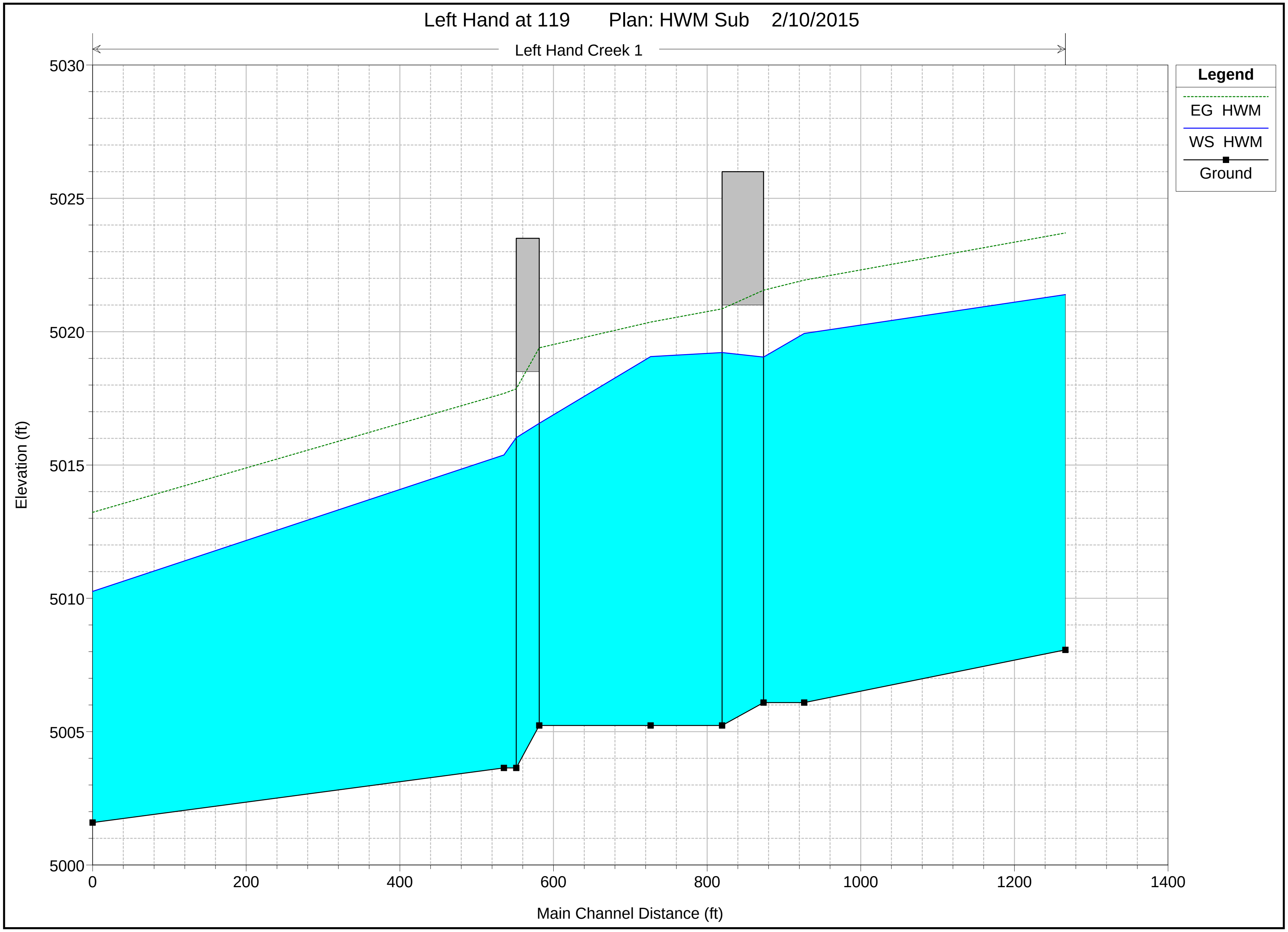


Left Hand at 119th

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

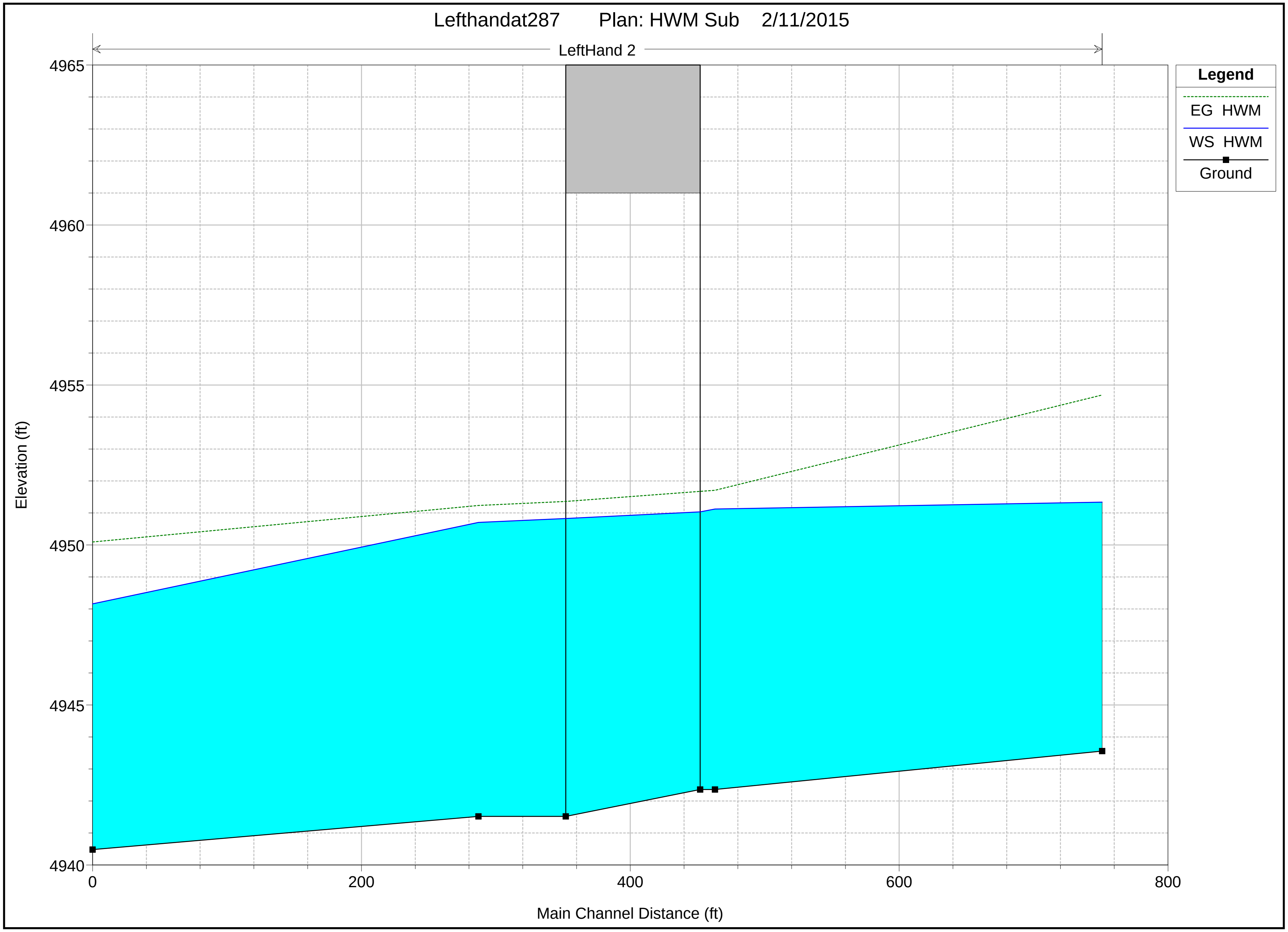


Left Hand at 287

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

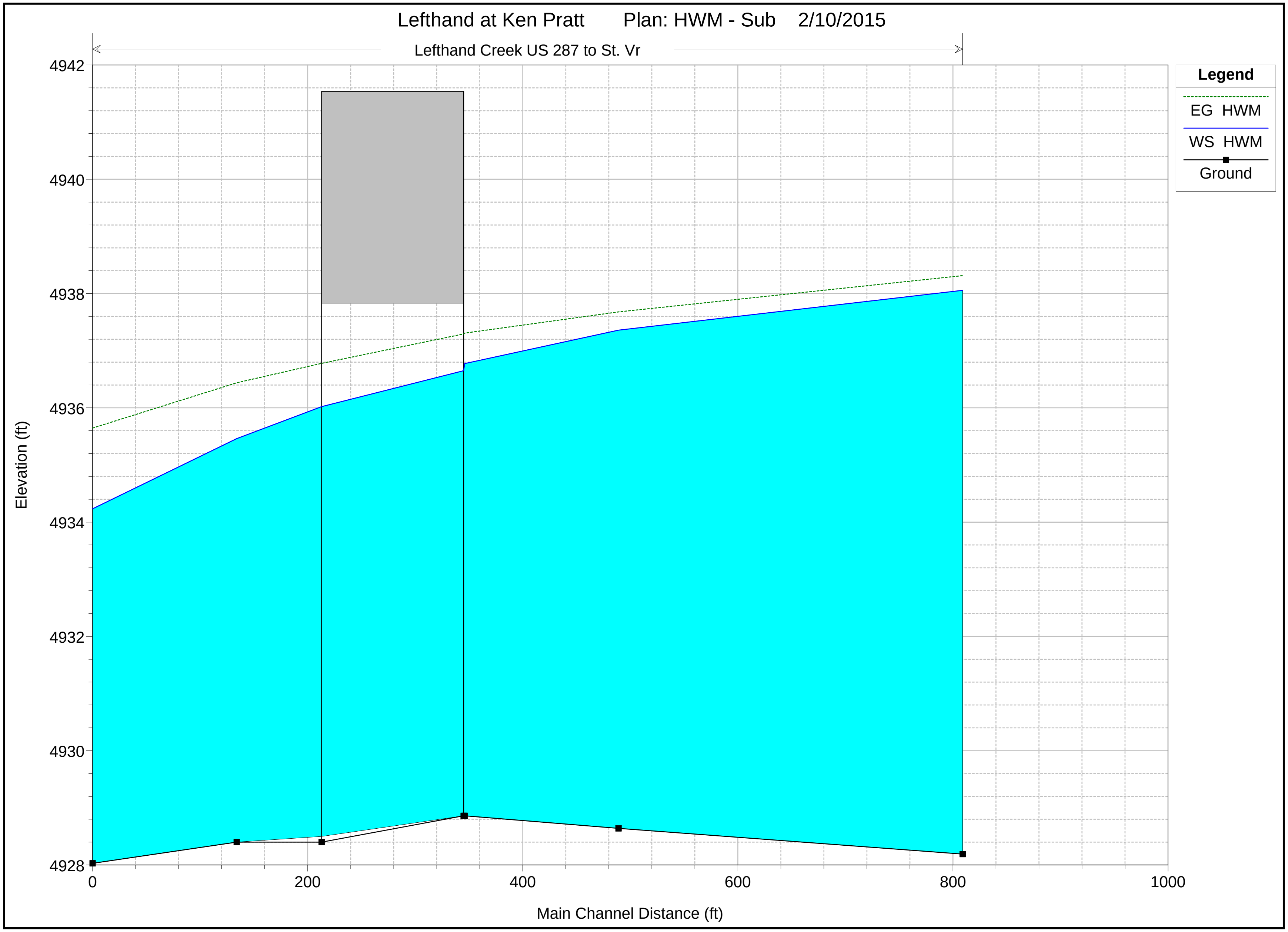


Left Hand at Ken Pratt

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

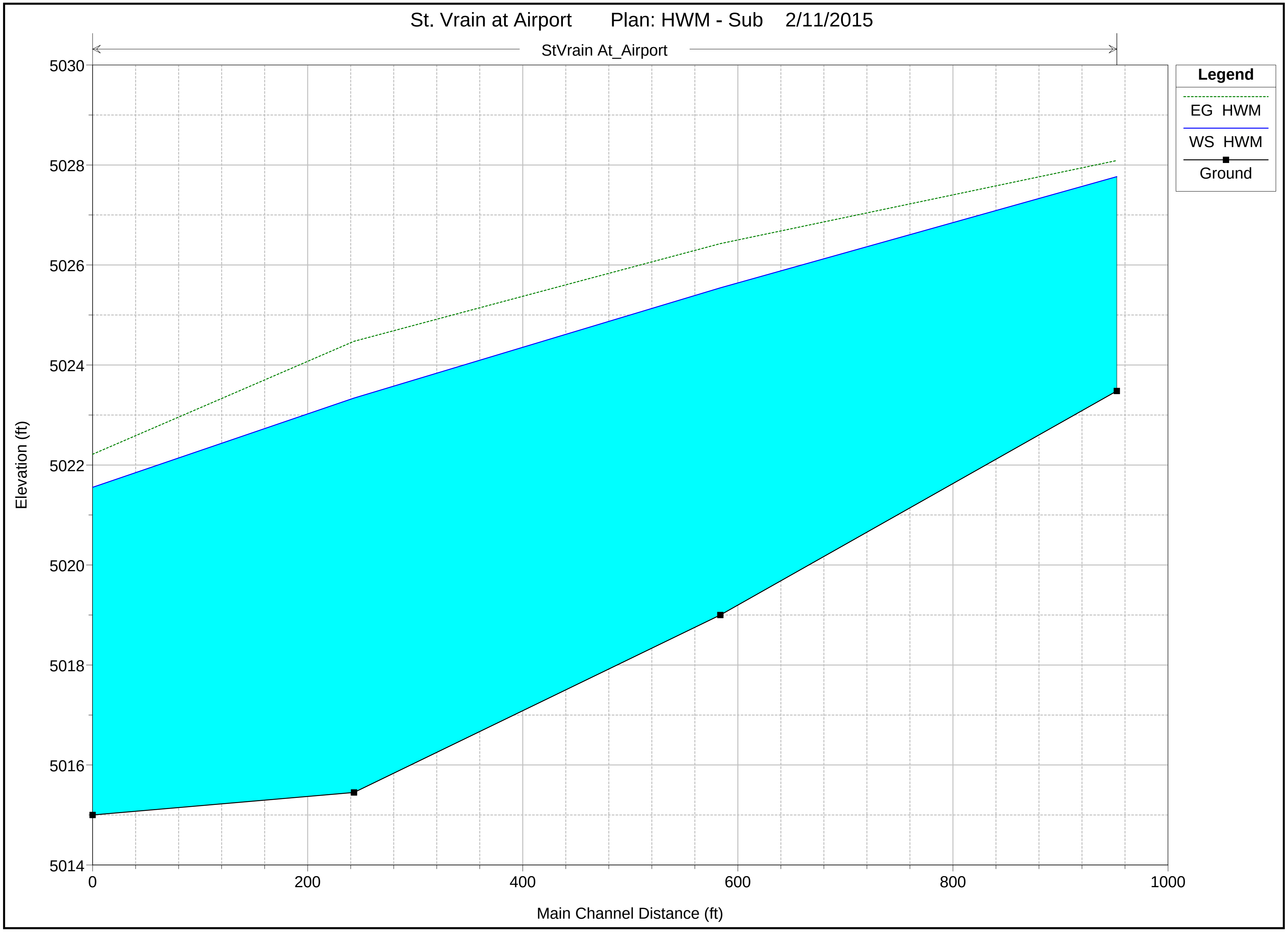
St. Vrain Creek
HEC-RAS Results

DRAFT

St. Vrain at Airport

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

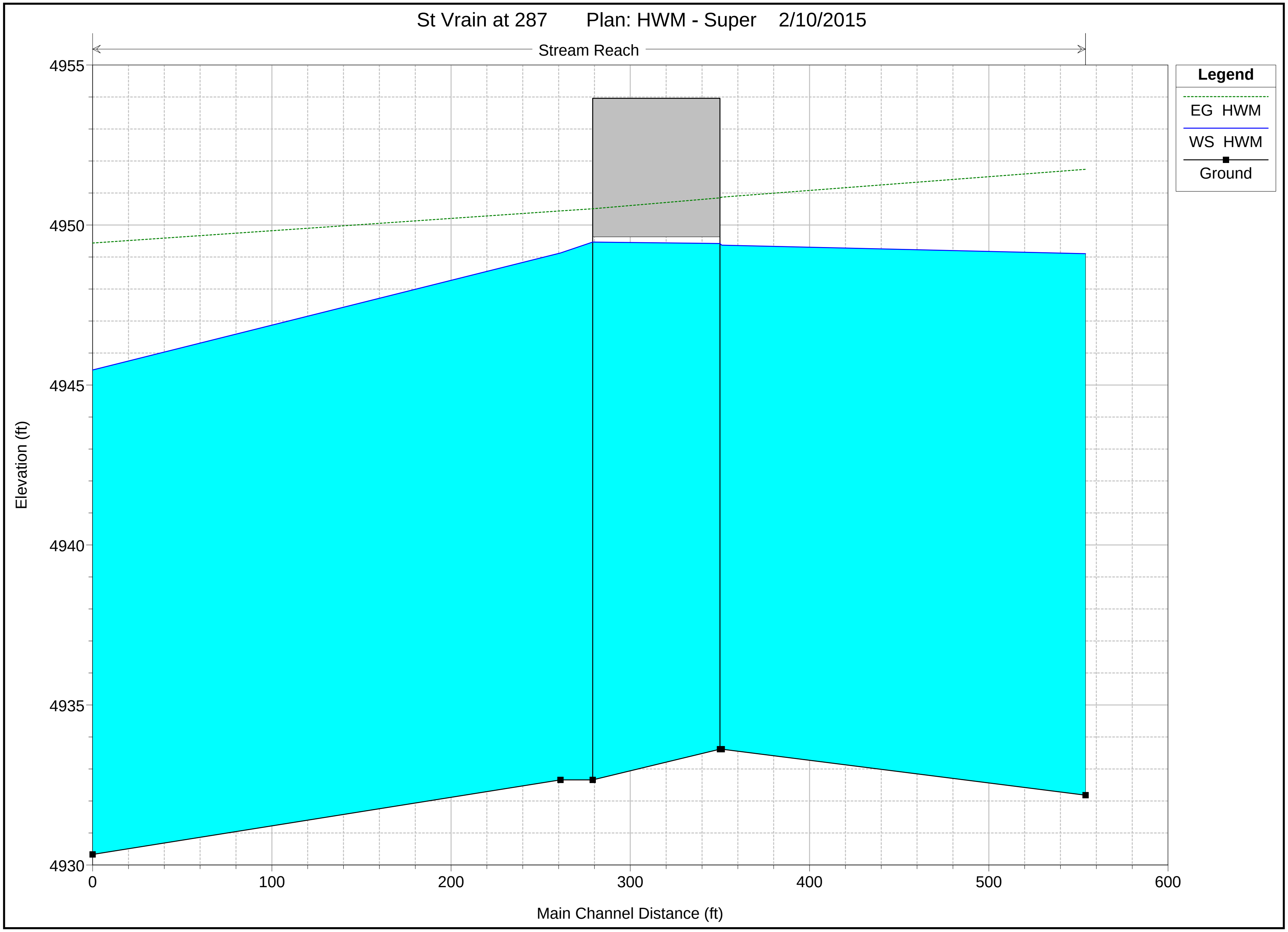
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
At_Airport	1486.316	HWM	14000.00	5023.48	5027.77	5026.83	5028.09	0.003893	6.35	4226.68	2147.94	0.58
At_Airport	1117.65	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 Plan: HWM - Sub 2/11/2015
StVrain At_Airport
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
5030
5028
5026
5024
5022
5020
5018
5016
5014
0
200
400
600
800
1000
Main Channel Distance (ft)

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

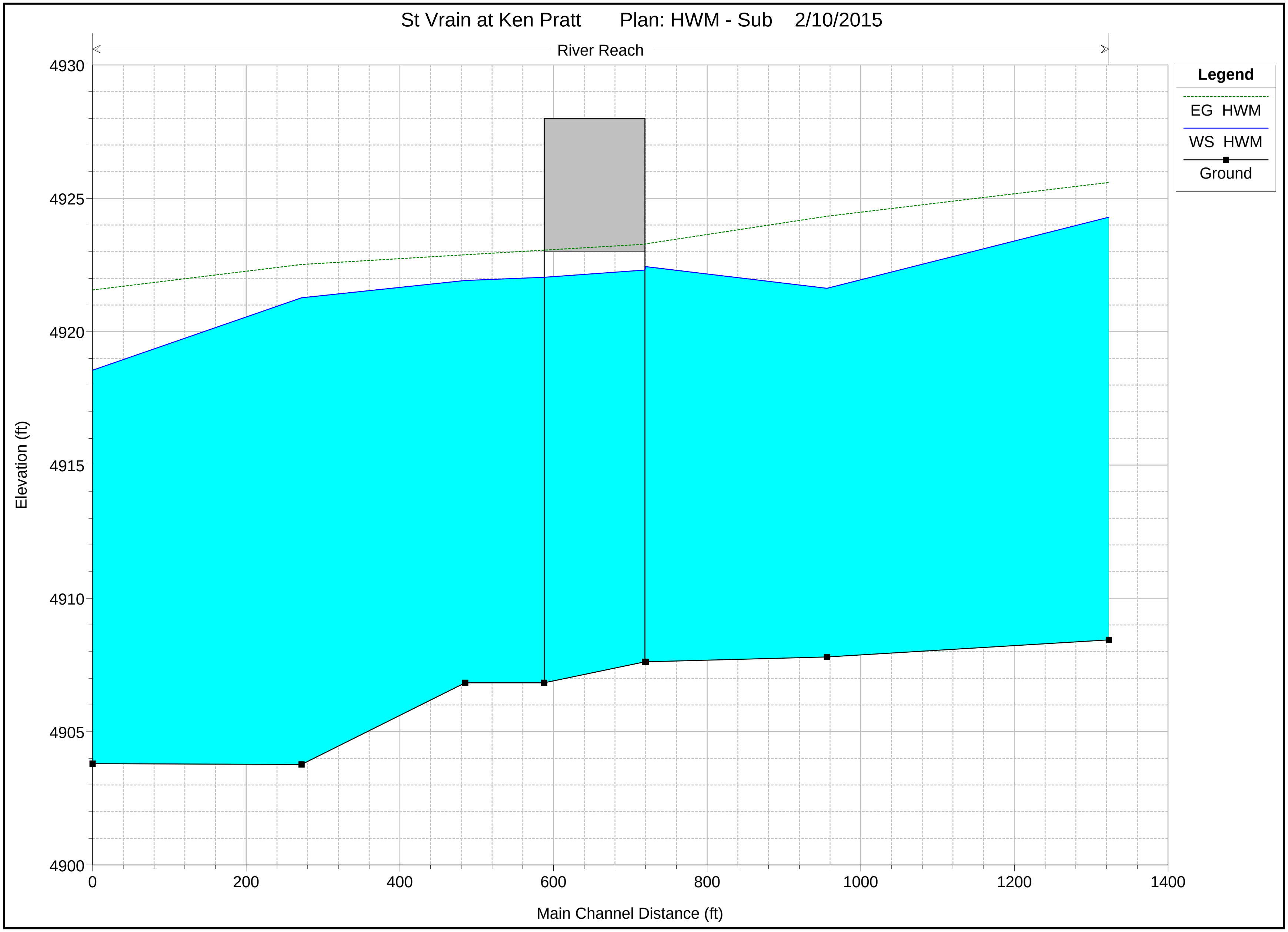


St. Vrain at Ken Pratt

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

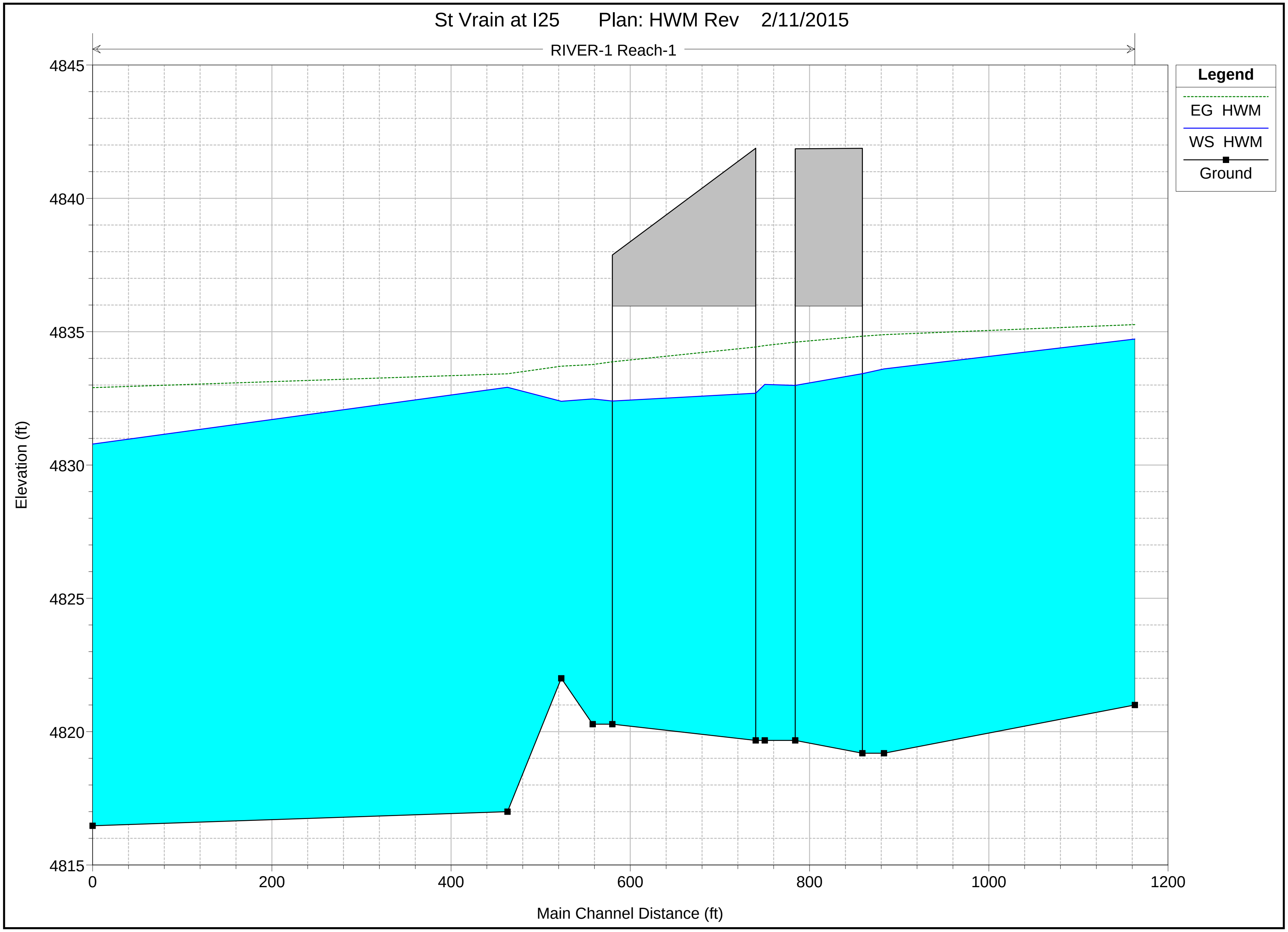


St. Vrain at I-25

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

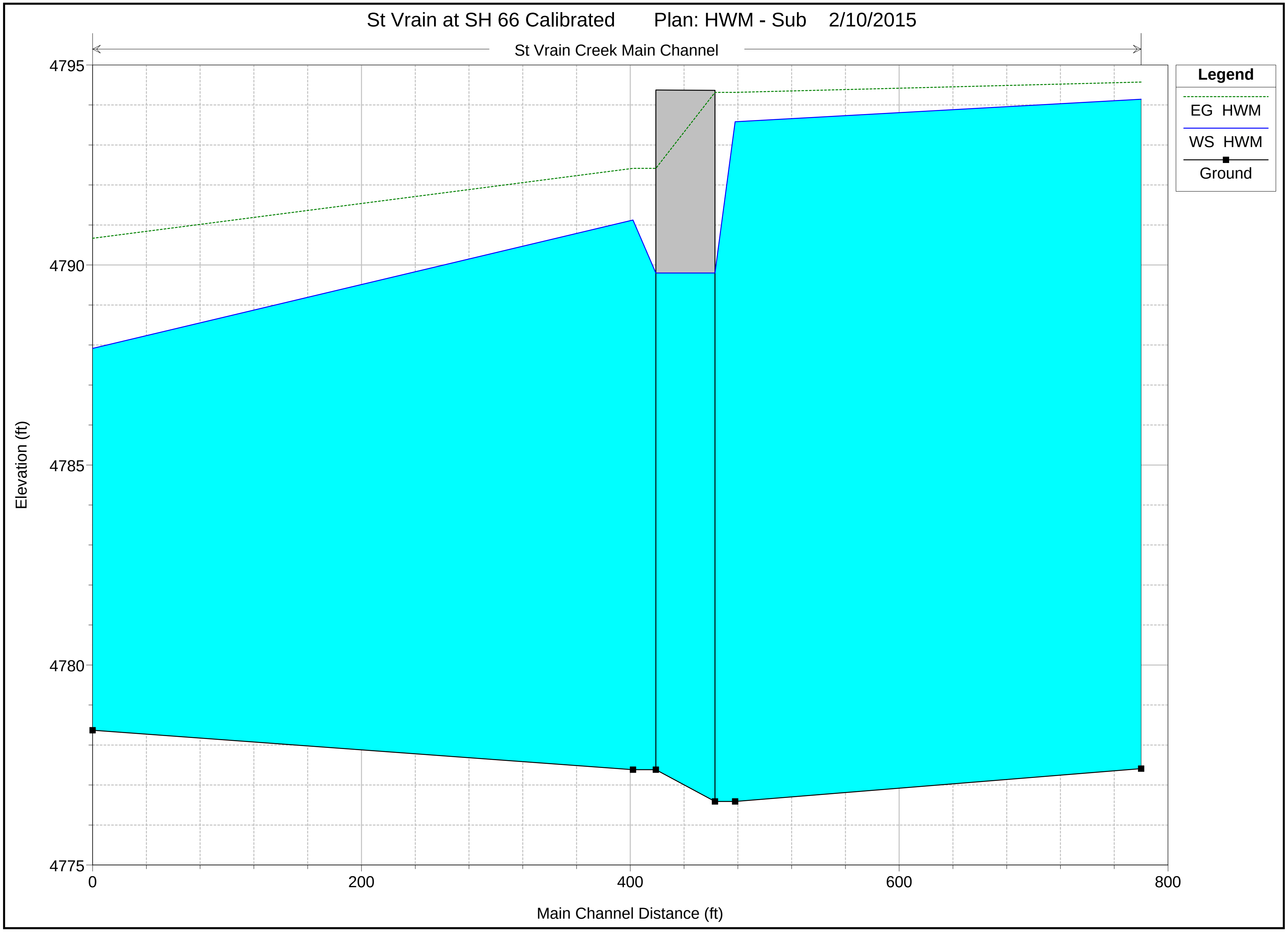
St. Vrain at I-25



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

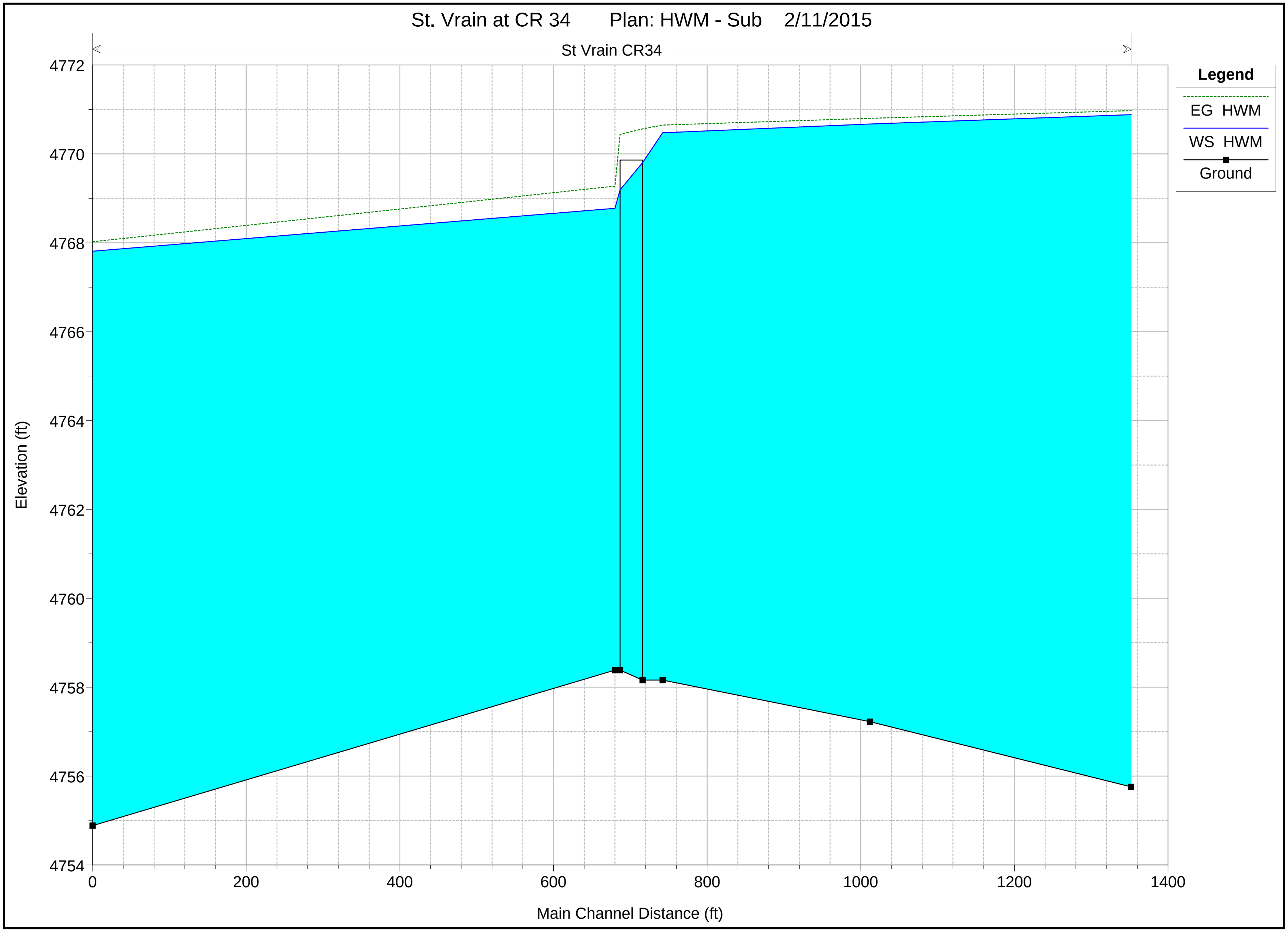
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main Channel	1780	HWM	23000.00	4777.41	4794.14	4786.83	4794.57	0.000586	5.82	4790.88	400.71	0.26
Main Channel	1478	HWM	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



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 Plan: HWM - Sub 2/11/2015
St Vrain CR34
Legend
EG HWM
WS HWM
Ground
Elevation (ft)
4772
4770
4768
4766
4764
4762
4760
4758
4756
4754
0
200
400
600
800
1000
1200
1400
Main Channel Distance (ft)

Appendix B

Flood Frequency Analysis at Stream Flow Gages

Bulletin 17B Frequency Analysis

Big Thompson River at Loveland near St. Louis Ave.

USGS Gage 06741510

CDWR Gage BIGLOVCO

1948 – 2013 (66 year period)

44 annual peaks recorded

Record extended to 99 years based on La Salle Gage

Bulletin 17B Frequency Analysis
24 Apr 2015 11:38 AM

--- Input Data ---

Analysis Name: 0674150 BigT Loveland Ext Hist
Description: Copy of Big Thompson Gage at Loveland 0674150 (+ 06741500) +2013 Flow of 22,000 cfs

Data Set Name: BT RIVER-LOVELAND 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: /BIG THOMPSON RIVER/LOVELAND, CO./FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\0674150_BigT_Loveland_Ext_Hist\0674150_BigT_Loveland_Ext_Hist.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\0674150_BigT_Loveland_Ext_Hist\0674150_BigT_Loveland_Ext_Hist.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: 19497.5

Use Historic Data
Historic Period Start Year: 1915
Historic Period End Year: 2013

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

<< Plotting Positions >>
BT RIVER-LOVELAND 2013

Events Analyzed				Ordered Events			
Day	Mon	Year	FLOW CFS	Rank	Water Year	FLOW CFS	Weibull Plot Pos
30	May	1948	4,380.0	1	2013	22,000.0*	2.22
04	Jun	1949	7,750.0	2	1949	7,750.0	4.44
18	Jun	1950	218.0	3	1980	6,970.0	6.67
04	Aug	1951	2,170.0	4	1976	6,000.0	8.89
08	Jun	1952	1,110.0	5	1999	4,960.0	11.11
14	Jun	1953	382.0	6	1948	4,380.0	13.33
20	Jul	1954	540.0	7	1995	3,780.0	15.56
23	Jul	1955	1,050.0	8	1994	2,710.0	17.78
31	Jul	1976	6,000.0	9	1982	2,270.0	20.00
19	Aug	1979	248.0	10	1983	2,240.0	22.22
30	Apr	1980	6,970.0	11	1951	2,170.0	24.44
08	Aug	1981	349.0	12	2010	1,800.0	26.67
14	Sep	1982	2,270.0	13	1997	1,790.0	28.89
12	Jun	1983	2,240.0	14	2012	1,600.0	31.11
16	May	1984	985.0	15	1952	1,110.0	33.33
18	Jun	1985	542.0	16	1989	1,070.0	35.56
06	Jul	1986	707.0	17	1955	1,050.0	37.78
09	Jun	1987	422.0	18	1984	985.0	40.00
05	Jun	1988	292.0	19	1991	873.0	42.22

03 Jun 1989	1,070.0	20	2005	780.0	44.44
29 May 1990	689.0	21	1986	707.0	46.67
02 Jun 1991	873.0	22	1990	689.0	48.89
07 Aug 1992	274.0	23	2003	679.0	51.11
24 May 1993	446.0	24	2011	627.0	53.33
11 Aug 1994	2,710.0	25	2004	577.0	55.56
30 May 1995	3,780.0	26	1985	542.0	57.78
23 Jun 1996	368.0	27	1954	540.0	60.00
10 Jun 1997	1,790.0	28	2001	468.0	62.22
11 Jul 1998	349.0	29	1993	446.0	64.44
30 Apr 1999	4,960.0	30	1987	422.0	66.67
25 Sep 2000	339.0	31	2009	400.0	68.89
24 Oct 2000	468.0	32	1953	382.0	71.11
10 Jun 2002	238.0	33	1996	368.0	73.33
17 Jun 2003	679.0	34	2006	357.0	75.56
24 Jul 2004	577.0	35	1998	349.0	77.78
26 Jun 2005	780.0	36	1981	349.0	80.00
25 Aug 2006	357.0	37	2000	339.0	82.22
27 Jul 2007	284.0	38	1988	292.0	84.44
05 Jun 2008	240.0	39	2007	284.0	86.67
22 May 2009	400.0	40	1992	274.0	88.89
12 Jun 2010	1,800.0	41	1979	248.0	91.11
12 Jul 2011	627.0	42	2008	240.0	93.33
07 Jul 2012	1,600.0	43	2002	238.0	95.56
12 Sep 2013	22,000.0	44	1950	218.0	97.78

* Outlier

<< Skew Weighting >>

Based on 44 events, mean-square error of station skew = 0.192
Mean-square error of regional skew = -?

<< Frequency Curve >>
BT RIVER-LOVELAND 2013

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
78,008.2	120,651.8	0.2	213,876.7	38,001.0
41,343.1	56,618.6	0.5	99,588.4	21,997.0
25,172.1	31,779.9	1.0	54,896.9	14,323.8
15,057.2	17,778.3	2.0	29,694.8	9,163.8
7,355.5	8,076.4	5.0	12,688.5	4,885.1
4,106.2	4,341.4	10.0	6,412.2	2,900.6
2,164.5	2,222.1	20.0	3,084.4	1,607.3
769.8	769.8	50.0	1,019.2	575.3
345.7	341.0	80.0	468.3	239.8
247.6	242.1	90.0	343.9	163.9
195.6	189.4	95.0	277.5	124.8
137.3	130.8	99.0	202.0	82.5

<< Systematic Statistics >>
BT RIVER-LOVELAND 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.958	Historic Events	0
Standard Dev	0.490	High Outliers	0
Station Skew	0.882	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.882	Systematic Events	44

--- End of Preliminary Results ---

```
-----------------------
<< High Outlier Test >>
-----------------------
 Based on 44 events, 10 percent outlier test deviate K(N) = 2.719
                     Computed high outlier test value = 19,497.45

   1 high outlier(s) identified above input threshold of 19,497.5

   * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
   * 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 >>
BT RIVER-LOVELAND 2013
```

Log Transform: FLOW, CFS		Number of Events	
Mean	2.940	Historic Events	0
Standard Dev	0.463	High Outliers	1
Station Skew	0.779	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.882	Systematic Events	44
		Historic Period	99

```
----------------------
<< Low Outlier Test >>
----------------------
 Based on 99 events, 10 percent outlier test deviate K(N) = 3.014
                           Computed low outlier test value = 34.9

             0 low outlier(s) identified below test value of 34.9




--- Final Results ---

<< Plotting Positions >>
BT RIVER-LOVELAND 2013
```

Events Analyzed Day Mon Year	FLOW CFS	Rank	Water Year	FLOW CFS	Weibull Plot Pos
30 May 1948	4,380.0	1	2013	22,000.0*	1.00
04 Jun 1949	7,750.0	2	1949	7,750.0	2.64
18 Jun 1950	218.0	3	1980	6,970.0	4.92
04 Aug 1951	2,170.0	4	1976	6,000.0	7.20
08 Jun 1952	1,110.0	5	1999	4,960.0	9.48
14 Jun 1953	382.0	6	1948	4,380.0	11.76
20 Jul 1954	540.0	7	1995	3,780.0	14.03
23 Jul 1955	1,050.0	8	1994	2,710.0	16.31
31 Jul 1976	6,000.0	9	1982	2,270.0	18.59
19 Aug 1979	248.0	10	1983	2,240.0	20.87
30 Apr 1980	6,970.0	11	1951	2,170.0	23.15
08 Aug 1981	349.0	12	2010	1,800.0	25.43
14 Sep 1982	2,270.0	13	1997	1,790.0	27.71
12 Jun 1983	2,240.0	14	2012	1,600.0	29.99
16 May 1984	985.0	15	1952	1,110.0	32.27
18 Jun 1985	542.0	16	1989	1,070.0	34.55

06 Jul 1986	707.0	17	1955	1,050.0	36.83
09 Jun 1987	422.0	18	1984	985.0	39.10
05 Jun 1988	292.0	19	1991	873.0	41.38
03 Jun 1989	1,070.0	20	2005	780.0	43.66
29 May 1990	689.0	21	1986	707.0	45.94
02 Jun 1991	873.0	22	1990	689.0	48.22
07 Aug 1992	274.0	23	2003	679.0	50.50
24 May 1993	446.0	24	2011	627.0	52.78
11 Aug 1994	2,710.0	25	2004	577.0	55.06
30 May 1995	3,780.0	26	1985	542.0	57.34
23 Jun 1996	368.0	27	1954	540.0	59.62
10 Jun 1997	1,790.0	28	2001	468.0	61.90
11 Jul 1998	349.0	29	1993	446.0	64.17
30 Apr 1999	4,960.0	30	1987	422.0	66.45
25 Sep 2000	339.0	31	2009	400.0	68.73
24 Oct 2000	468.0	32	1953	382.0	71.01
10 Jun 2002	238.0	33	1996	368.0	73.29
17 Jun 2003	679.0	34	2006	357.0	75.57
24 Jul 2004	577.0	35	1998	349.0	77.85
26 Jun 2005	780.0	36	1981	349.0	80.13
25 Aug 2006	357.0	37	2000	339.0	82.41
27 Jul 2007	284.0	38	1988	292.0	84.69
05 Jun 2008	240.0	39	2007	284.0	86.97
22 May 2009	400.0	40	1992	274.0	89.24
12 Jun 2010	1,800.0	41	1979	248.0	91.52
12 Jul 2011	627.0	42	2008	240.0	93.80
07 Jul 2012	1,600.0	43	2002	238.0	96.08
12 Sep 2013	22,000.0	44	1950	218.0	98.36

Note: Plotting positions based on historic period (H) = 99
Number of historic events plus high outliers (Z) = 1
Weighting factor for systematic events (W) = 2.2791

* Outlier

<< Skew Weighting >>

Based on 99 events, mean-square error of station skew = 0.1
Mean-square error of regional skew = -?

<< Frequency Curve >>
BT RIVER-LOVELAND 2013

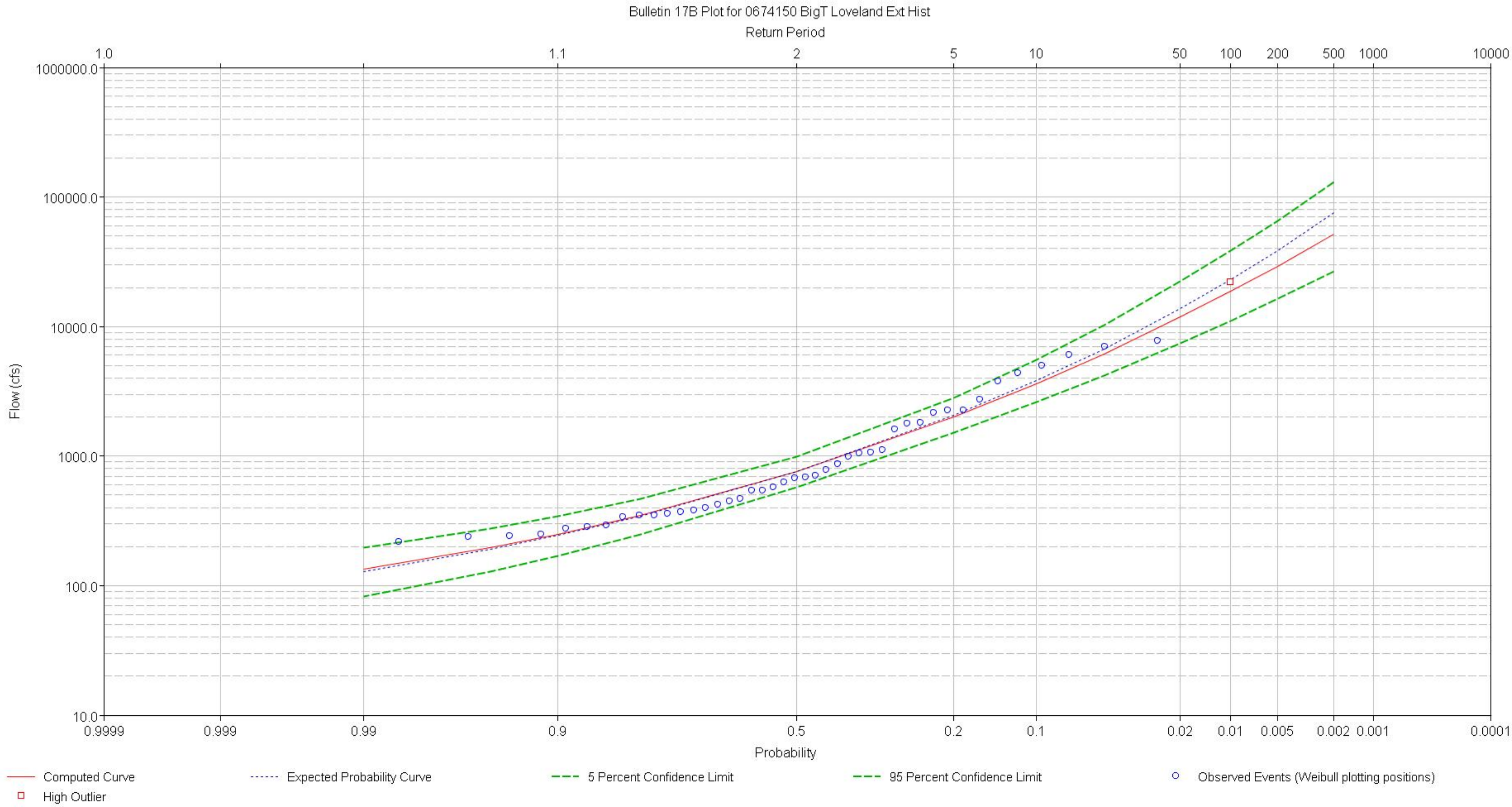
Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
51,518.3	75,911.0	0.2	130,155.4	26,582.6
29,237.4	38,706.1	0.5	65,832.2	16,315.2
18,742.5	23,096.7	1.0	38,615.8	11,104.4
11,800.8	13,704.5	2.0	22,215.9	7,425.4
6,165.8	6,712.7	5.0	10,286.1	4,197.3
3,619.7	3,808.6	10.0	5,514.7	2,606.3
2,004.9	2,054.0	20.0	2,806.6	1,512.0
758.7	758.7	50.0	989.5	576.6
349.0	344.2	80.0	465.0	246.8
249.8	244.1	90.0	341.4	168.6
196.2	189.7	95.0	274.2	127.4
134.7	127.6	99.0	195.8	82.1

<< Adjusted Statistics >>
BT RIVER-LOVELAND 2013

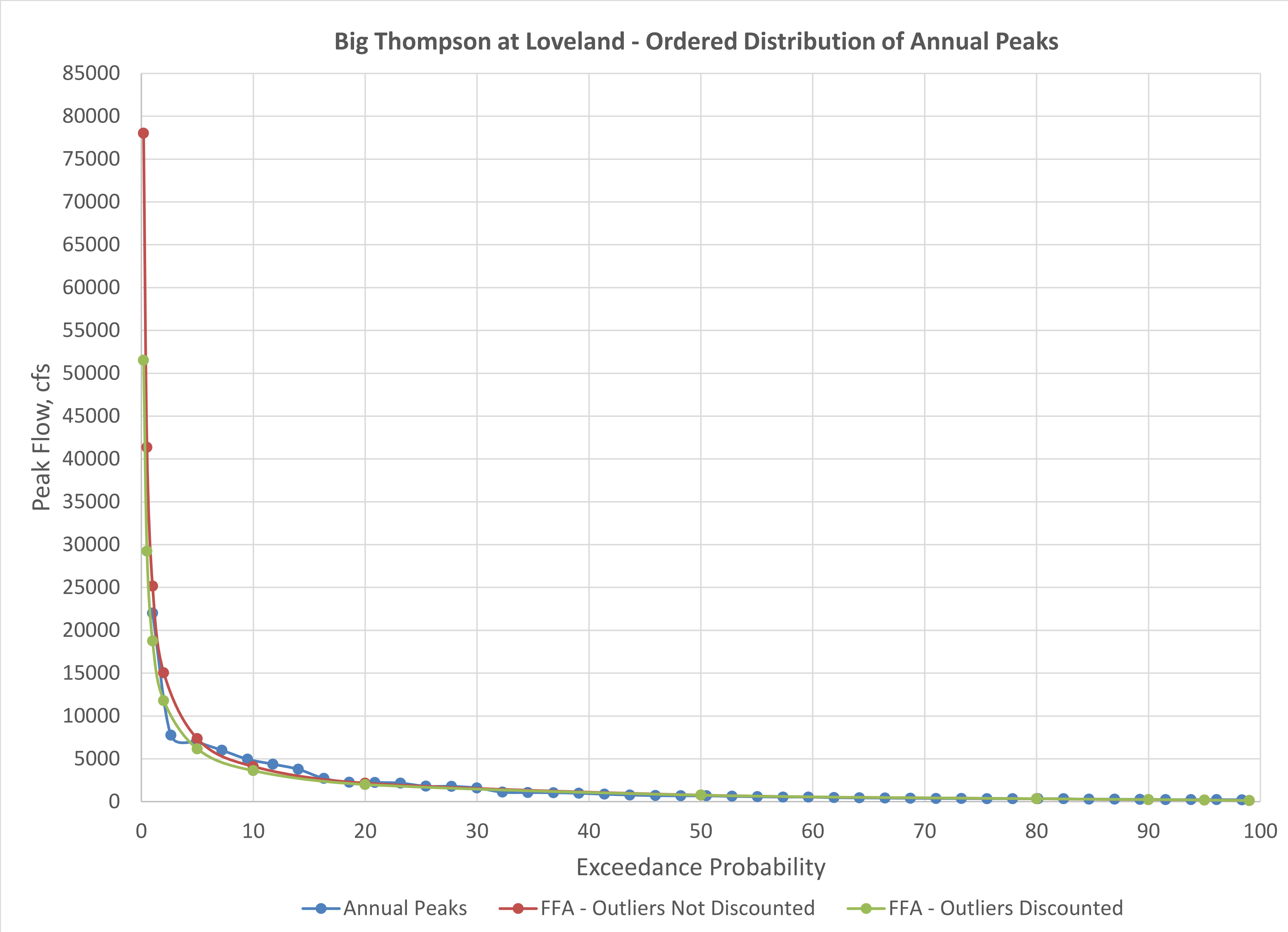
Log Transform: FLOW, CFS		Number of Events	
Mean	2.940	Historic Events	0
Standard Dev	0.463	High Outliers	1

Station Skew	0.779	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.779	Systematic Events	44
		Historic Period	99

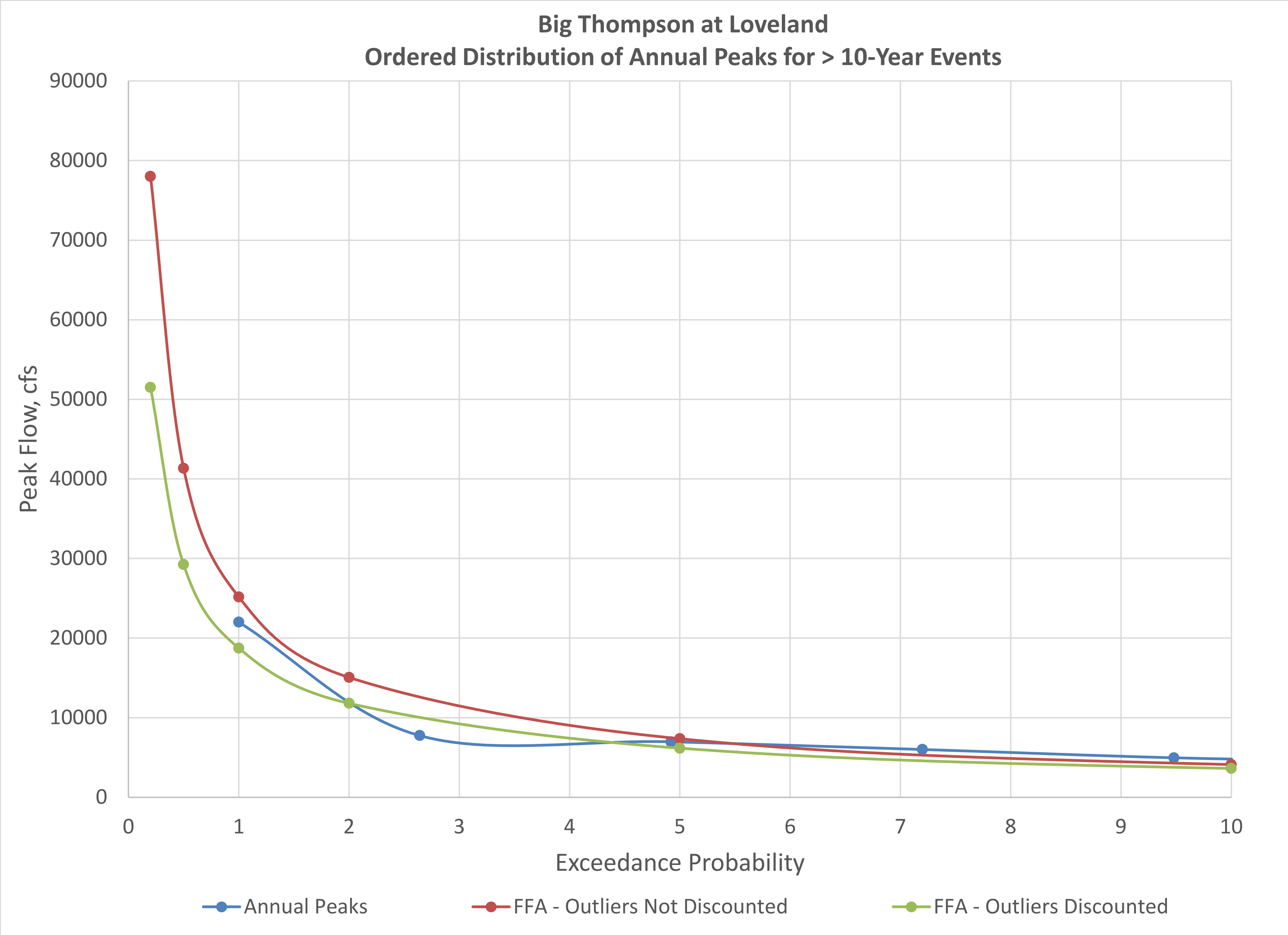
--- End of Analytical Frequency Curve ---

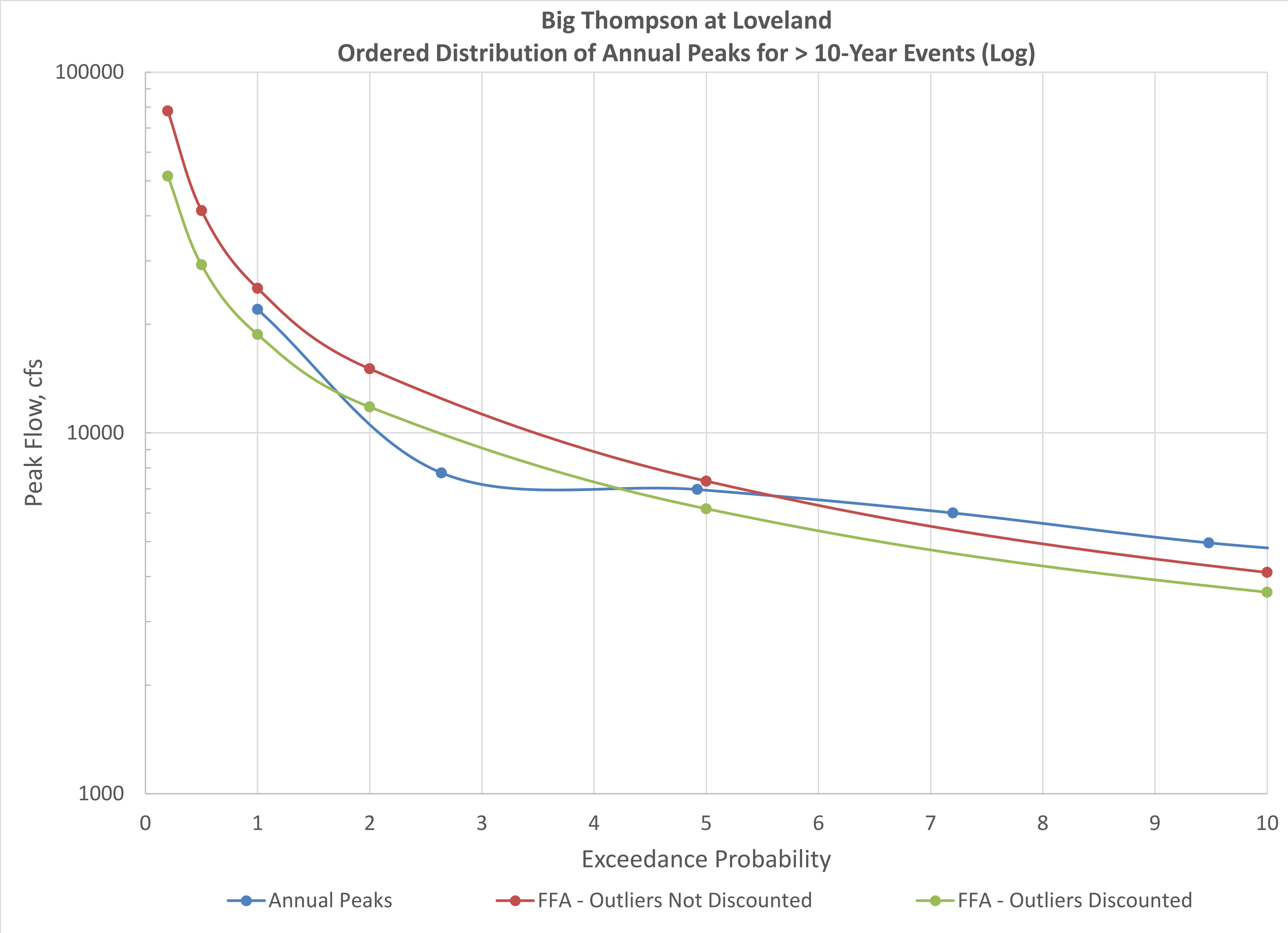


Bulletin 17B Plot for 0674150 BigT Loveland Ext Hist
Return Period
1.0
1.1
2
5
10
50
100
200
500
1000
10000
Flow (cfs)
1000000.0
100000.0
10000.0
1000.0
100.0
10.0
0.9999
0.999
0.99
0.9
0.5
0.2
0.1
0.02
0.01
0.005
0.002
0.001
0.0001
Probability
Computed Curve
Expected Probability Curve
5 Percent Confidence Limit
95 Percent Confidence Limit
Observed Events (Weibull plotting positions)
High Outlier



Big Thompson at Loveland - Ordered Distribution of Annual Peaks
Peak Flow, cfs
85000
80000
75000
70000
65000
60000
55000
50000
45000
40000
35000
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


Big Thompson at Loveland
Ordered Distribution of Annual Peaks for > 10-Year Events
Peak Flow, cfs
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
FFA - Outliers Not Discounted
FFA - Outliers Discounted



Big Thompson at Loveland
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

Big Thompson River at La Salle near Mouth

USGS Gage 06744000

CDWR Gage BIGLASCO

1915 – 2013 (99 year period)

78 annual peaks recorded

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

--- Input Data ---

Analysis Name: 0674400 BigT La Salle Nr Mouth
Description: Downloaded from USGS website. Station 06744000

Data Set Name: BT RIVER-MO, NR LA S 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: /BIG THOMPSON RIVER/MOUTH, NEAR LA SALLE, CO./FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/USGS/

Report File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\0674400_BigT_La_Salle_Nr_Mouth\0674400_BigT_La_Salle_Nr_Mouth.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\2BT_3StV_1LhC\2BT_3St.V_1Lt.H\Bulletin17bResults\0674400_BigT_La_Salle_Nr_Mouth\0674400_BigT_La_Salle_Nr_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: 16496.6

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 >>
BT RIVER-MO, NR LA S 2013

Events Analyzed		Ordered Events			
Day Mon Year	FLOW CFS	Rank	Water Year	FLOW CFS	Weibull Plot Pos
13 Jun 1915	667	1	2013	24,900*	1.27
12 Jun 1927	235	2	1995	6,710	2.53
11 May 1928	399	3	1980	6,220	3.80
07 Aug 1929	375	4	1951	6,100	5.06
15 Aug 1930	488	5	1999	5,760	6.33
24 Jun 1931	512	6	1949	4,440	7.59
29 Jul 1932	1,300	7	1938	3,000	8.86
27 May 1933	156	8	1969	2,900	10.13
15 Oct 1933	73	9	1997	2,620	11.39
12 Jun 1935	748	10	1957	2,460	12.66
12 Jul 1936	364	11	1976	2,440	13.92
04 Sep 1937	935	12	1961	2,050	15.19
03 Sep 1938	3,000	13	1965	1,960	16.46
29 Jun 1939	118	14	1979	1,790	17.72
03 Jul 1940	124	15	1958	1,760	18.99
22 Jun 1941	581	16	1971	1,700	20.25
03 May 1942	578	17	2010	1,640	21.52
03 Jun 1943	878	18	1973	1,610	22.78

14 May 1944	306	19	1947	1,600	24.05
24 Jun 1945	301	20	1977	1,490	25.32
19 Jul 1946	758	21	1970	1,440	26.58
22 Jun 1947	1,600	22	1978	1,300	27.85
30 May 1948	435	23	1932	1,300	29.11
05 Jun 1949	4,440	24	2005	1,000	30.38
14 Nov 1949	76	25	1952	960	31.65
03 Aug 1951	6,100	26	1994	940	32.91
05 Jun 1952	960	27	1937	935	34.18
14 Jun 1953	253	28	1972	906	35.44
30 Jun 1954	452	29	1963	904	36.71
07 Aug 1955	861	30	1975	902	37.97
27 May 1956	396	31	1943	878	39.24
09 May 1957	2,460	32	1955	861	40.51
16 May 1958	1,760	33	1959	781	41.77
25 May 1959	781	34	1991	776	43.04
03 Jul 1960	332	35	1946	758	44.30
04 Jun 1961	2,050	36	1935	748	45.57
28 Jul 1962	592	37	2004	691	46.84
17 Jun 1963	904	38	1974	667	48.10
29 May 1964	420	39	1915	667	49.37
18 Jun 1965	1,960	40	1962	592	50.63
20 Jul 1966	300	41	1941	581	51.90
30 May 1967	570	42	1942	578	53.16
24 May 1968	504	43	1967	570	54.43
08 May 1969	2,900	44	2012	516	55.70
12 Jun 1970	1,440	45	1931	512	56.96
26 Apr 1971	1,700	46	1968	504	58.23
06 Jun 1972	906	47	1930	488	59.49
07 May 1973	1,610	48	1981	453	60.76
09 Jun 1974	667	49	1954	452	62.03
18 Jun 1975	902	50	1948	435	63.29
01 Aug 1976	2,440	51	1964	420	64.56
25 Jul 1977	1,490	52	2011	413	65.82
18 May 1978	1,300	53	1998	408	67.09
16 Jun 1979	1,790	54	1992	405	68.35
01 May 1980	6,220	55	1928	399	69.62
28 May 1981	453	56	1956	396	70.89
02 Jun 1991	776	57	1929	375	72.15
24 Aug 1992	405	58	1936	364	73.42
23 May 1993	351	59	1993	351	74.68
11 Aug 1994	940	60	1996	348	75.95
31 May 1995	6,710	61	2003	342	77.22
23 Jun 1996	348	62	2002	333	78.48
13 Jun 1997	2,620	63	1960	332	79.75
22 May 1998	408	64	2009	321	81.01
01 May 1999	5,760	65	2000	321	82.28
17 May 2000	321	66	2006	318	83.54
24 Oct 2001	333	67	1944	306	84.81
24 May 2002	181	68	1945	301	86.08
18 Jun 2003	342	69	1966	300	87.34
24 Jul 2004	691	70	2008	294	88.61
04 Jun 2005	1,000	71	1953	253	89.87
29 Oct 2005	318	72	1927	235	91.14
16 Aug 2008	294	73	2002	181	92.41
18 Apr 2009	321	74	1933	156	93.67
13 Jun 2010	1,640	75	1940	124	94.94
11 May 2011	413	76	1939	118	96.20
08 Jul 2012	516	77	1950	76	97.47
13 Sep 2013	24,900	78	1934	73	98.73

* Outlier

<< Skew Weighting >>

Based on 78 events, mean-square error of station skew = 0.103
Mean-square error of regional skew = -?

<< Frequency Curve >>
BT RIVER-MO, NR LA S 2013

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
33,299	40,123	0.2	61,955	20,517
20,156	23,106	0.5	34,914	13,102
13,542	15,011	1.0	22,188	9,174
8,918	9,609	2.0	13,804	6,301
4,924	5,143	5.0	7,057	3,678
3,000	3,080	10.0	4,054	2,334
1,713	1,735	20.0	2,190	1,381
659	659	50.0	804	539
294	291	80.0	366	229
204	201	90.0	259	153
154	151	95.0	201	112
98	94	99.0	132	67

<< Systematic Statistics >>
BT RIVER-MO, NR LA S 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.864	Historic Events	0
Standard Dev	0.462	High Outliers	0
Station Skew	0.586	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.586	Systematic Events	78

--- End of Preliminary Results ---

<< High Outlier Test >>

Based on 78 events, 10 percent outlier test deviate K(N) = 2.931
Computed high outlier test value = 16,496.6

1 high outlier(s) identified above input threshold of 16,496.6

```
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
* 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 >>
BT RIVER-MO, NR LA S 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.860	Historic Events	0
Standard Dev	0.455	High Outliers	1
Station Skew	0.530	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.586	Systematic Events	78
		Historic Period	99

<< Low Outlier Test >>

Based on 99 events, 10 percent outlier test deviate K(N) = 3.014
Computed low outlier test value = 30.9

0 low outlier(s) identified below test value of 30.9

--- Final Results ---

<< Plotting Positions >>
BT RIVER-MO, NR LA S 2013

Events Analyzed				Ordered Events		
Day Mon Year	FLOW CFS	Rank	Water Year	FLOW CFS	Weibull Plot Pos
13 Jun 1915	667	1	2013	24,900*	1.00
12 Jun 1927	235	2	1995	6,710	2.14
11 May 1928	399	3	1980	6,220	3.41
07 Aug 1929	375	4	1951	6,100	4.68
15 Aug 1930	488	5	1999	5,760	5.95
24 Jun 1931	512	6	1949	4,440	7.23
29 Jul 1932	1,300	7	1938	3,000	8.50
27 May 1933	156	8	1969	2,900	9.77
15 Oct 1933	73	9	1997	2,620	11.05
12 Jun 1935	748	10	1957	2,460	12.32
12 Jul 1936	364	11	1976	2,440	13.59
04 Sep 1937	935	12	1961	2,050	14.86
03 Sep 1938	3,000	13	1965	1,960	16.14
29 Jun 1939	118	14	1979	1,790	17.41
03 Jul 1940	124	15	1958	1,760	18.68
22 Jun 1941	581	16	1971	1,700	19.95
03 May 1942	578	17	2010	1,640	21.23
03 Jun 1943	878	18	1973	1,610	22.50
14 May 1944	306	19	1947	1,600	23.77
24 Jun 1945	301	20	1977	1,490	25.05
19 Jul 1946	758	21	1970	1,440	26.32
22 Jun 1947	1,600	22	1978	1,300	27.59
30 May 1948	435	23	1932	1,300	28.86
05 Jun 1949	4,440	24	2005	1,000	30.14
14 Nov 1949	76	25	1952	960	31.41
03 Aug 1951	6,100	26	1994	940	32.68
05 Jun 1952	960	27	1937	935	33.95
14 Jun 1953	253	28	1972	906	35.23
30 Jun 1954	452	29	1963	904	36.50
07 Aug 1955	861	30	1975	902	37.77
27 May 1956	396	31	1943	878	39.05
09 May 1957	2,460	32	1955	861	40.32
16 May 1958	1,760	33	1959	781	41.59
25 May 1959	781	34	1991	776	42.86
03 Jul 1960	332	35	1946	758	44.14
04 Jun 1961	2,050	36	1935	748	45.41
28 Jul 1962	592	37	2004	691	46.68
17 Jun 1963	904	38	1974	667	47.95
29 May 1964	420	39	1915	667	49.23
18 Jun 1965	1,960	40	1962	592	50.50
20 Jul 1966	300	41	1941	581	51.77
30 May 1967	570	42	1942	578	53.05
24 May 1968	504	43	1967	570	54.32
08 May 1969	2,900	44	2012	516	55.59
12 Jun 1970	1,440	45	1931	512	56.86
26 Apr 1971	1,700	46	1968	504	58.14
06 Jun 1972	906	47	1930	488	59.41
07 May 1973	1,610	48	1981	453	60.68
09 Jun 1974	667	49	1954	452	61.95
18 Jun 1975	902	50	1948	435	63.23
01 Aug 1976	2,440	51	1964	420	64.50
25 Jul 1977	1,490	52	2011	413	65.77
18 May 1978	1,300	53	1998	408	67.05
16 Jun 1979	1,790	54	1992	405	68.32
01 May 1980	6,220	55	1928	399	69.59

28 May 1981	453	56	1956	396	70.86
02 Jun 1991	776	57	1929	375	72.14
24 Aug 1992	405	58	1936	364	73.41
23 May 1993	351	59	1993	351	74.68
11 Aug 1994	940	60	1996	348	75.95
31 May 1995	6,710	61	2003	342	77.23
23 Jun 1996	348	62	2002	333	78.50
13 Jun 1997	2,620	63	1960	332	79.77
22 May 1998	408	64	2009	321	81.05
01 May 1999	5,760	65	2000	321	82.32
17 May 2000	321	66	2006	318	83.59
24 Oct 2001	333	67	1944	306	84.86
24 May 2002	181	68	1945	301	86.14
18 Jun 2003	342	69	1966	300	87.41
24 Jul 2004	691	70	2008	294	88.68
04 Jun 2005	1,000	71	1953	253	89.95
29 Oct 2005	318	72	1927	235	91.23
16 Aug 2008	294	73	2002	181	92.50
18 Apr 2009	321	74	1933	156	93.77
13 Jun 2010	1,640	75	1940	124	95.05
11 May 2011	413	76	1939	118	96.32
08 Jul 2012	516	77	1950	76	97.59
13 Sep 2013	24,900	78	1934	73	98.86

Note: Plotting positions based on historic period (H) = 99
Number of historic events plus high outliers (Z) = 1
Weighting factor for systematic events (W) = 1.2727

* Outlier

<< Skew Weighting >>

Based on 99 events, mean-square error of station skew = 0.082
Mean-square error of regional skew = -?

<< Frequency Curve >>
BT RIVER-MO, NR LA S 2013

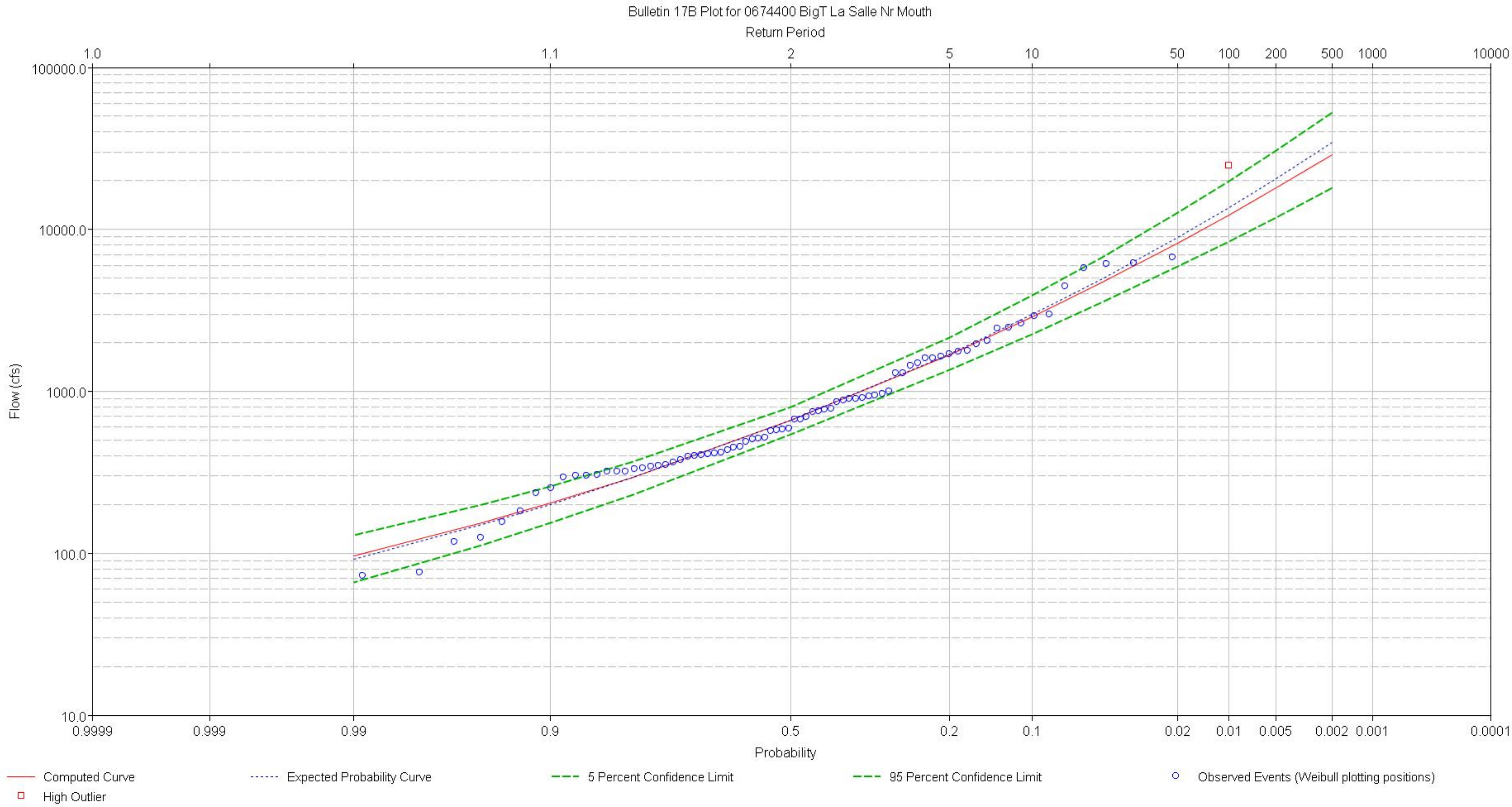
Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
28,963	34,556	0.2	52,832	18,117
17,980	20,470	0.5	30,650	11,832
12,310	13,579	1.0	19,905	8,425
8,260	8,870	2.0	12,651	5,883
4,672	4,871	5.0	6,648	3,510
2,897	2,972	10.0	3,896	2,264
1,683	1,704	20.0	2,144	1,361
660	660	50.0	803	541
295	293	80.0	366	231
204	201	90.0	259	153
154	150	95.0	199	112
96	92	99.0	129	66

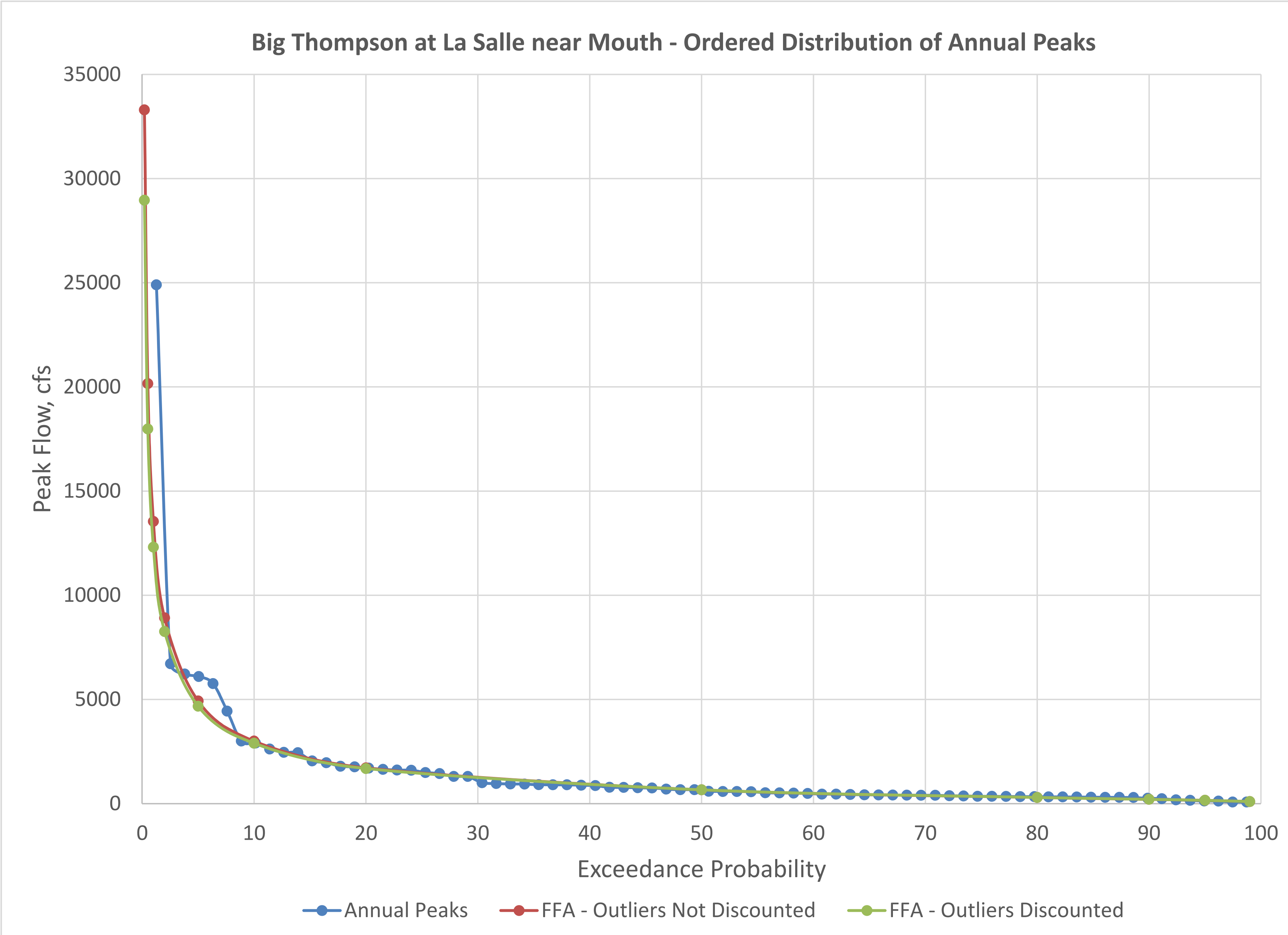
<< Adjusted Statistics >>
BT RIVER-MO, NR LA S 2013

Log Transform: FLOW, CFS		Number of Events	
Mean	2.860	Historic Events	0
Standard Dev	0.455	High Outliers	1
Station Skew	0.530	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	0.530	Systematic Events	78
		Historic Period	99

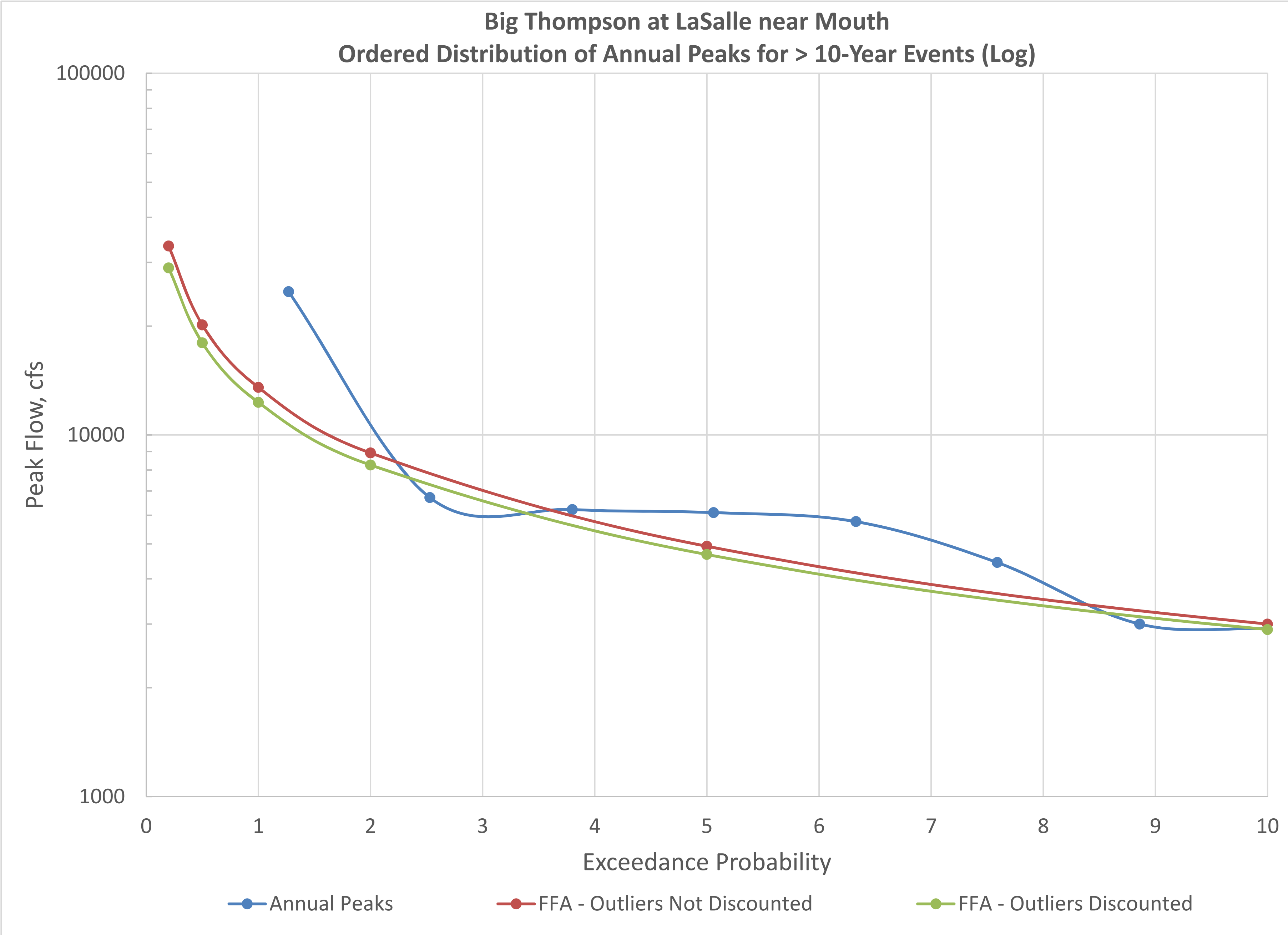
|------------------------------|-------------------------------|

--- End of Analytical Frequency Curve ---


Bulletin 17B Plot for 0674400 BigT La Salle Nr Mouth
Return Period
1.0
1.1
2
5
10
50
100
200
500
1000
10000
Flow (cfs)
100000.0
10000.0
1000.0
100.0
10.0
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


Big Thompson at La Salle near Mouth - Ordered Distribution of Annual Peaks
Peak Flow, cfs
35000
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

Big Thompson at LaSalle near Mouth
Ordered Distribution of Annual Peaks for > 10-Year Events
Peak Flow, cfs
35000
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


Big Thompson at LaSalle near 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

Appendix C

Rainfall Depth-Area Reduction Factors

PO Box 175
Monument, CO 80132
(719) 488-4311
http://appliedweatherassociates.com

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).

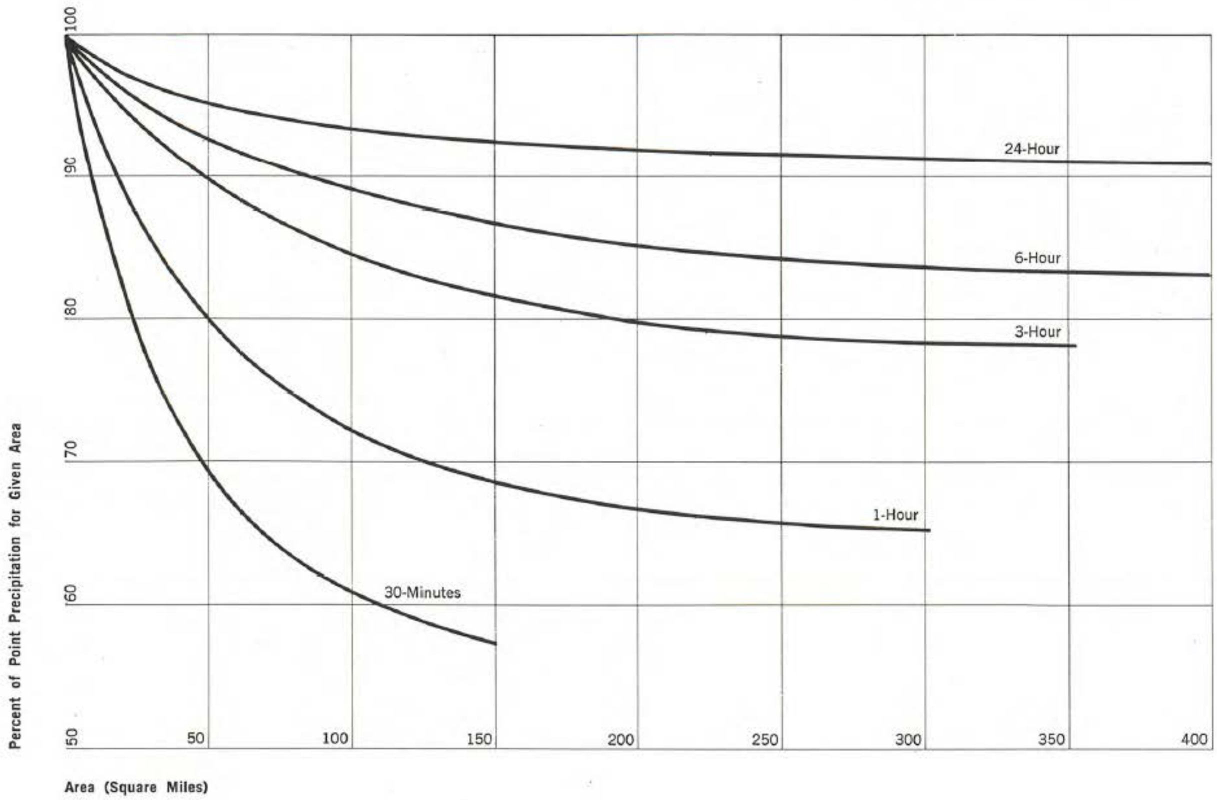


Figure 1: NOAA Atlas 2 Volume 3 ARF curves

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.

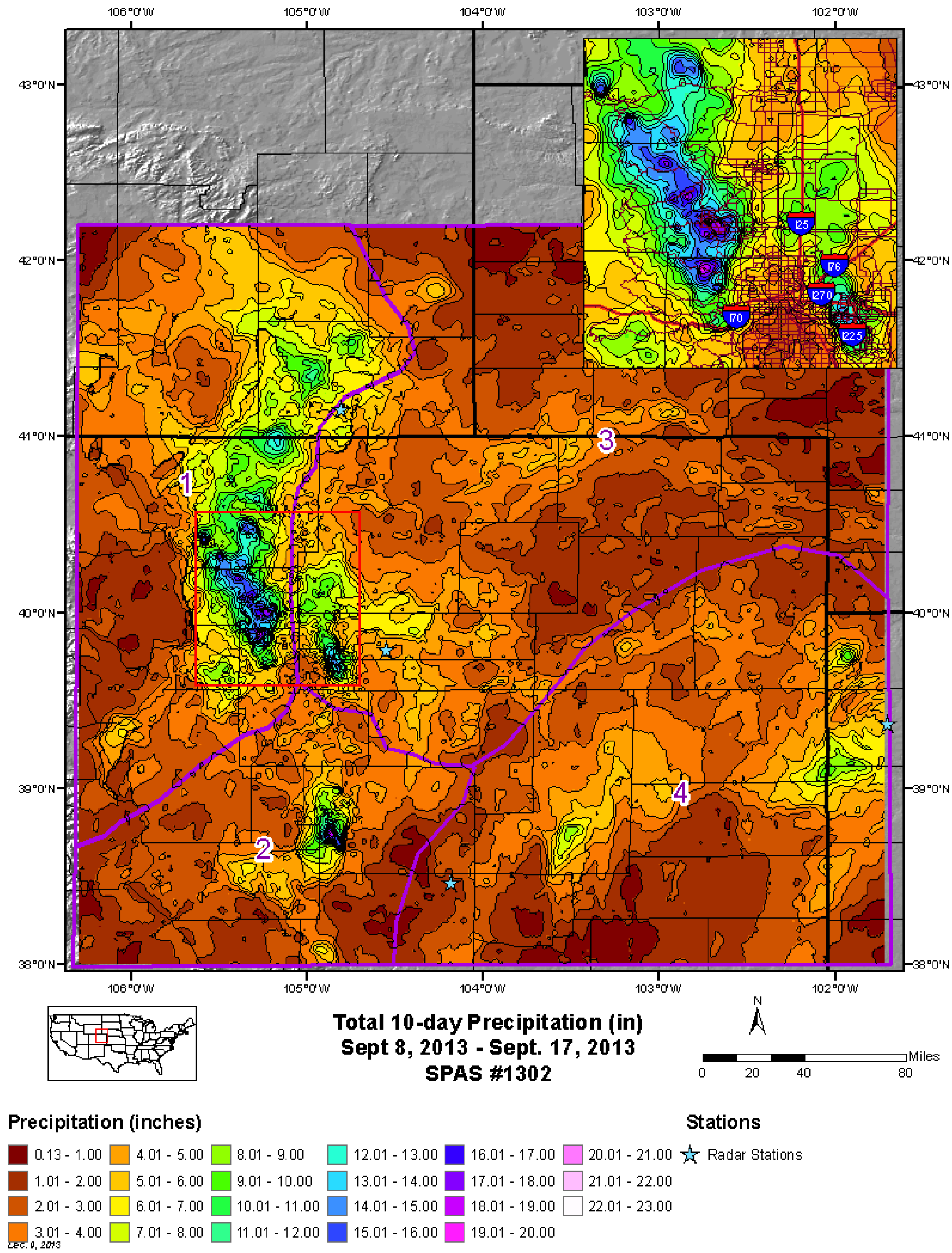


Figure 2: Basin specific ARFs for the September 2013 event compared to NOAA Atlas 2 ARF curve and to the HMR 55a Orographic C ARF

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.

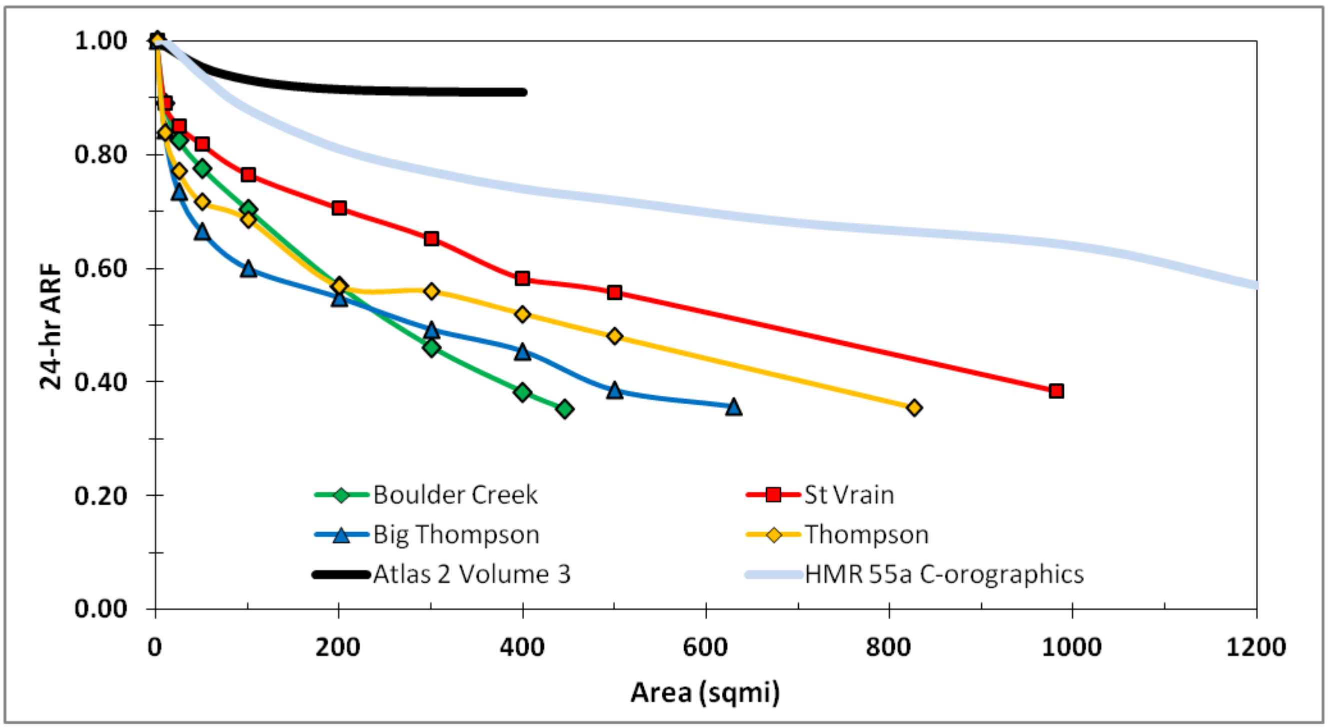


Figure 3: Basin specific 24-hour ARFs for the September 2013 event compared to NOAA Atlas 2 24-hour ARF curve and to the HMR 55A Orographic C 24-hour ARF curve

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"

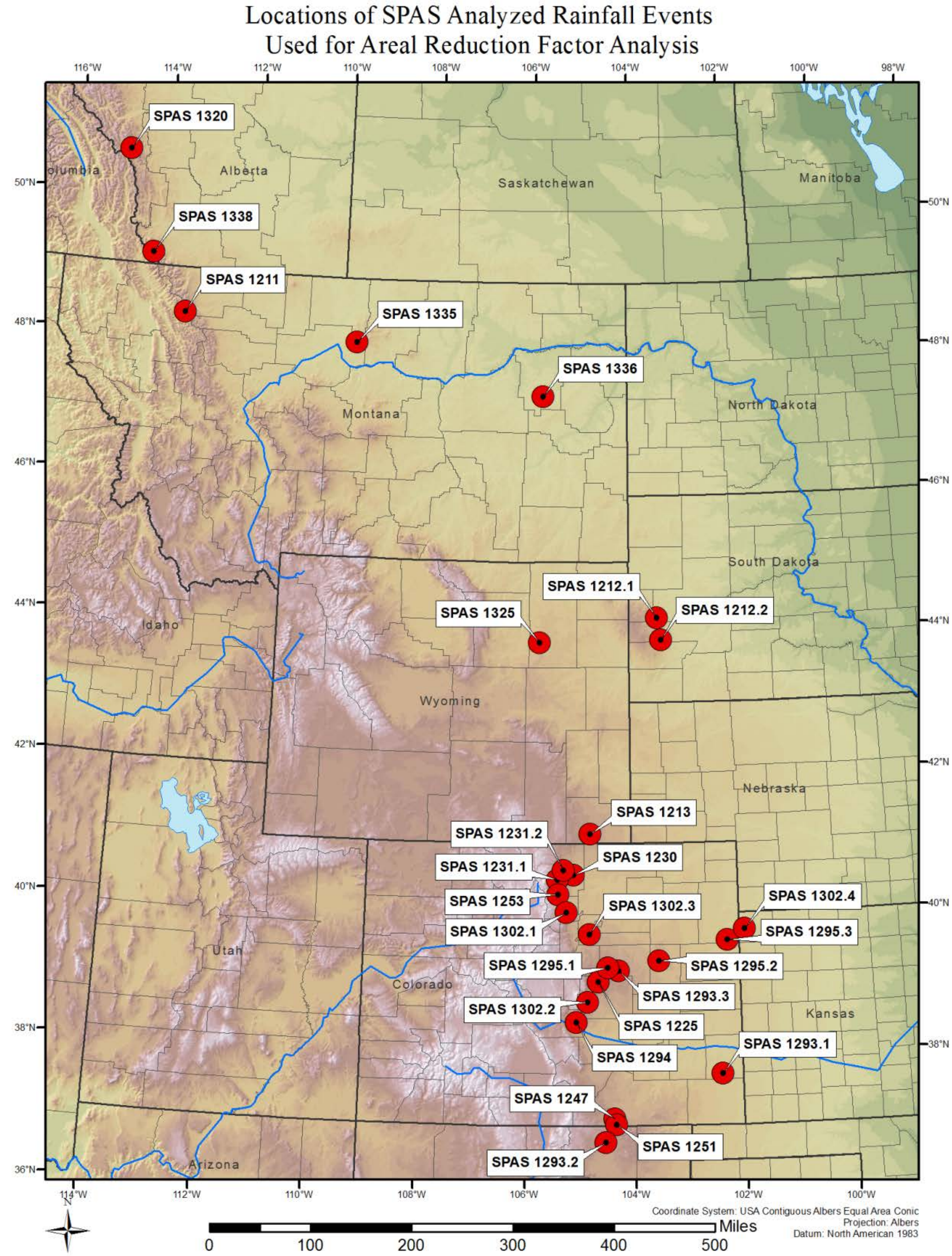


Figure 4: Initial twenty-eight SPAS storm center locations with similar meteorology and topography as the September 2013 storm event

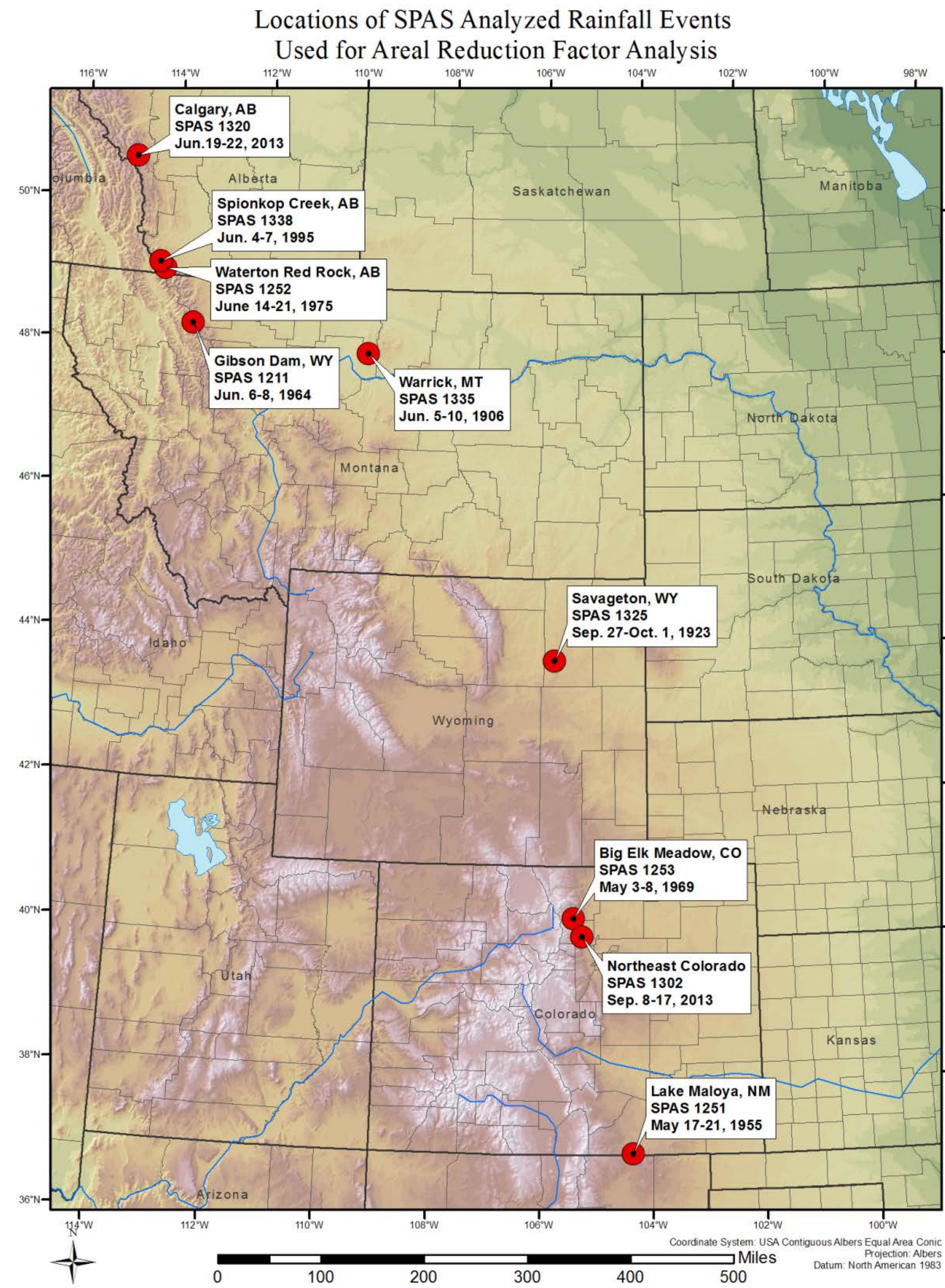


Figure 5: Final SPAS storm center locations used to derive 24-hr ARFs

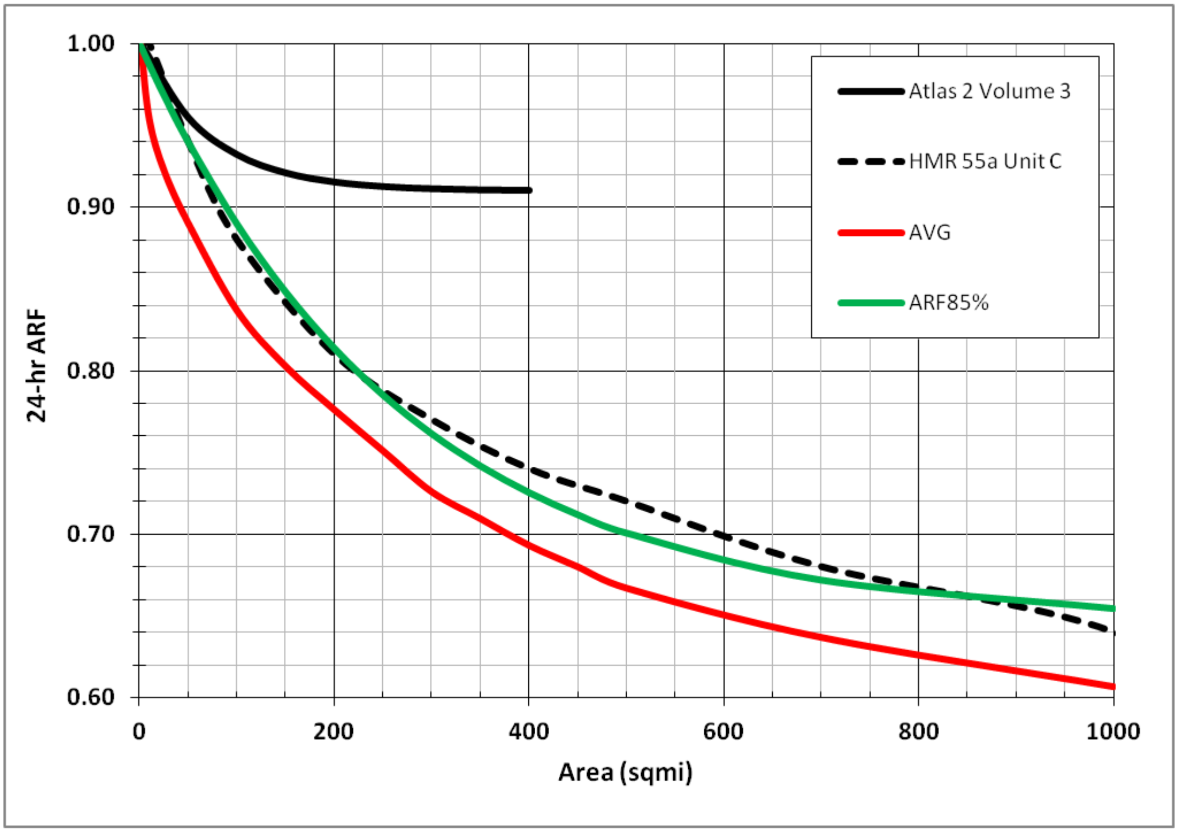


Figure 6: The average 24-hour ARF curve and final 85% confidence 24-hour ARF curve. The NOAA Atlas 2 24-hour ARF curve and HMR 55A Orographic C 24-hour ARF curve are shown for comparison.

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).

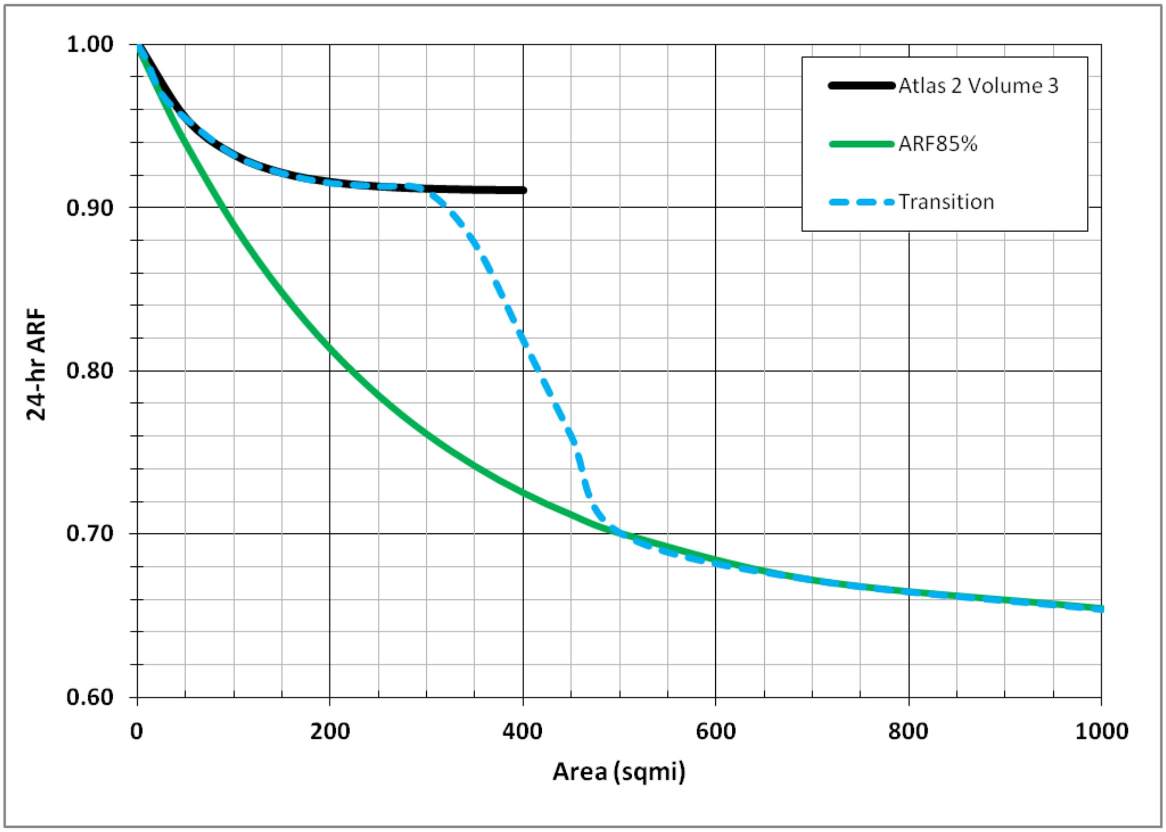


Figure 7: Final 24-hr ARF curve with transition between NOAA Atlas 2 and AWA $ARF_{85\%}$

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	-

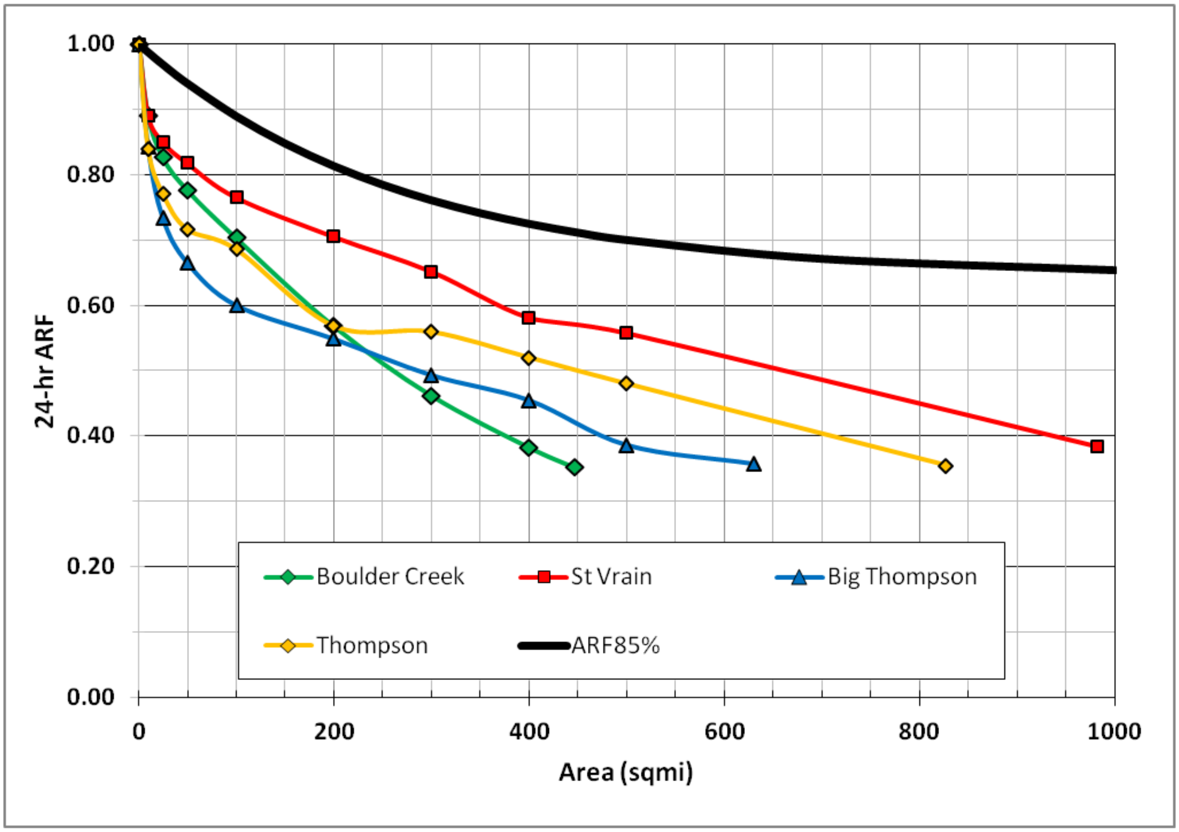


Figure 8: 24-hour ARF curve compared to basin specific ARFs for the September 2013 event

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Appendix D

Rainfall/Runoff Modeling

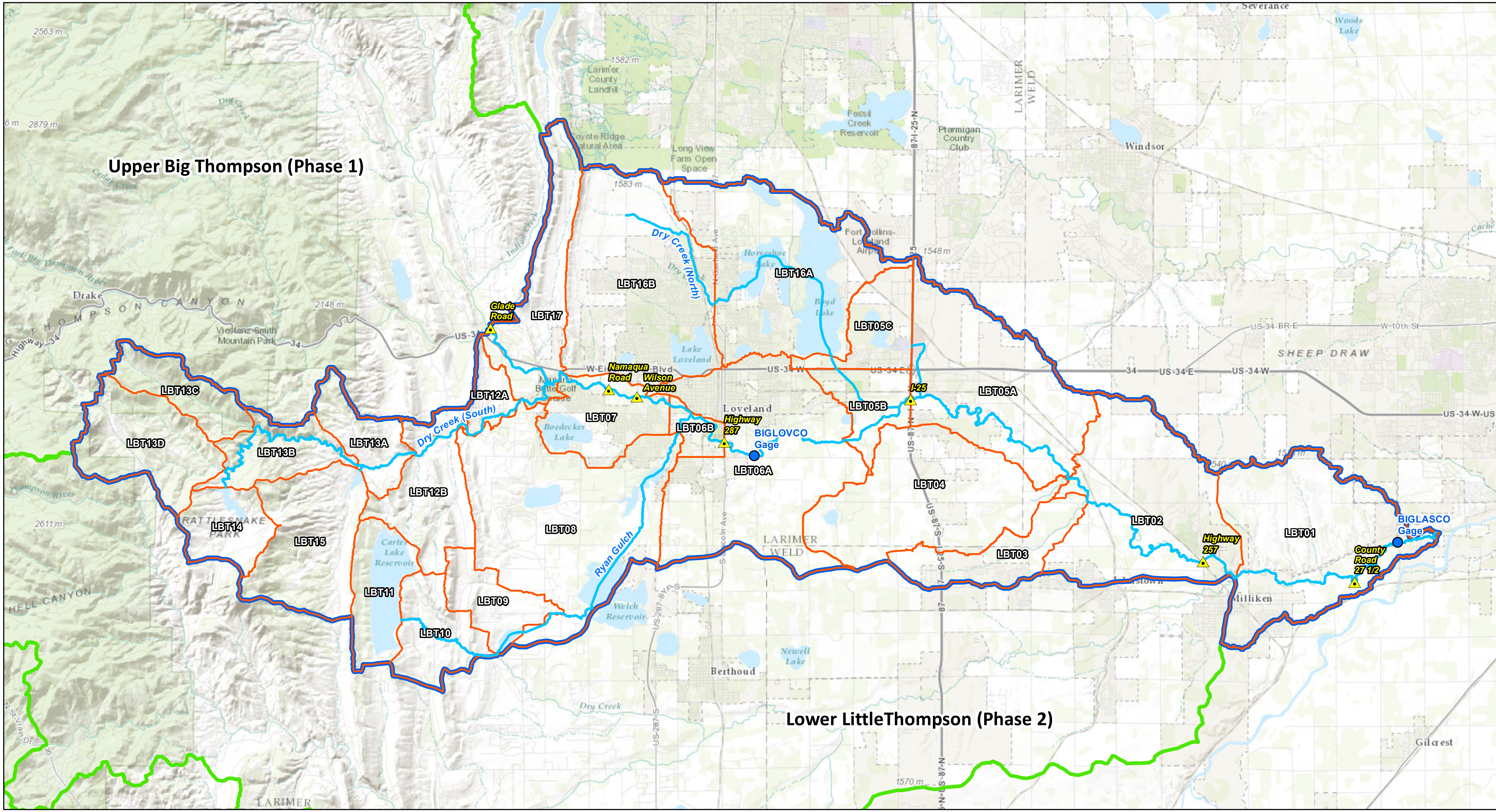
Appendix D.1

HEC-HMS Model Input

Figure D.1 – Lower Big Thompson Watershed (Phase 2)

HEC-HMS Model Input Parameters

Stage-Storage-Discharge Rating Tables



Lower Big Thompson 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 Big Thompson SubBasins
- Lower Big Thompson Watershed Boundary
- Big Thompson Watershed Boundary

Location	Peak Discharge	Tributary Area
Big Thompson near Glade Road	19,000	461
Big Thompson at Namaqua Road	22,000	499
Big Thompson at Wilson Avenue	22,000	499
Big Thompson at Highway 287	22,000	531
Big Thompson at I-25	19,600	577
Big Thompson at Highway 257	17,700	620
Big Thompson at County Road 27 1/2	24,900	829

Appendix D.1

Lower Big Thompson 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 Big Thompson	Source	Upper Big Thompson Model Inflow Hydrographs	461.14														
J1260B	Junction	BT below Buckhorn Creek	461.14														
R1260B	Reach	BT between Buckhorn Creek and Rossum Drive	461.14									14804	0.004	0.100	0.100	0.100	0.074
LBT17	Subbasin	Spring Glade near Devils Backbone	6.10	62.3	79.7	0.4	0.10	7.4	3.1	111.1	2.9						
LBT14	Subbasin	Basin Tributary to Pinewood Reservoir	3.07	58.0	73.7	0.4	0.15	2.6	1.3	223.4	2.0						
Pinewood Reservoir	Reservoir	Pinewood Reservoir - No Discharge to Dry Creek in 2013 Flood	3.07														
R1230	Reach	Mill Gulch below Pinewood Reservoir	3.07									3089	0.018	0.150	0.150	0.150	
LBT13D	Subbasin	Skinner Gulch and Quillian Gulch	5.97	51.7	66.5	0.4	0.15	4.1	2.6	446.7	2.6						
J1220	Junction	Confluence of Skinner Gulch and Mill Gulch	9.03														
R1220	Reach	Upper Cottonwood Creek	9.03									11011	0.033	0.150	0.150	0.150	
LBT13C	Subbasin	Saddle Notch Gulch	3.10	50.6	65.1	0.4	0.15	4.4	2.3	450.1	2.6						
J1210	Junction	Cottonwood Creek at Confluence with Saddle Notch Gulch	12.13														
R1210	Reach	Middle Cottonwood Creek	12.13									19898	0.032	0.150	0.150	0.150	
LBT13B	Subbasin	Cottonwood Creek Area	3.41	60.0	74.8	0.4	0.15	7.1	3.0	240.2	3.7						
J1200	Junction	Cottonwood Creek above Flatiron Reservoir	15.54														
R1200	Reach	Lower Cottonwood Creek	15.54									587	0.019	0.150	0.150	0.150	
LBT15	Subbasin	Flatiron Reservoir Tributary Area	7.03	61.4	77.4	0.4	0.15	6.1	1.9	396.7	2.8						
Flatiron Reservoir	Reservoir	Flatiron Reservoir - No Discharge to Dry Creek in 2013 Flood	7.03														
J1180	Junction	Dry Creek below Flatiron Reservoir	22.57														
R1180	Reach	Dry Creek below Flatiron Reservoir	22.57									7421	0.018	0.150	0.150	0.150	
LBT13A	Subbasin	Bear Track Road Tributary	2.88	61.6	77.7	0.4	0.15	3.9	1.9	303.5	2.5						
J1170	Junction	Dry Creek at Hogback Ridge	25.45														
R1170	Reach	Dry Creek below Hogback Ridge	25.45									8830	0.012	0.100	0.100	0.100	
LBT12B	Subbasin	Dry Creek Area along Sedona Hills Drive	5.22	64.6	83.1	0.4	0.15	4.4	2.5	206.8	3.0						
J1160	Junction	Dry Creek at CR 20	30.68														
R1160	Reach	Dry Creek above Golf Course	30.68									10058	0.009	0.050	0.050	0.050	
LBT12A	Subbasin	Dry Creek Area above Golf Course	1.39	69.5	84.7	0.4	0.15	3.2	1.3	25.8	3.1						
J1150	Junction	Dry Creek above Confluence with BT	32.07														
R1150	Reach	Dry Creek through Golf Course	32.07									9082	0.007	0.050	0.050	0.050	
J1260	Junction	BT at Rossum Drive (Dry Creek Confl.)	499.31														
R1260	Reach	BT between Rossum Drive and Wilson Drive	499.31									10971	0.004	0.100	0.100	0.100	0.055
J1140	Junction	BT at Wilson Avenue	499.31														
R1140	Reach	BT between Wilson Ave. and Taft Ave.	499.31									5180	0.005	0.100	0.100	0.100	0.026
LBT07	Subbasin	BT Area at Taft Ave.	4.41	69.4	85.9	0.4	0.10	4.7	3.1	28.2	3.1						
J1130	Junction	BT at Taft Avenue	503.72														
R1130	Reach	BT between Taft Ave. and 1st St.	503.72									3042	0.002	0.100	0.100	0.100	0.015
LBT11	Subbasin	Carter Lake	3.74	52.4	73.4	0.4	0.10	3.0	0.8	124.6	1.3						
Carter Lake	Reservoir	Carter Lake can contain PMP storm event per USBR	3.74														
R1120	Reach	Ryan Gulch between Carter Lake and Hertha Reservoir	3.74									15082	0.018	0.050	0.050	0.050	
LBT10	Subbasin	Area upstream of Hertha Reservoir	3.67	51.8	74.2	0.4	0.10	3.3	1.8	202.5	1.6						
J1110	Junction	Ryan Gulch at Hertha Reservoir	7.41														
R1110	Reach	Ryan Gulch between Hertha and Lonetree Reservoirs	7.41									10412	0.010	0.050	0.050	0.050	
LBT09	Subbasin	Area upstream of Lonetree Reservoir	2.96	50.8	75.1	0.4	0.10	3.7	1.7	198.3	1.7						
J1100	Junction	Ryan Gulch upstream of Lonetree Reservoir	10.37														
R1100	Reach	Ryan Gulch above Taft Avenue	10.37									30226	0.006	0.050	0.050	0.050	
LBT08	Subbasin	Ryan Gulch Area	15.94	54.3	76.6	0.4	0.10	6.6	3.7	90.4	3.0						
J1090	Junction	Ryan Gulch upstream of BT	26.31														
R1090	Reach	Ryan Gulch at BT confluence	26.31									3128	0.003	0.050	0.050	0.050	
J1080	Junction	BT at Railroad Avenue	530.03														
R1080	Reach	BT between Airport Road and Hwy 287	530.03									7627	0.003	0.100	0.100	0.100	0.019
LBT06B	Subbasin	BT area between Taft Ave. and Hwy 287	1.14	54.2	78.0	0.4	0.10	2.3	1.3	30.2	1.8						
J1070	Junction	BT at Hwy 287 (Lincoln Ave.)	531.17														
R1070	Reach	BT between Hwy 287 and CR 9E	531.17									20026	0.003	0.100	0.100	0.100	0.050
LBT06A	Subbasin	BT area upstream of CR 9E	15.46	61.1	82.5	0.4	0.10	5.3	3.1	36.6	3.1						
J1060	Junction	BT downstream of County Road 9E	546.63														
R1060	Reach	BT upstream of I-25	546.63									11036	0.002	0.100	0.100	0.100	0.028
LBT16B	Subbasin	Lake Loveland Area upstream of Hwy 287	14.11	79.8	90.7	0.4	0.10	7.0	6.1	72.1	3.8						
J1290	Junction	Dry Creek above Boyd Lake	14.11														
R1290	Reach	Dry Creek through Boyd Lake	14.11									34916	0.001	0.050	0.050	0.050	
LBT16A	Subbasin	Boyd Lake Area	13.24	80.0	91.2	0.4	0.10	10.2	2.9	17.2	4.2						
Boyd Lake	Reservoir	Zero discharge in 2013 Flood, Provides 880 ac-ft of storage for zero release	27.35														
R1280	Reach	Boyd Lake Overflow Path	27.35									14875	0.006	0.050	0.050	0.050	
LBT05B	Subbasin	BT Area upstream of I-25	2.75	50.9	73.6	0.4	0.10	2.2	0.9	79.6	1.4						
J1050	Junction	BT at I-25	576.73														
LBT05C	Subbasin	Equalizer Lake Area (Centerra)	3.36	57.7	77.6	0.4	0.10	3.6	0.9	33.6	1.8						
J1270	Junction	Loveland and Greeley Canal at I-25	3.36														
R1270	Reach	Loveland and Greeley Canal overflow east of I-25	3.36									10892	0.011	0.050	0.050	0.050	
J1040b	Junction	Downstream of I-25	580.09														
R1050	Reach	BT downstream of I-25	580.09									44850.18	0.001	0.10	0.10	0.10	0.07
LBT05A	Subbasin	BT Area downstream of I-25	11.71	56.1	75.8	0.4	0.10	9.3	4.1	27.6	4.2						
LBT04	Subbasin	BT Area above CR 15	11.43	53.7	76.7	0.4	0.10	8.5	5.9	44.6	4.3						
J1030	Junction	BT upstream of County Road 15 1/2	603.23														
R1030	Reach	BT upstream of CR 15 1/2	603.23									4882	0.001	0.100	0.100	0.100	0.010
LBT03	Subbasin	BT Area at CR 15 1/2	3.92	53.8	78.5	0.4	0.10	5.5	2.8	71.7	2.7						
J1010	Junction	BT at County Road 15 1/2 (Road 52 & Railroad)	607.15														
R1010	Reach	BT above confluence with Little Thompson	607.15									30863	0.002	0.100	0.100	0.100	0.062
LBT02	Subbasin	BT Area above confluence with Little Thompson	12.69	50.9	79.6	0.4	0.10	6.3	3.5	34.8	3.4						
J1000B	Junction	Big Thompson Upstream of Confluence with Little Thompson	619.84														
Little Thompson	Source	Little Thompson Model Inflow Hydrographs	196.00														
J1000	Junction	Confluence of Big Thompson and Little Thompson	815.84														
R1000	Reach	BT between Little Thompson and South Platte	815.84									35076	0.001	0.100	0.100	0.100	0.094
LBT01	Subbasin	BT Area below Little Thompson Confluence	12.70	51.8	82.6	0.4	0.10	7.7	5.4	49.8	4.0						
Outlet1	Sink	BT Confluence with South Platte River	828.54														

Appendix D.1 (continued)

Boyd Lake Stage-Storage-Discharge Rating Table

Source - Boyd Lake Spillway Project Final Design Report (Boyle 2005)

Description	Reservoir WSEL (ft)	Volume (ac-ft)	Discharge (cfs)
Bottom of Reservoir	4905.30	3	0
Principal Spillway Crest Elevation	**4959.59**	**44274**	**0**
	4959.60	44291	1
	4959.70	44456	12
	4959.80	44622	23
Emergency Spillway Crest Elevation	**4959.84**	**44688**	**27**
	4959.90	44788	39
	4960.00	44953	58
	4960.08	45086	73
	4960.10	45119	254
	4960.20	45284	1157
	4960.27	45400	1789
	4960.30	45450	2337
	4960.40	45619	4162
	4960.46	45720	5257
	4960.50	45788	6155
	4960.60	45958	8399
	4960.66	46059	9745
	4960.70	46127	10812
	4960.80	46296	13478
	4960.85	46381	14811
	4960.90	46465	16200
	4961.00	46634	18977
	4961.04	46702	20088
	4961.10	46804	21731
	4961.20	46973	24470
	4961.23	47024	25292
	4961.30	47142	27108
	4961.40	47315	29703
	4961.42	47350	30222
	4961.50	47488	32042
	4961.60	47661	34317
	4961.62	47696	34772
	4961.70	47834	36517
	4961.80	48006	38699
	4961.81	48023	38917
	4961.90	48179	40714
Maximum Water Surface Elevation (ft)	**4962.00**	**48352**	**42710**

*Electronic version of Appendix D includes full S-V-D relationship from bottom of reservoir up to principal spillway crest.

Pinewood Reservoir Stage-Storage-Discharge Rating Table

Source - Dry Creek Floodplain Report - (Anderson 2002)

Description	Reservoir WSEL (ft)	Volume (ac-ft)	Discharge (cfs)
Bottom of Reservoir	**6550.0**	**416**	**0**
	6551.0	444	0
	6552.0	475	0
	6553.0	507	0
	6554.0	541	0
	6555.0	576	0
	6556.0	613	0
	6557.0	653	0
	6558.0	694	0
	6559.0	737	0
	6560.0	782	0
	6561.0	829	0
	6562.0	878	0
	6563.0	929	0
	6564.0	982	0
	6565.0	1036	0
	6566.0	1092	0
	6567.0	1151	0
	6568.0	1213	0
	6569.0	1277	0
	6570.0	1344	0
	6571.0	1413	0
	6572.0	1486	0
	6573.0	1562	0
	6574.0	1643	0
Starting Elevation in 2013 Flood (6574.19')	**6575.0**	**1726**	**0**
	6576.0	1812	0
	6577.0	1901	0
	6578.0	1992	0
	6579.0	2085	0
Emergency Spillway Crest	**6580.0**	**2181**	**0**
	6581.0	2280	300
	6582.0	2381	750
	6583.0	2486	1500
	6584.0	2594	2550
	6585.0	2705	3750

Flatiron Reservoir Stage-Storage-Discharge Rating Table

Source - Dry Creek Floodplain Report - (Anderson 2002)

Description	Reservoir WSEL (ft)	Volume (ac-ft)	Discharge (cfs)
Bottom of Reservoir	**5455.0**	**130**	**0**
	5456.0	152	0
	5457.0	176	0
	5458.0	202	0
	5459.0	230	0
	5460.0	260	0
	5461.0	291	0
	5462.0	324	0
	5463.0	358	0
	5464.0	393	0
Starting Elevation in 2013 Flood (5465.01')	**5465.0**	**429**	**0**
	5466.0	467	0
	5467.0	506	0
	5468.0	547	0
	5469.0	589	0
	5470.0	632	0
	5471.0	676	0
	5472.0	722	0
Emergency Spillway Crest	**5472.8**	**760**	**0**
	5473.0	769	298
	5474.0	817	1820
	5475.0	867	3940
	5476.0	918	6970
	5477.0	970	10300
	5478.0	1024	14240
	5479.0	1079	19390
	5480.0	1136	24727

Appendix D.2

Rainfall Data

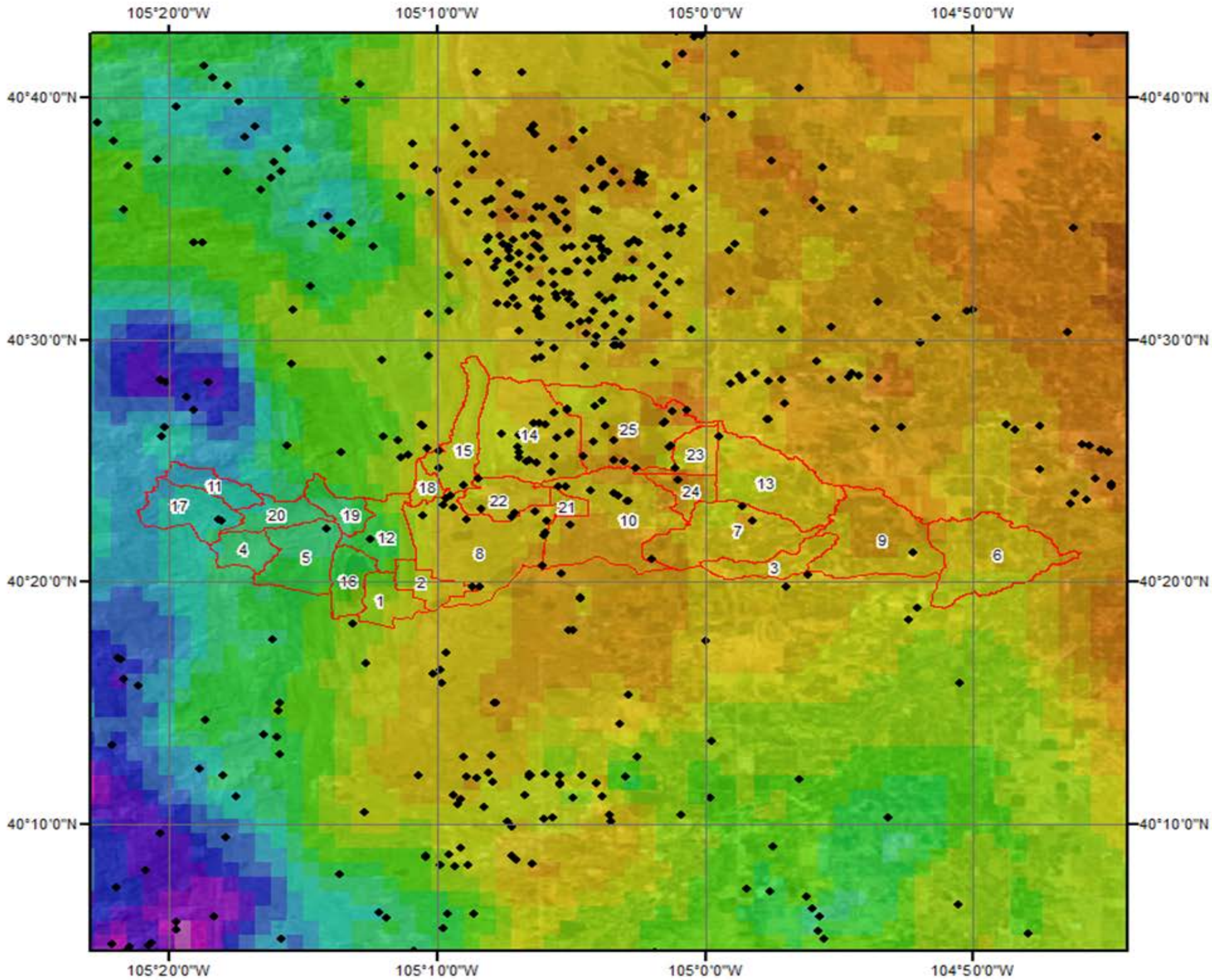
10-day Precipitation Map

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

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
12.01 - 13.00
13.01 - 14.00
14.01 - 15.00
15.01 - 16.00
16.01 - 17.00
17.01 - 18.00
18.01 - 19.00
19.01 - 20.00
20.01 - 21.00

Miles: 0 3.5 7 14
Kilometers: 0 4.5 9 18 27

07/21/2014

Appendix D.2 (continued)

NRCS 24-hour Rainfall Distributions

		(Cumulative Precipitation)/(Total Storm Precipitation)			
Time hours	**t/T**	**Type 1 Storm**	**Type 1A Storm**	**Type II Storm**	**Type III Storm**
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

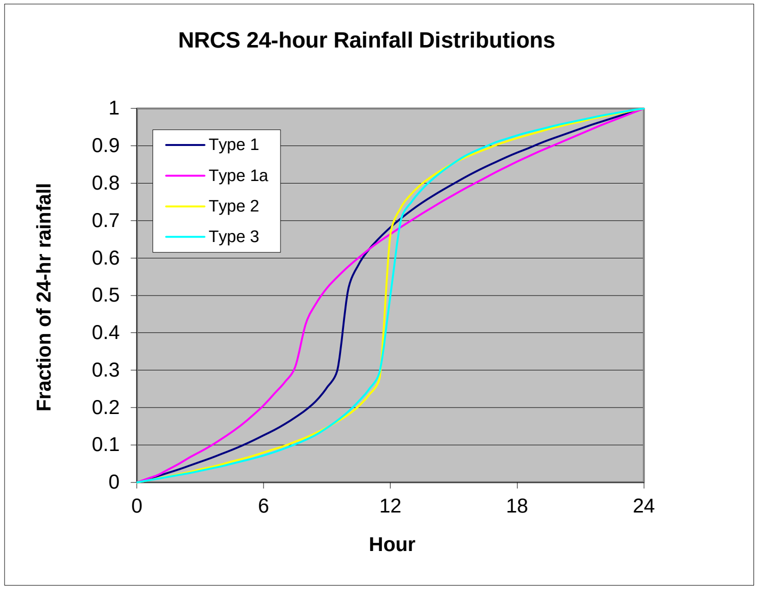
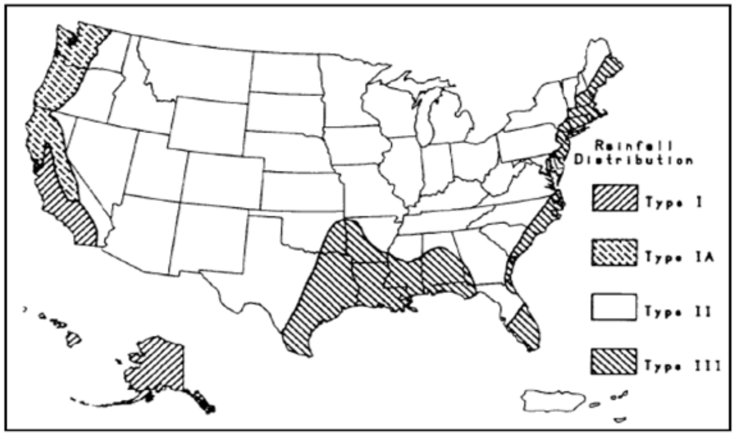


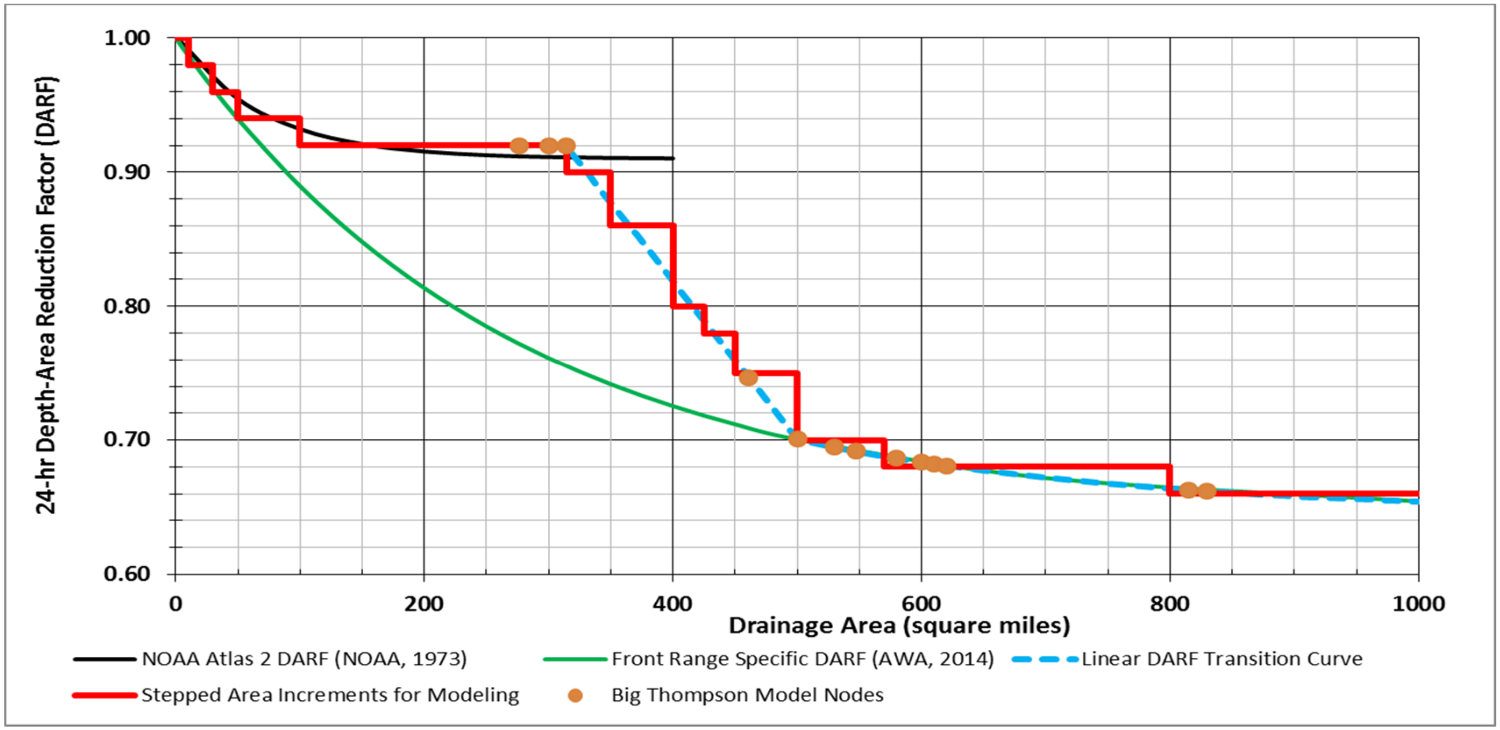
Figure 1: Geographic boundaries for the NRCS rainfall distributions



Lower Big Thompson - Phase 2
NOAA Atlas 14, Volume 8, Version 2
Point Precipitation Frequency Estimates

Basin ID	Centroid Lat.	Centroid Long.
LBT01	40.348	-104.814
LBT02	40.368	-104.894
LBT03	40.347	-104.959
LBT04	40.365	-104.983
LBT05A	40.406	-104.953
LBT05B	40.395	-105.013
LBT05C	40.421	-105.012
LBT06A	40.376	-105.059
LBT06B	40.389	-105.085
LBT07	40.39	-105.126
LBT08	40.357	-105.14
LBT09	40.331	-105.168
LBT10	40.321	-105.194
LBT11	40.333	-105.218
LBT12A	40.402	-105.174
LBT12B	40.367	-105.205
LBT13A	40.386	-105.228
LBT13B	40.381	-105.266
LBT13C	40.399	-105.307
LBT13D	40.38	-105.318
LBT14	40.354	-105.286
LBT15	40.351	-105.254
LBT16A	40.439	-105.046
LBT16B	40.437	-105.109
LBT17	40.437	-105.154

100% - No DARF Adjustment (0 to 10 sq.mi.)					
Basin ID	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**500-yr**
LBT01	2.77	3.50	4.13	4.81	6.64
LBT02	2.82	3.57	4.21	4.92	6.82
LBT03	2.84	3.61	4.28	5.01	6.97
LBT04	2.85	3.63	4.31	5.05	7.06
LBT05A	2.85	3.62	4.29	5.03	7.02
LBT05B	2.87	3.66	4.35	5.11	7.18
LBT05C	2.87	3.66	4.36	5.13	7.22
LBT06A	2.91	3.72	4.42	5.20	7.31
LBT06B	2.94	3.77	4.49	5.29	7.47
LBT07	3.01	3.87	4.62	5.46	7.74
LBT08	2.98	3.83	4.58	5.40	7.66
LBT09	2.95	3.79	4.54	5.36	7.61
LBT10	2.94	3.78	4.53	5.36	7.64
LBT11	2.96	3.81	4.57	5.41	7.73
LBT12A	3.15	4.05	4.84	5.73	8.14
LBT12B	3.07	3.95	4.73	5.60	8.00
LBT13A	3.13	4.03	4.84	5.73	8.20
LBT13B	3.08	3.98	4.78	5.68	8.17
LBT13C	2.99	3.87	4.67	5.56	8.06
LBT13D	2.93	3.79	4.57	5.44	7.89
LBT14	2.96	3.82	4.58	5.44	7.83
LBT15	3.00	3.87	4.64	5.50	7.88
LBT16A	2.92	3.73	4.45	5.24	7.40
LBT16B	3.02	3.88	4.63	5.47	7.77
LBT17	3.12	4.02	4.81	5.69	8.10



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 Big Thompson watershed, several model scenarios were run using adjusted rainfall depths. The nine different scenarios included the unadjusted NOAA rainfall depths and eight levels of reduced NOAA rainfall depths (98%, 96%, 94%, 92%, 75%, 70%, 68%, and 66%). Each of these nine scenarios were run for all five predictive storms (10-, 25-, 50-, 100-, and 500-yr). The Upper Big Thompson and Little Thompson models were also rerun for the lower DARF values to develop the required (provided 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 Big Thompson are shown as orange circles 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 Big Thompson and Little Thompson Rivers

2013 Flood 10-day Period

2013 Flood Maximum 24-hour Rainfall Period

10-year Predictive Storm

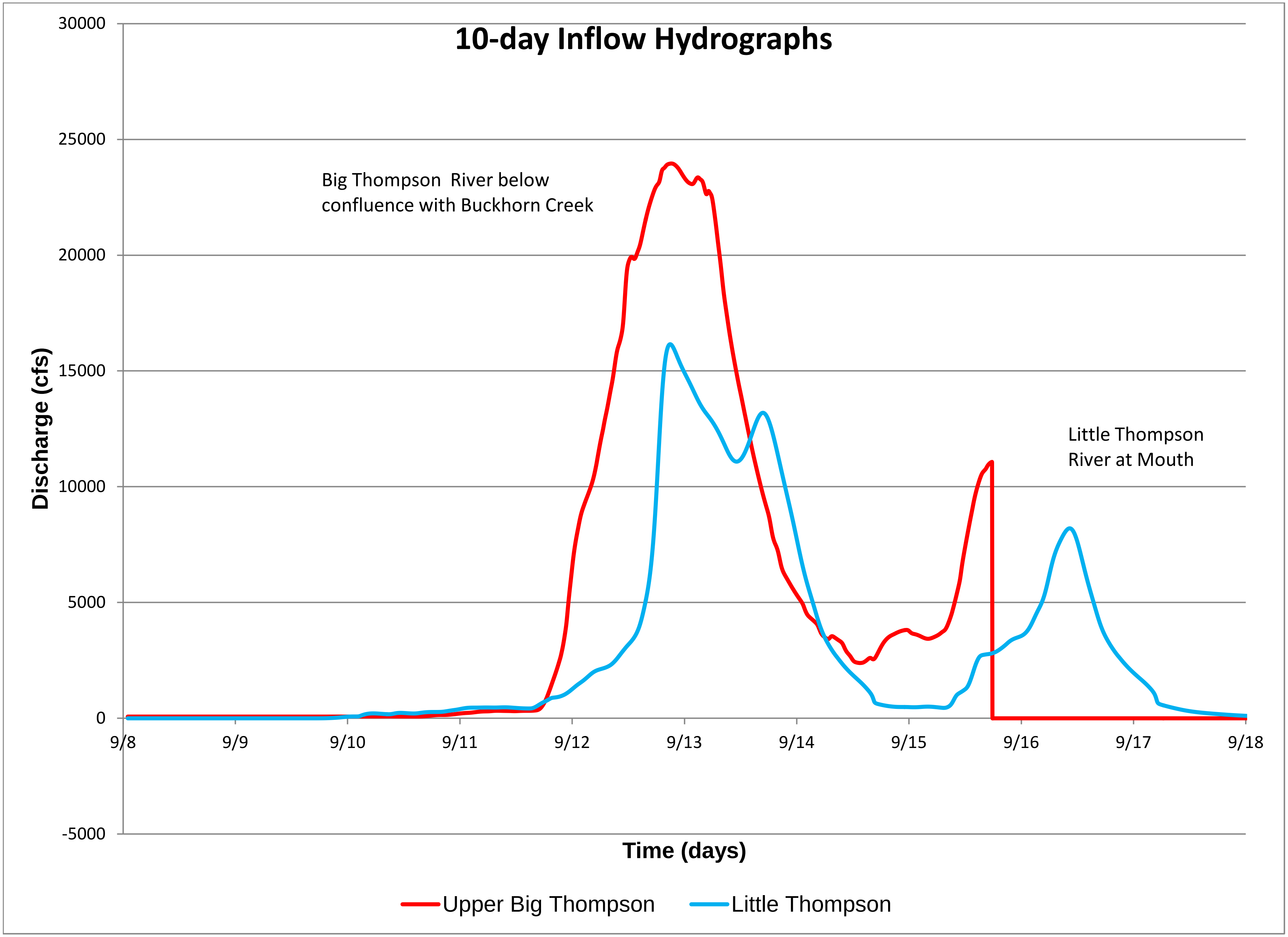
25-year Predictive Storm

50-year Predictive Storm

100-year Predictive Storm

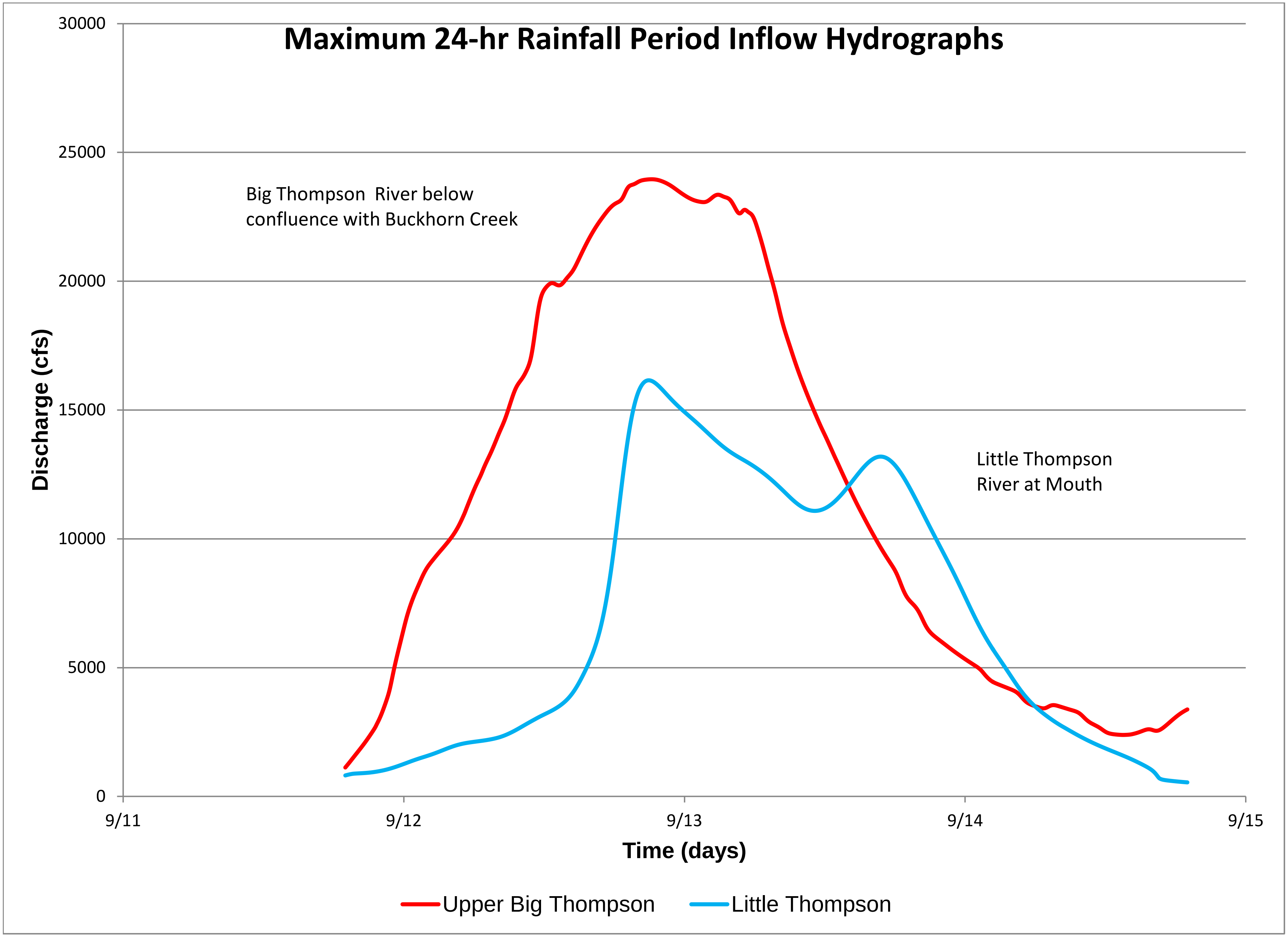
500-year Predictive Storm

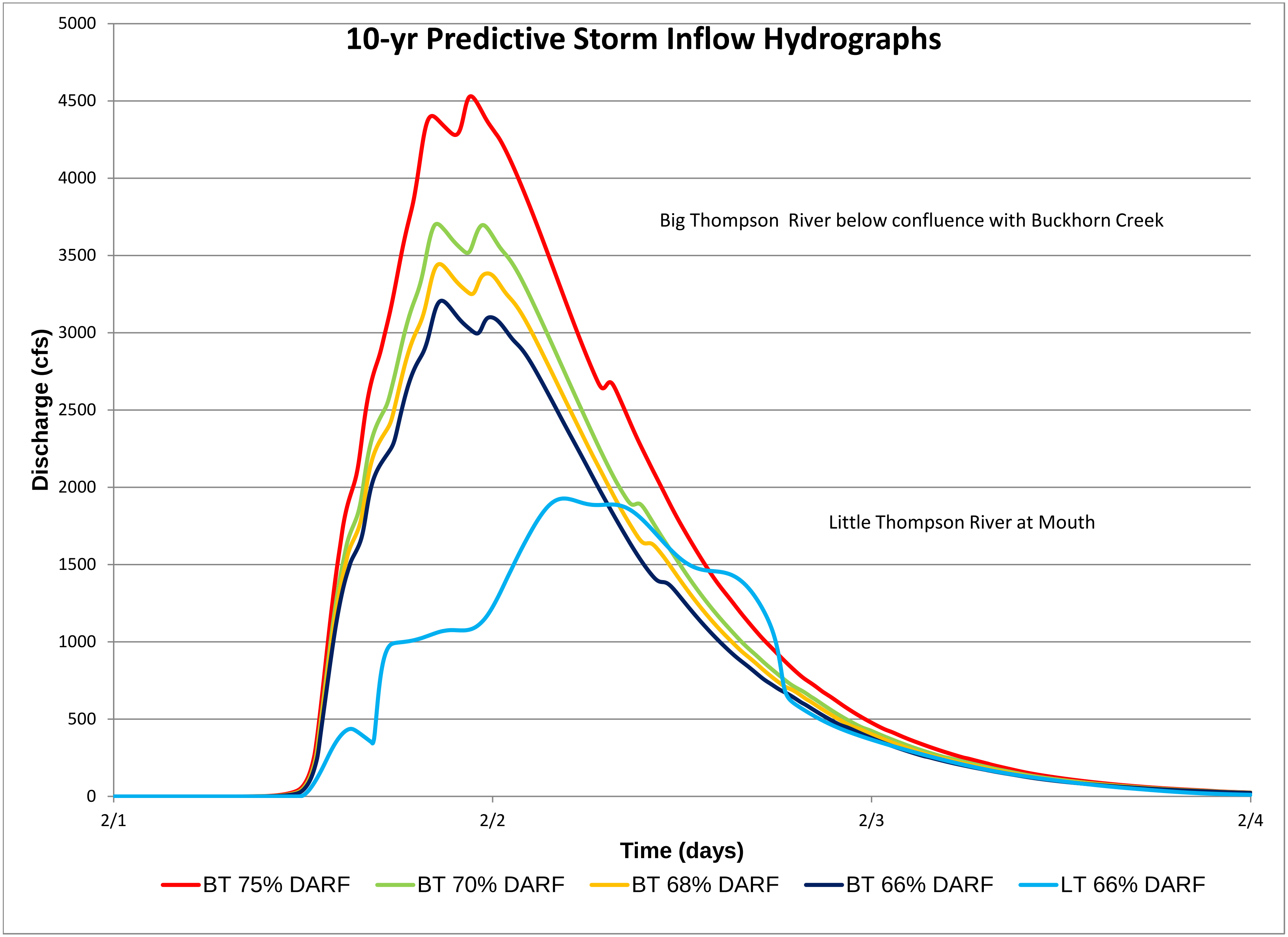
Appendix D.3



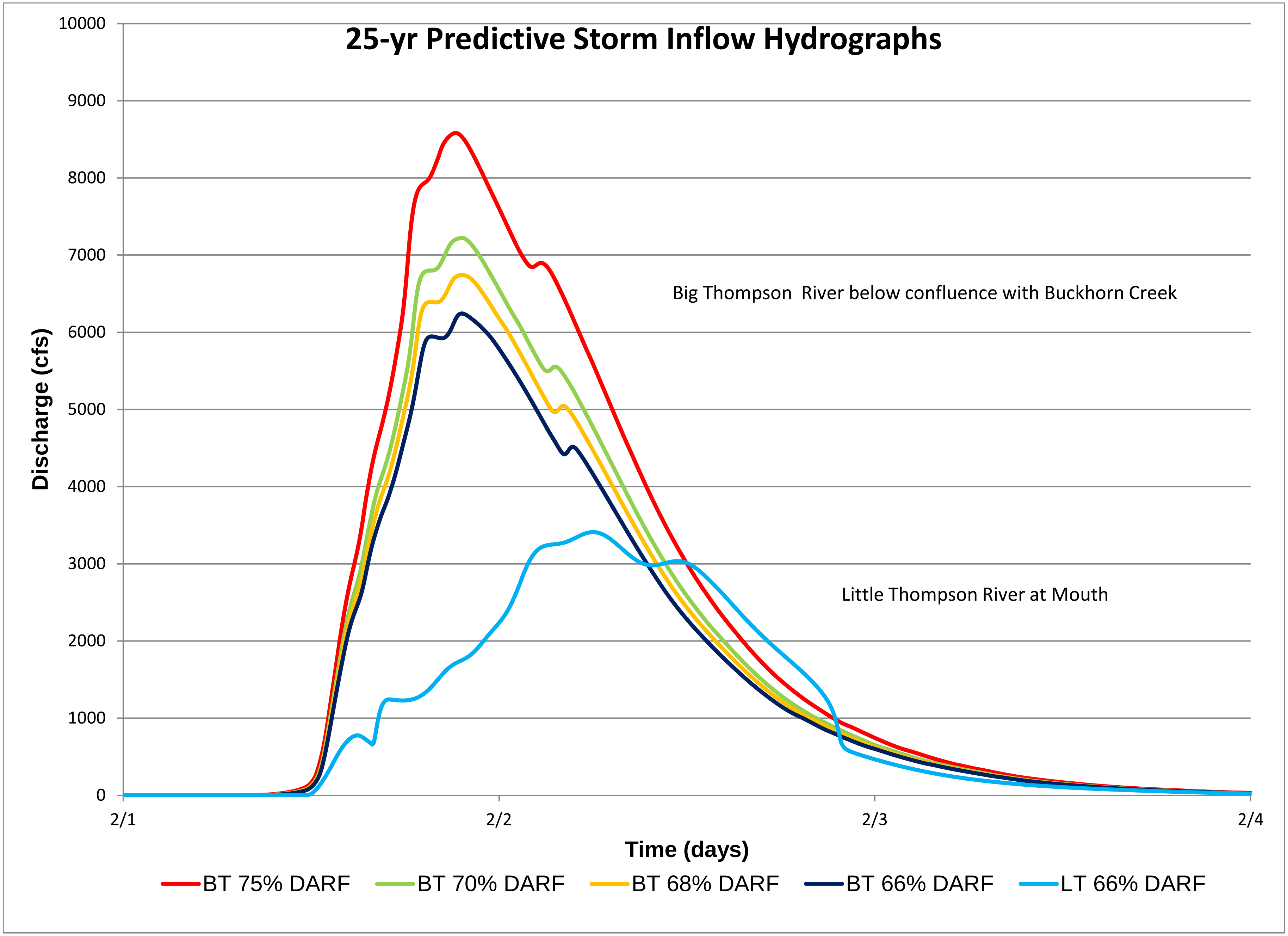
10-day Inflow Hydrographs
Discharge (cfs)
30000
25000
20000
15000
10000
5000
0
-5000
Big Thompson River below
confluence with Buckhorn Creek
Little Thompson
River at Mouth
9/8
9/9
9/10
9/11
9/12
9/13
9/14
9/15
9/16
9/17
9/18
Time (days)
Upper Big Thompson
Little Thompson

Appendix D.3 (continued)

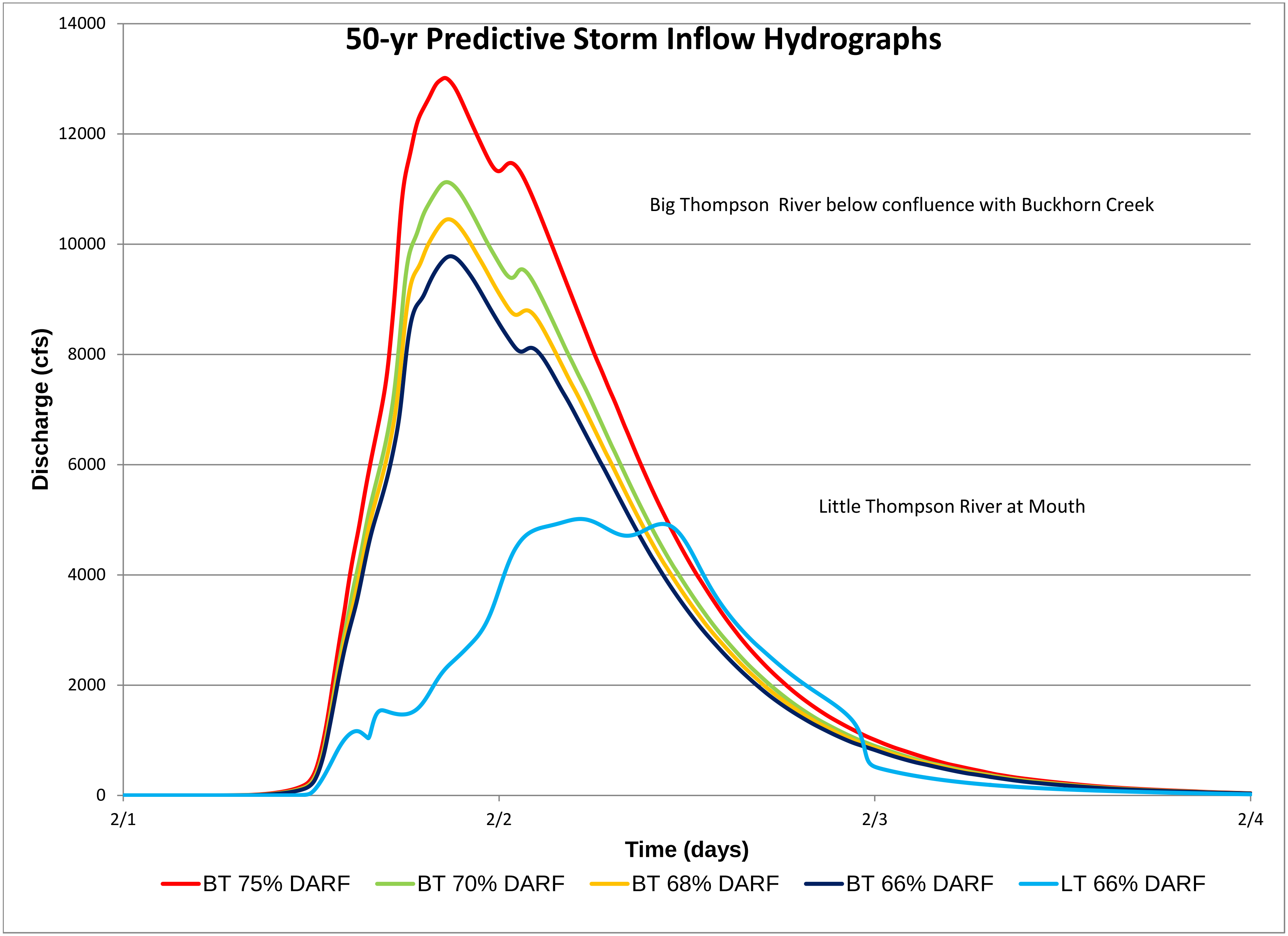




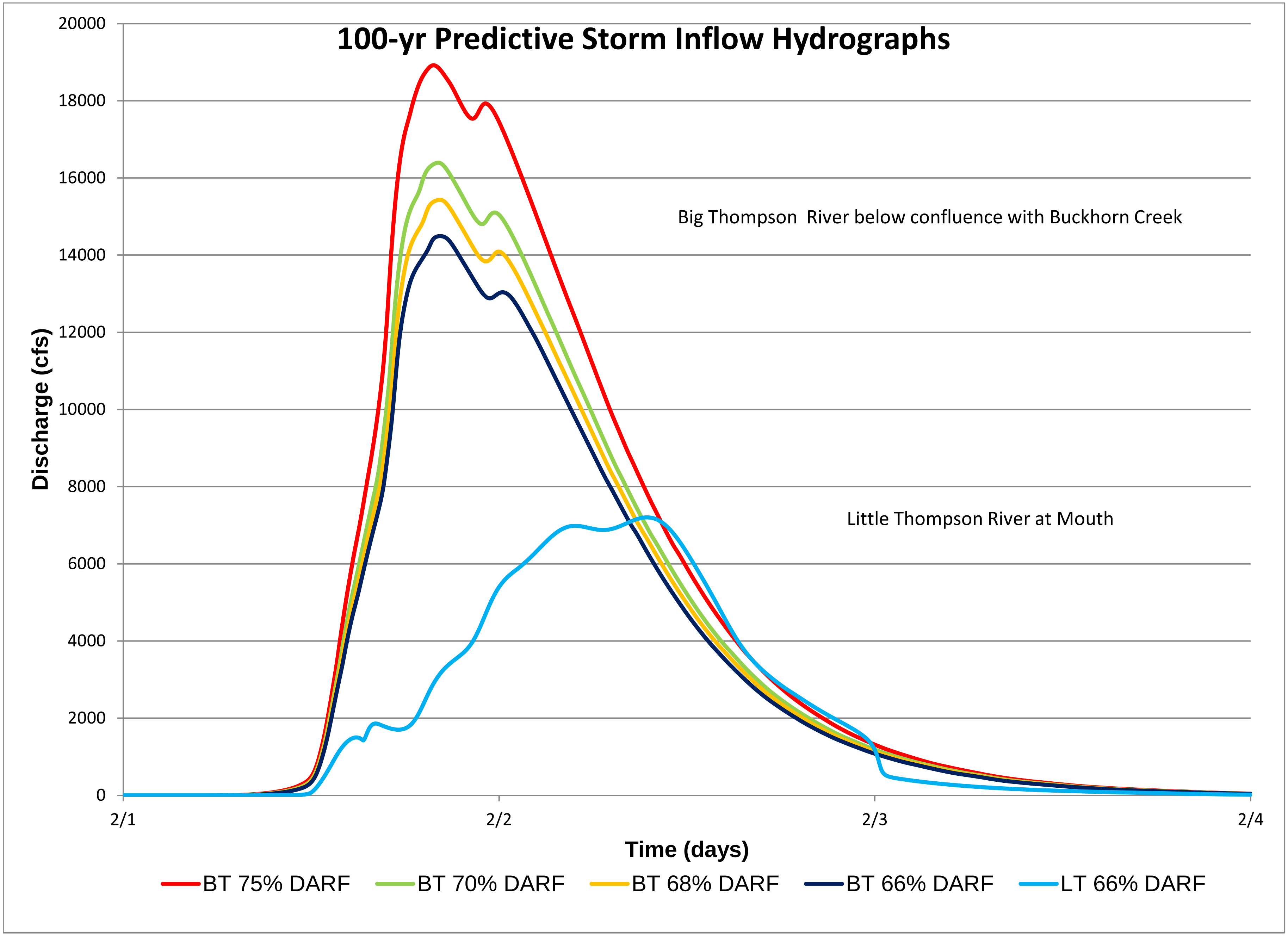
10-yr Predictive Storm Inflow Hydrographs
Big Thompson River below confluence with Buckhorn Creek
Little Thompson River at Mouth
Discharge (cfs)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
2/1
2/2
2/3
2/4
Time (days)
BT 75% DARF
BT 70% DARF
BT 68% DARF
BT 66% DARF
LT 66% DARF

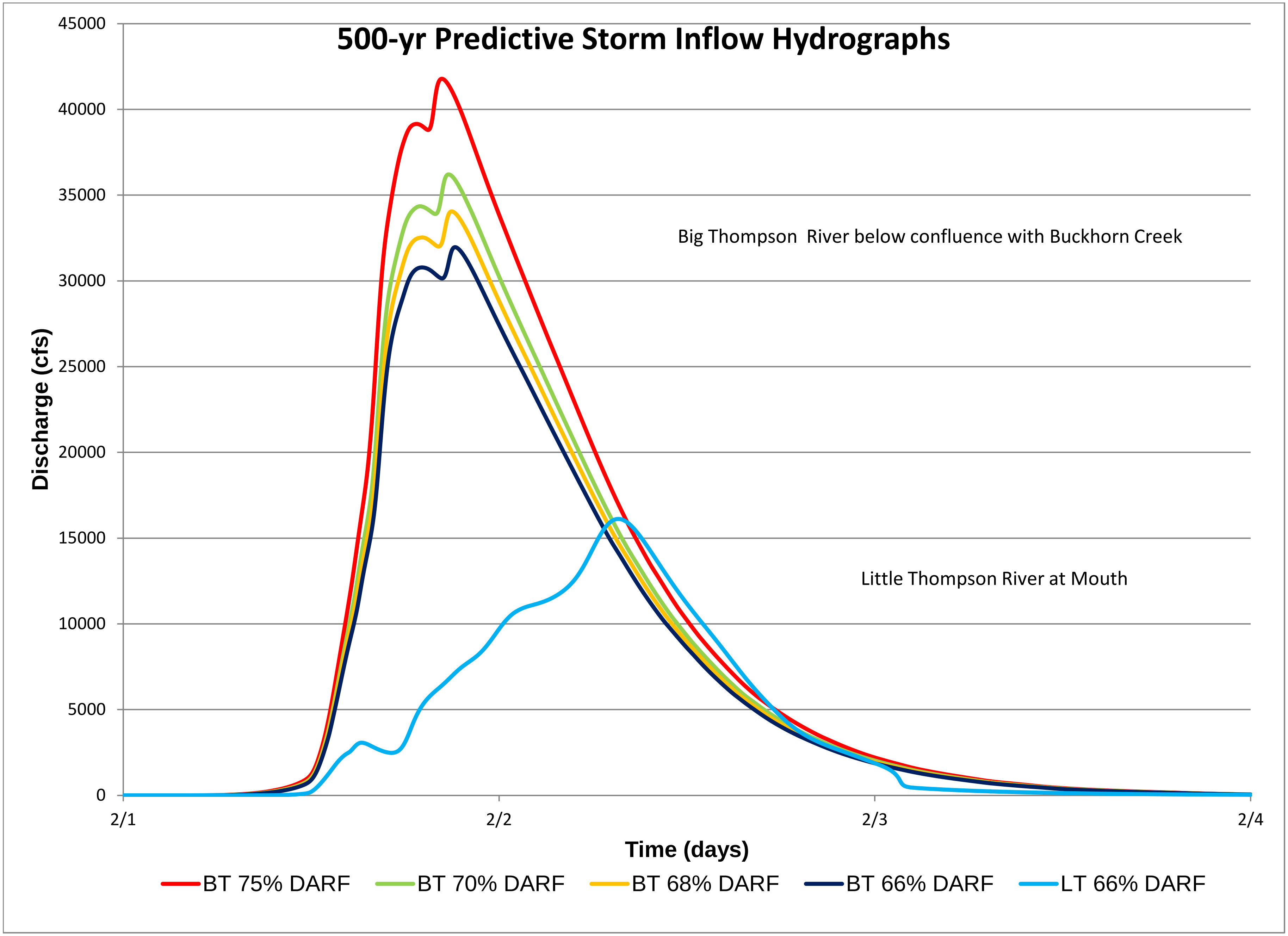

25-yr Predictive Storm Inflow Hydrographs
Discharge (cfs)
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
2/1
2/2
2/3
2/4
Time (days)
Big Thompson River below confluence with Buckhorn Creek
Little Thompson River at Mouth
BT 75% DARF
BT 70% DARF
BT 68% DARF
BT 66% DARF
LT 66% DARF

Appendix D.3 (continued)



Appendix D.3 (continued)




500-yr Predictive Storm Inflow Hydrographs
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
Discharge (cfs)
Big Thompson River below confluence with Buckhorn Creek
Little Thompson River at Mouth
2/1
2/2
2/3
2/4
Time (days)
BT 75% DARF
BT 70% DARF
BT 68% DARF
BT 66% DARF
LT 66% DARF

Appendix D.4

HEC-HMS Model Results

Appendix D.4

Lower Big Thompson Design Point	Description	Area (sq. mi.)	Estimated Peak (cfs)	HEC-HMS 2013 Flood Calibrated Model 10-day (cfs)	HEC-HMS (Cal) 2013 Flood Max 24hr Period CN calibrated (cfs)	HEC-HMS Model (Max24hr CN Calibrated) NOAA Design Storms (DARF 66% to 100%) 10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	FIS Regulatory Peak Discharge 10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	Ayres 2013 Flood Frequency Analysis 10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	DARF %
Upper Big Thompson	Upper Big Thompson Model Inflow Hydrographs	461.14	19,000	23,957	23,957	4,529	8,582	13,012	18,920	41,780	4,700	12,300	19,000	44,000					75%
J1260B	BT below Buckhorn Creek	461.14		23,957	23,957	4,529	8,582	13,012	18,920	41,780									75%
R1260B	BT between Buckhorn Creek and Rossum Drive	461.14		22,244	22,244	4,463	8,557	12,980	18,879	41,610									75%
LBT17	Spring Glade near Devils Backbone	6.10		412	254	476	756	1,020	1,326	2,205									100%
LBT14	Basin Tributary to Pinewood Reservoir	3.07		417	290	187	330	474	649	1,179									100%
Pinewood Reservoir	Pinewood Reservoir - No Discharge to Dry Creek in 2013 Flood	3.07	0	0	0	118	202	285	406	771	119	230	296	492					100%
R1230	Mill Gulch below Pinewood Reservoir	3.07		0	0	118	202	285	406	770									100%
LBT13D	Skinner Gulch and Quillian Gulch	5.97		631	502	168	337	525	762	1,534									100%
J1220	Confluence of Skinner Gulch and Mill Gulch	9.03		631	502	269	497	741	1,062	2,110	755	2,125	2,923	5,186					100%
R1220	Upper Cottonwood Creek	9.03		631	502	269	497	741	1,062	2,110									100%
LBT13C	Saddle Notch Gulch	3.10		325	272	83	171	269	394	804									100%
J1210	Cottonwood Creek at Confluence with Saddle Notch Gulch	12.13		948	762	325	619	946	1,354	2,753									98%
R1210	Middle Cottonwood Creek	12.13		947	762	325	619	946	1,353	2,753									98%
LBT13B	Cottonwood Creek Area	3.41		348	272	159	270	380	513	908									100%
J1200	Cottonwood Creek above Flatiron Reservoir	15.54		1,292	996	463	860	1,296	1,833	3,618									98%
R1200	Lower Cottonwood Creek	15.54		1,292	996	463	860	1,296	1,833	3,617									98%
LBT15	Flatiron Reservoir Tributary Area	7.03		803	610	451	745	1,031	1,369	2,364									100%
Flatiron Reservoir	Flatiron Reservoir - No Discharge to Dry Creek in 2013 Flood	7.03	0	0	0	444	731	1,010	1,339	2,328									100%
J1180	Dry Creek below Flatiron Reservoir	22.57		1,292	996	832	1,494	2,217	3,073	5,776	2,329	6,023	8,128	13,885					98%
R1180	Dry Creek below Flatiron Reservoir	22.57		1,291	996	832	1,494	2,216	3,072	5,773									98%
LBT13A	Bear Track Road Tributary	2.88		466	300	222	362	499	657	1,121									100%
J1170	Dry Creek at Hogback Ridge	25.45		1,668	1,150	999	1,776	2,620	3,607	6,710	2,403	6,224	8,425	14,440					98%
R1170	Dry Creek below Hogback Ridge	25.45		1,667	1,149	999	1,775	2,619	3,605	6,706									98%
LBT12B	Dry Creek Area along Sedona Hills Drive	5.22		947	374	458	698	923	1,181	1,915									100%
J1160	Dry Creek at CR 20	30.68		2,467	1,380	1,289	2,245	3,260	4,443	8,106	2,873	7,192	9,719	16,582					96%
R1160	Dry Creek above Golf Course	30.68		2,465	1,379	1,288	2,244	3,257	4,441	8,102									96%
LBT12A	Dry Creek Area above Golf Course	1.39		134	67	135	201	262	332	525									100%
J1150	Dry Creek above Confluence with BT	32.07		2,577	1,442	1,387	2,397	3,463	4,709	8,542	2,970	7,365	9,955	16,930					96%
R1150	Dry Creek through Golf Course	32.07	2,448	2,575	1,441	1,386	2,396	3,461	4,707	8,539	3,020	7,465	10,090	17,135					96%
J1260	BT at Rossum Drive (Dry Creek Confl.)	499.31		23,523	23,684	4,332	8,383	12,941	19,021	40,536									70%
R1260	BT between Rossum Drive and Wilson Drive	499.31	22,000	22,258	22,410	4,318	8,371	12,923	18,997	40,400									70%
J1140	BT at Wilson Avenue	499.31	22,000	22,258	22,410	4,318	8,371	12,923	18,997	40,400									70%
R1140	BT between Wilson Ave. and Taft Ave.	499.31		21,686	21,833	4,313	8,365	12,915	18,988	40,362									70%
LBT07	BT Area at Taft Ave.	4.41		277	154	427	632	817	1,029	1,617									100%
J1130	BT at Taft Avenue	503.72		21,749	21,894	4,417	8,515	13,140	19,305	40,841									70%
R1130	BT between Taft Ave. and 1st St.	503.72		21,422	21,563	4,410	8,495	13,122	19,276	40,766									70%
LBT11	Carter Lake	3.74		595	304	304	542	783	1,070	1,935									100%
Carter Lake	Carter Lake can contain PMP storm event per USBR	3.74	0	0	0	0	0	0	0	0									100%
R1120	Ryan Gulch between Carter Lake and Hertha Reservoir	3.74		0	0	0	0	0	0	0									100%
LBT10	Area upstream of Hertha Reservoir	3.67		303	189	269	470	673	916	1,635									100%
J1110	Ryan Gulch at Hertha Reservoir	7.41		303	189	269	470	673	916	1,635									100%
R1110	Ryan Gulch between Hertha and Lonetree Reservoirs	7.41		303	189	269	470	673	914	1,634									100%
LBT09	Area upstream of Lonetree Reservoir	2.96		216	80	228	392	556	747	1,314									100%
J1100	Ryan Gulch upstream of Lonetree Reservoir	10.37		517	266	471	821	1,177	1,596	2,850									98%
R1100	Ryan Gulch above Taft Avenue	10.37		515	265	469	817	1,171	1,588	2,835									98%
LBT08	Ryan Gulch Area	15.94		708	250	863	1,437	2,003	2,656	4,594									98%
J1090	Ryan Gulch upstream of BT	26.31		1,221	507	1,332	2,254	3,173	4,236	7,383									98%
R1090	Ryan Gulch at BT confluence	26.31		1,217	503	1,329	2,249	3,166	4,225	7,373									98%
J1080	BT at Railroad Avenue	530.03		21,724	21,877	4,707	8,974	13,897	20,411	42,647	4,700	12,300	19,000	44,000					70%
R1080	BT between Airport Road and Hwy 287	530.03		21,310	21,460	4,694	8,965	13,883	20,387	42,561									70%
LBT06B	BT area between Taft Ave. and Hwy 287	1.14		47	22	101	166	228	300	507									100%
J1070	BT at Hwy 287 (Lincoln Ave.)	531.17	22,000	21,314	21,463	4,703	8,979	13,912	20,429	42,628					3,617	11,801	18,743	51,518	70%
R1070	BT between Hwy 287 and CR 9E	531.17		20,257	20,394	4,663	8,948	13,858	20,341	42,406									70%
LBT06A	BT area upstream of CR 9E	15.46		599	292	1,131	1,745	2,300	2,951	4,764									98%
J1060	BT downstream of County Road 9E	546.63		20,339	20,458	4,775	9,166	14,276	21,008	43,757	4,700	12,300	19,000	44,000					70%
R1060	BT upstream of I-25	546.63		19,763	19,875	4,762	9,141	14,223	20,920	43,601									70%
LBT16B	Lake Loveland Area upstream of Hwy 287	14.11		930	821	1,440	2,021	2,541	3,122	4,724									98%
J1290	Dry Creek above Boyd Lake	14.11		930	821	1,440	2,021	2,541	3,122	4,724									98%
R1290	Dry Creek through Boyd Lake	14.11		919	809	1,376	1,928	2,424	2,977	4,507									98%
LBT16A	Boyd Lake Area	13.24		729	629	1,206	1,681	2,104	2,578	3,869									98%
Boyd Lake	Boyd Lake Labyrinth Spillway	27.35	0	0	0	1,712	2,803	3,720	4,731	7,570									98%
R1280	Boyd Lake Overflow Path	27.35		0	0	1,712	2,802	3,718	4,730	7,567									98%
LBT05B	BT Area upstream of I-25	2.75		82	70	209	367	524	711	1,265									100%
J1050	BT at I-25	576.73	19,600	19,768	19,875	5,089	9,528	14,902	21,750	45,136	4,300	8,800	11,500	21,000					68%
LBT05C	Equalizer Lake Area (Centerra)	3.36		154	190	273	448	618	817	1,390									100%
J1270	Loveland and Greeley Canal at I-25	3.36		154	190	273	448	618	817	1,390									100%
R1270	Loveland and Greeley Canal overflow east of I-25	3.36		153	190	273	448	618	816	1,389									100%
J1040b	Downstream of I-25	580.09		19,777	19,878	5,096	9,542	14,936	21,820	45,314									68%
R1050	BT downstream of I-25	580.09		18,396	18,463	4,918	9,171	14,199	20,637	43,793									68%
LBT05A	BT Area downstream of I-25	11.71		353	286	421	698	963	1,281	2,201									98%
LBT04	BT Area above CR 15	11.43		285	165	430	709	973	1,285	2,192									98%
J1030	BT upstream of County Road 15 1/2	603.23		18,542	18,522	4,949	9,261	14,385	20,965	44,649	3,600	7,600	10,000	18,500					68%
R1030	BT upstream of CR 15 1/2	603.23		18,343	18,321	4,938	9,216	14,281	20,865	44,509									68%
LBT03	BT Area at CR 15 1/2	3.92		111	58	246	395	537	700	1,165									100%
J1010	BT at County Road 15 1/2 (Road 52 & Railroad)	607.15		18,372	18,323	4,939	9,221	14,292	20,889	44,588									68%
R1010	BT above confluence with Little Thompson	607.15		17,219	17,159	4,893	9,034	13,875	20,248	44,012									68%
LBT02	BT Area above confluence with Little Thompson	12.69		412	106	673	1,062	1,422	1,837	3,022									98%
J1000B	Big Thompson Upstream of Confluence with Little Thompson	619.84	17,700	17,260	17,163	4,895	9,048	13,906	20,309	44,244	2,200	4,700	6,500	12,000					68%
Little Thompson	Little Thompson Model Inflow Hydrographs	196.00	18,000	16,153	16,153	4,475	7,158	10,544	15,364	31,381	1,630	3,600	4,800	8,400					92%
J1000	Confluence of Big Thompson and Little Thompson	815.84		30,305	29,986	4,857	9,470	15,377	23,168	56,189	3,200	7,300	9,900	20,000					66%
R1000	BT between Little Thompson and South Platte	815.84		27,436	27,154	4,772	8,962	14,783	22,196	53,377									66%
LBT01	BT Area below Little Thompson Confluence	12.70		538	113	693	1,053	1,384	1,749	2,790									98%
Outlet1	BT Confluence with South Platte River	828.54	24,900	27,463	27,159	4,774	8,973	14,808	22,247	53,569	2,500	5,900	8,000	15,000	2,897	8,260	12,310	28,963	66%

Appendix D.5

HEC-HMS Model Results for Profiles

Appendix D.5

Lower Big Thompson

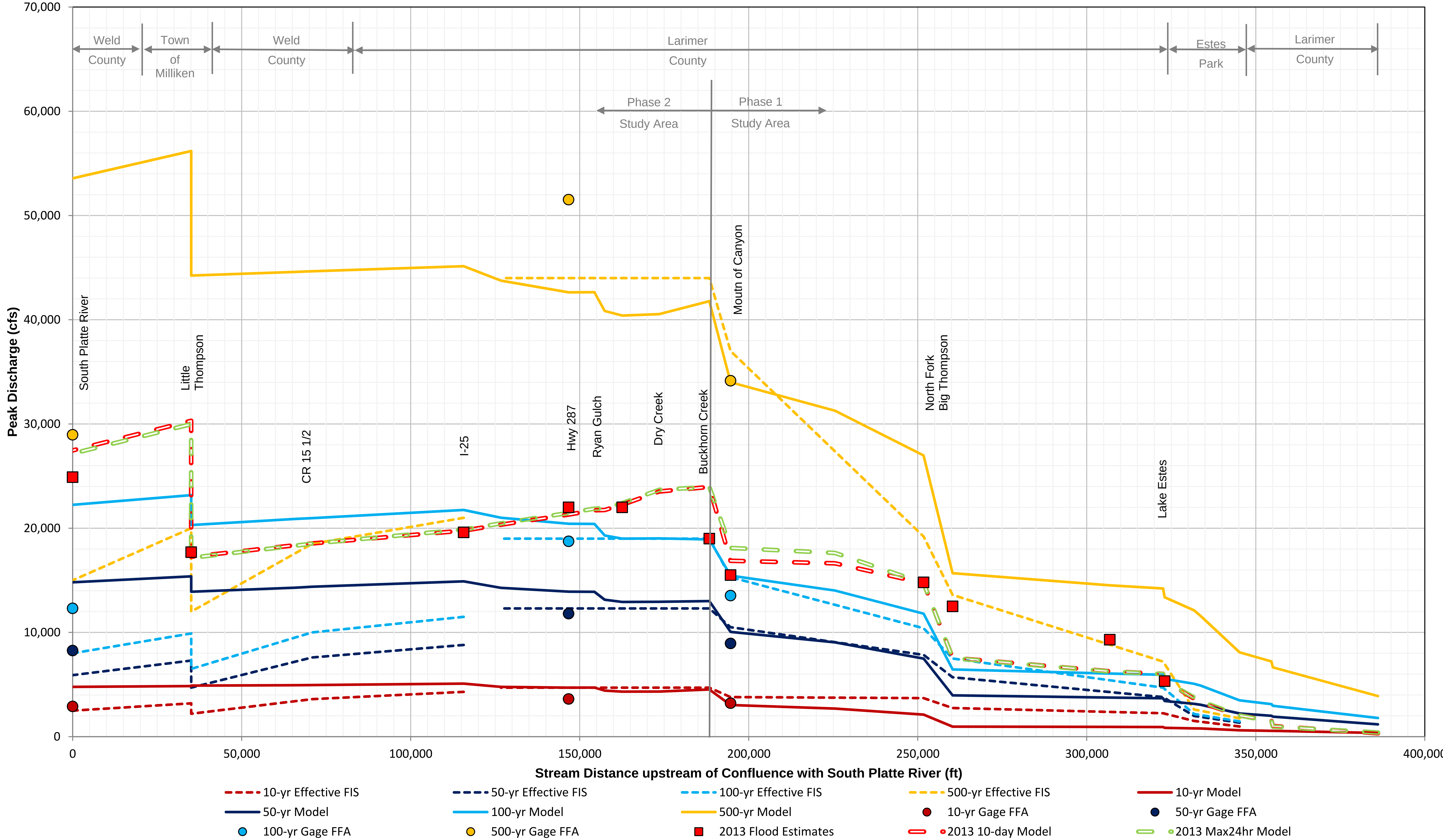
Design Point	Description	Approx. Station (ft)	Area (sq. mi.)	Estimated Peak (cfs)	2013 Flood Calibrated Model 10-day (cfs)	2013 Flood Max 24hr Period CN Calibrated (cfs)	NOAA Design Storms (CN Calib & DARF) 10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	FIS Regulatory Peak Discharge 10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	Ayres 2013 Flood Frequency Analysis 10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)
J4026	BT at confluence with Fern Creek in RMNP	386,147	32.51		329	394	354	737	1,183	1,793	3,893								
J4040	BT at Confluence of Glacier Creek	355,083	64.79		1,020	958	557	1,177	1,923	2,959	6,650								
J4037	BT at confluence with Wind River upstream of Estes Park	354,644	75.04		1,514	1,460	558	1,204	2,000	3,124	7,216								
J4047	BT at confluence with Beaver Brook	345,224	83.81		2,032	2,029	609	1,328	2,221	3,482	8,089	980	1,340	1,460	1,760				
J4052	BT at confluence with Fall River	333,709	126.10		3,274	3,398	786	1,794	3,056	4,896	11,580								
J4061	BT at confluence with Black Canyon Creek	331,762	136.27		3,639	3,773	794	1,834	3,148	5,074	12,118	1,510	1,990	2,180	2,600				
J4055	BT inflow to Lake Estes	323,054	154.12		5,415	5,342	846	1,980	3,424	5,548	13,370								
Lake Estes	Lake Estes (Olympus Dam)	323,054	154.12	5,327	5,327	5,327	846	1,980	3,424	5,548	13,370								
J4058	BT at confluence with Dry Gulch below Lake Estes	322,578	160.42		6,023	6,003	923	2,142	3,683	5,942	14,219	2,250	3,800	4,700	7,200				
ICC_62	BT at Loveland Heightes (Jarrett Estimation Point #62)	306,818	164.32	9,300	6,269	6,252	936	2,176	3,748	6,055	14,520								
ICC_65	BT at Mountain Shadows Lane above Drake	260,324	187.59	12,500	7,566	7,534	960	2,278	3,961	6,453	15,686	2,750	5,700	7,500	13,600				
J4080	Confluence of BT and NFBT at Drake	251,748	275.51	14,800	14,731	14,728	2,116	4,538	7,495	11,803	26,983	3,700	7,850	10,400	19,200				
J4083	BT at confluence with Cedar Creek	225,528	300.40		16,632	17,624	2,693	5,582	9,048	14,020	31,273								
ICC_66	BT at Mouth of Canyon (Jarrett Estimation Point #66)	194,636	314.03	15,500	16,876	18,106	3,041	6,249	10,054	15,449	34,002	3,800	10,500	15,300	37,000	3,208	8,942	13,533	34,145
Upper Big Thompson	Upper Big Thompson Model Inflow Hydrographs	188,356	461.14	19,000	23,957	23,957	4,529	8,582	13,012	18,920	41,780	4,700	12,300	19,000	44,000				
J1260	BT at Rossum Drive (Dry Creek Confl.)	173,552	499.31		23,523	23,684	4,332	8,383	12,941	19,021	40,536								
J1140	BT at Wilson Avenue	162,581	499.31	22,000	22,258	22,410	4,318	8,371	12,923	18,997	40,400								
J1130	BT at Taft Avenue	157,401	503.72		21,749	21,894	4,417	8,515	13,140	19,305	40,841								
J1080	BT at Railroad Avenue	154,359	530.03		21,724	21,877	4,707	8,974	13,897	20,411	42,647	4,700	12,300	19,000	44,000				
J1070	BT at Hwy 287 (Lincoln Ave.)	146,733	531.17	22,000	21,314	21,463	4,703	8,979	13,912	20,429	42,628					3,617	11,801	18,743	51,518
J1060	BT downstream of County Road 9E	126,707	546.63		20,339	20,458	4,775	9,166	14,276	21,008	43,757	4,700	12,300	19,000	44,000				
J1050	BT at I-25	115,671	576.73	19,600	19,768	19,875	5,089	9,528	14,902	21,750	45,136	4,300	8,800	11,500	21,000				
J1030	BT upstream of County Road 15 1/2	70,821	603.23		18,542	18,522	4,949	9,261	14,385	20,965	44,649	3,600	7,600	10,000	18,500				
J1010	BT at County Road 15 1/2 (Road 52 & Railroad)	65,939	607.15		18,372	18,323	4,939	9,221	14,292	20,889	44,588								
J1000B	Big Thompson Upstream of Confluence with Little Thompson	35,077	619.84	17,700	17,260	17,163	4,895	9,048	13,906	20,309	44,244	2,200	4,700	6,500	12,000				
J1000	Confluence of Big Thompson and Little Thompson	35,077	815.84		30,305	29,986	4,857	9,470	15,377	23,168	56,189	3,200	7,300	9,900	20,000				
Outlet1	BT Confluence with South Platte River	0	828.54	24,900	27,463	27,159	4,774	8,973	14,808	22,247	53,569	2,500	5,900	8,000	15,000	2,897	8,260	12,310	28,963
Little Thompson	Little Thompson Model Inflow Hydrographs	0	196.00	18,000	16,153	16,153	4,475	7,158	10,544	15,364	31,381	1,630	3,600	4,800	8,400				
Pinewood Reservoir	Pinewood Reservoir - No Discharge to Dry Creek in 2013 Flood	69,974	3.07	0	0	0	118	202	285	406	771	119	230	296	492				
J1220	Confluence of Skinner Gulch and Mill Gulch	66,886	9.03		631	502	269	497	741	1,062	2,110	755	2,125	2,923	5,186				
J1210	Cottonwood Creek at Confluence with Saddle Notch Gulch	55,875	12.13		948	762	325	619	946	1,354	2,753								
J1200	Cottonwood Creek above Flatiron Reservoir	35,977	15.54		1,292	996	463	860	1,296	1,833	3,618								
J1180	Dry Creek below Flatiron Reservoir	35,390	22.57		1,292	996	832	1,494	2,217	3,073	5,776	2,329	6,023	8,128	13,885				
J1170	Dry Creek at Hogback Ridge	27,969	25.45		1,668	1,150	999	1,776	2,620	3,607	6,710	2,403	6,224	8,425	14,440				
J1160	Dry Creek at CR 20	19,140	30.68		2,467	1,380	1,289	2,245	3,260	4,443	8,106	2,873	7,192	9,719	16,582				
J1150	Dry Creek above Confluence with BT	9,082	32.07		2,577	1,442	1,387	2,397	3,463	4,709	8,542	2,970	7,365	9,955	16,930				
R1150	Dry Creek through Golf Course	0	32.07	2,448	2,575	1,441	1,386	2,396	3,461	4,707	8,539	3,020	7,465	10,090	17,135				
Carter Lake	Carter Lake can contain PMP storm event per USBR	58,848	3.74	0	0	0	0	0	0	0	0								
J1110	Ryan Gulch at Hertha Reservoir	43,766	7.41		303	189	269	470	673	916	1,635								
J1100	Ryan Gulch upstream of Lonetree Reservoir	33,354	10.37		517	266	471	821	1,177	1,596	2,850								
J1090	Ryan Gulch upstream of BT	3,128	26.31		1,221	507	1,332	2,254	3,173	4,236	7,383								
R1090	Ryan Gulch at BT confluence	0	26.31		1,217	503	1,329	2,249	3,166	4,225	7,373								
J1290	Dry Creek above Boyd Lake	49,792	14.11		930	821	1,440	2,021	2,541	3,122	4,724								
Boyd Lake	Boyd Lake Labyrinth Spillway	14,875	27.35	0	0	0	1,712	2,803	3,720	4,731	7,570								
R1280	Boyd Lake Overflow Path	0	27.35		0	0	1,712	2,802	3,718	4,730	7,567								

Appendix D.6

Big Thompson River Peak Discharge Profile

Appendix D.6

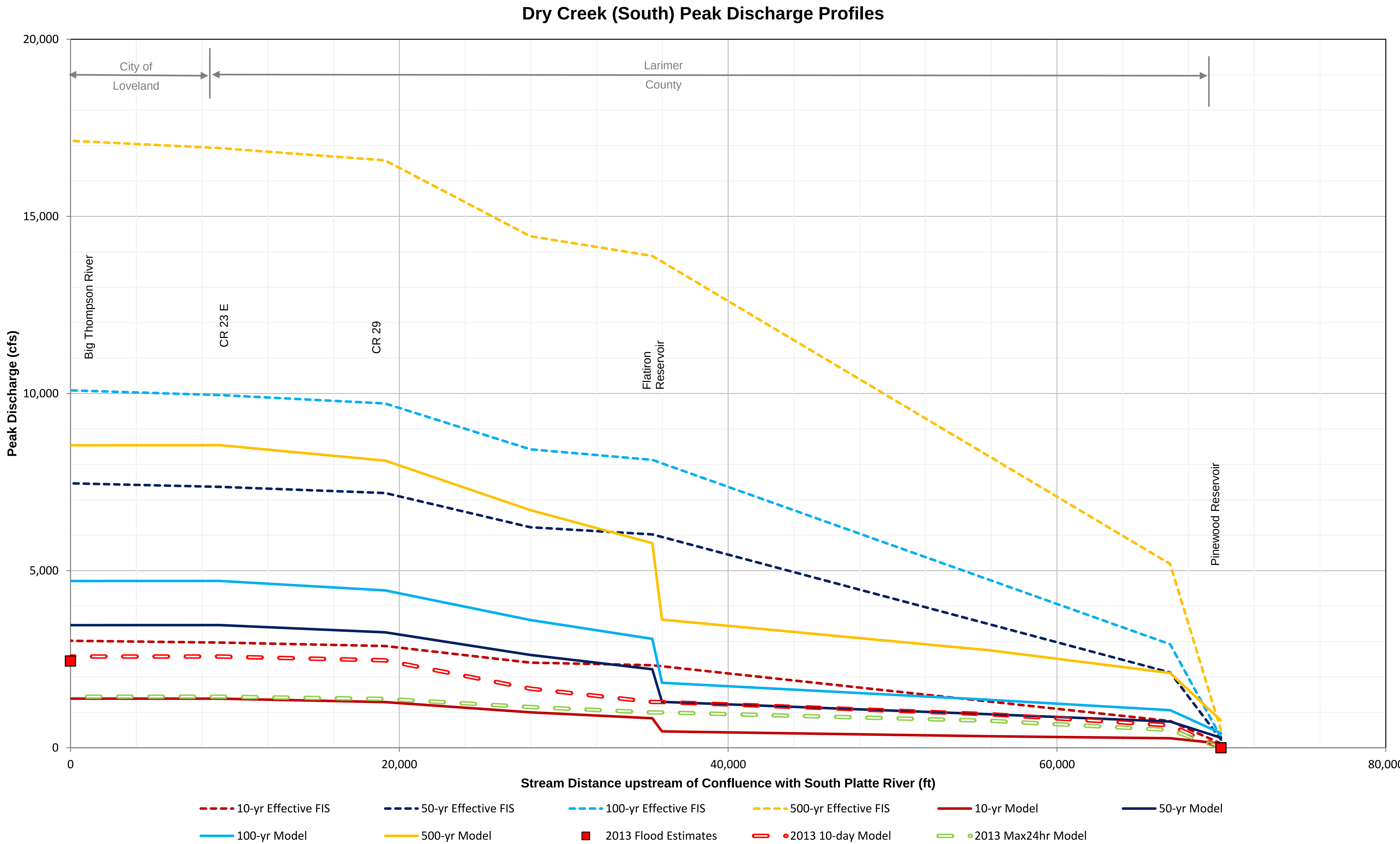
Big Thompson River Peak Discharge Profiles



Appendix D.7

Dry Creek (South) Peak Discharge Profiles

Appendix D.7



Dry Creek (South) Peak Discharge Profiles
City of Loveland
Larimer County
Big Thompson River
CR 23 E
CR 29
Flatiron Reservoir
Pinewood Reservoir
Peak Discharge (cfs)
20,000
15,000
10,000
5,000
0
0
20,000
40,000
60,000
80,000
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
2013 Flood Estimates
2013 10-day Model
2013 Max24hr Model

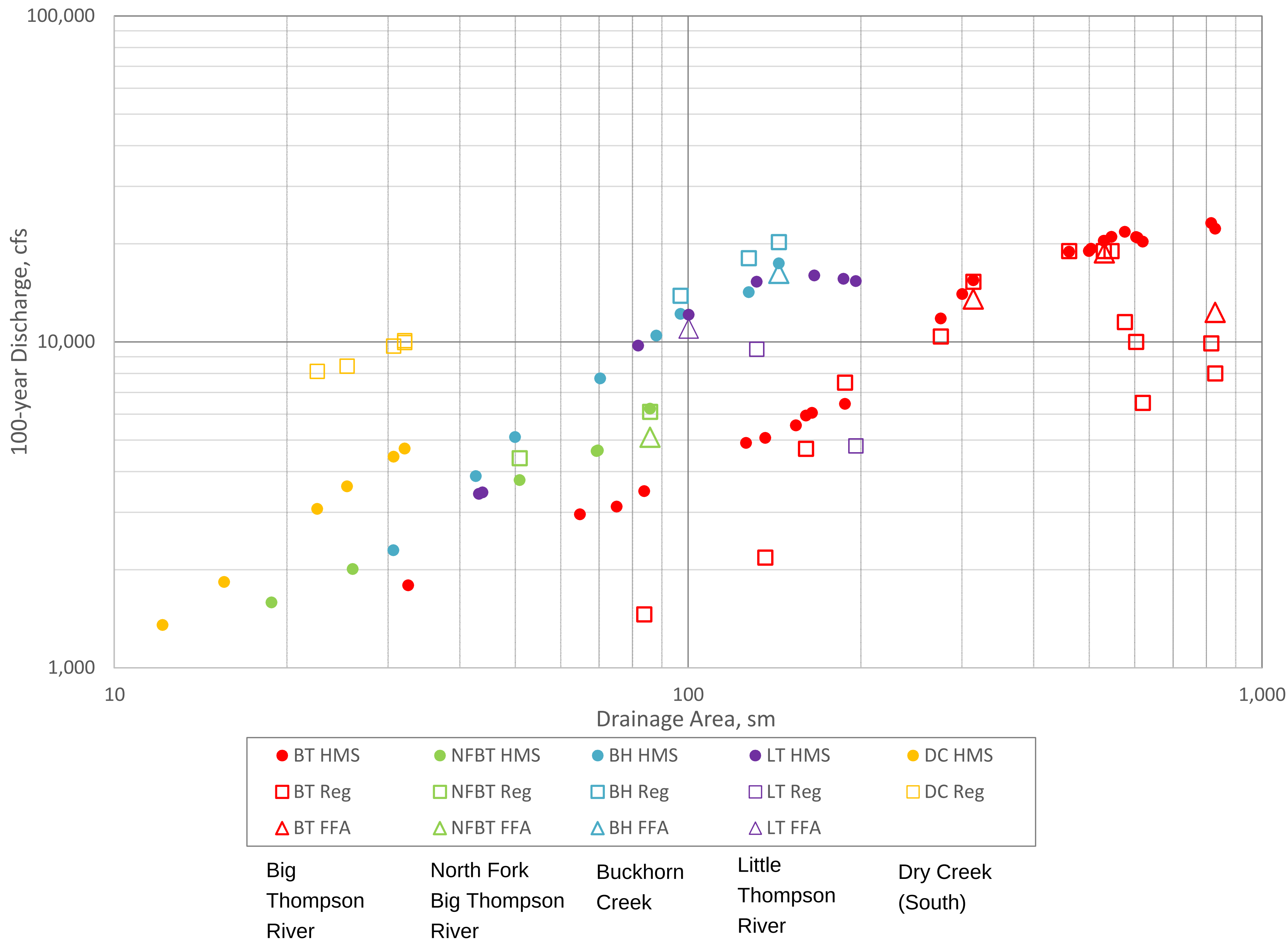
Appendix D.8

Discharge Comparison Plots

Big Thompson Watershed

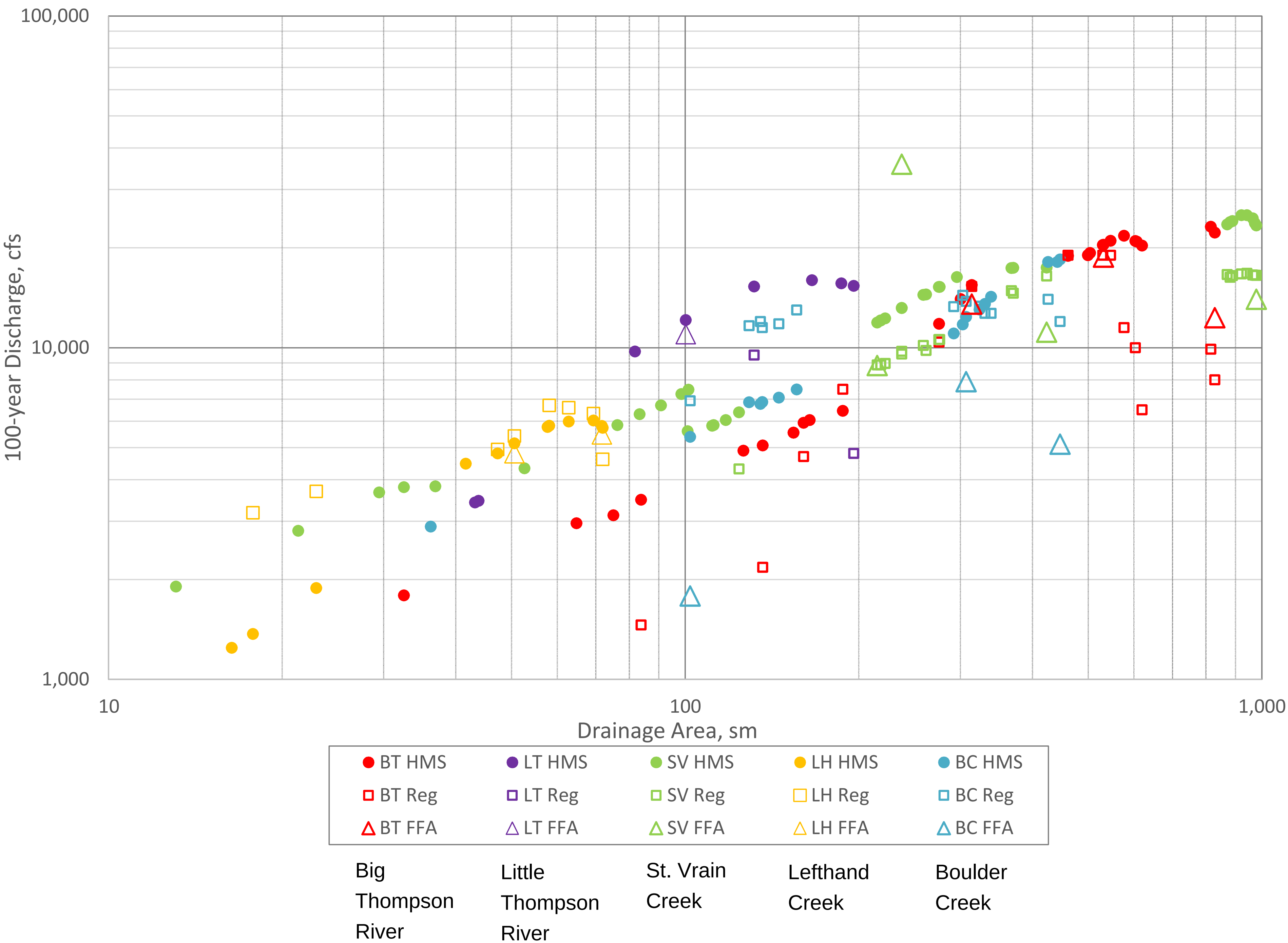
Big Thompson and St. Vrain Watersheds

Appendix D.8



100,000
10,000
1,000
10
100
1,000
100-year Discharge, cfs
Drainage Area, sm
BT HMS
NFBT HMS
BH HMS
LT HMS
DC HMS
BT Reg
NFBT Reg
BH Reg
LT Reg
DC Reg
BT FFA
NFBT FFA
BH FFA
LT FFA
Big Thompson River
North Fork Big Thompson River
Buckhorn Creek
Little Thompson River
Dry Creek (South)

Appendix D.8 (continued)



Appendix D.9

Calibrated Model Comparison Plots

Pinewood Reservoir

Flatiron Reservoir

Dry Creek at Marianna Butte Golf Course

Big Thompson River at Glade Road

Big Thompson River at Namaqua Road

Big Thompson River at Wilson Avenue

Big Thompson River at Highway 287

Big Thompson River at Loveland Gage

Boyd Lake

Big Thompson River at I-25

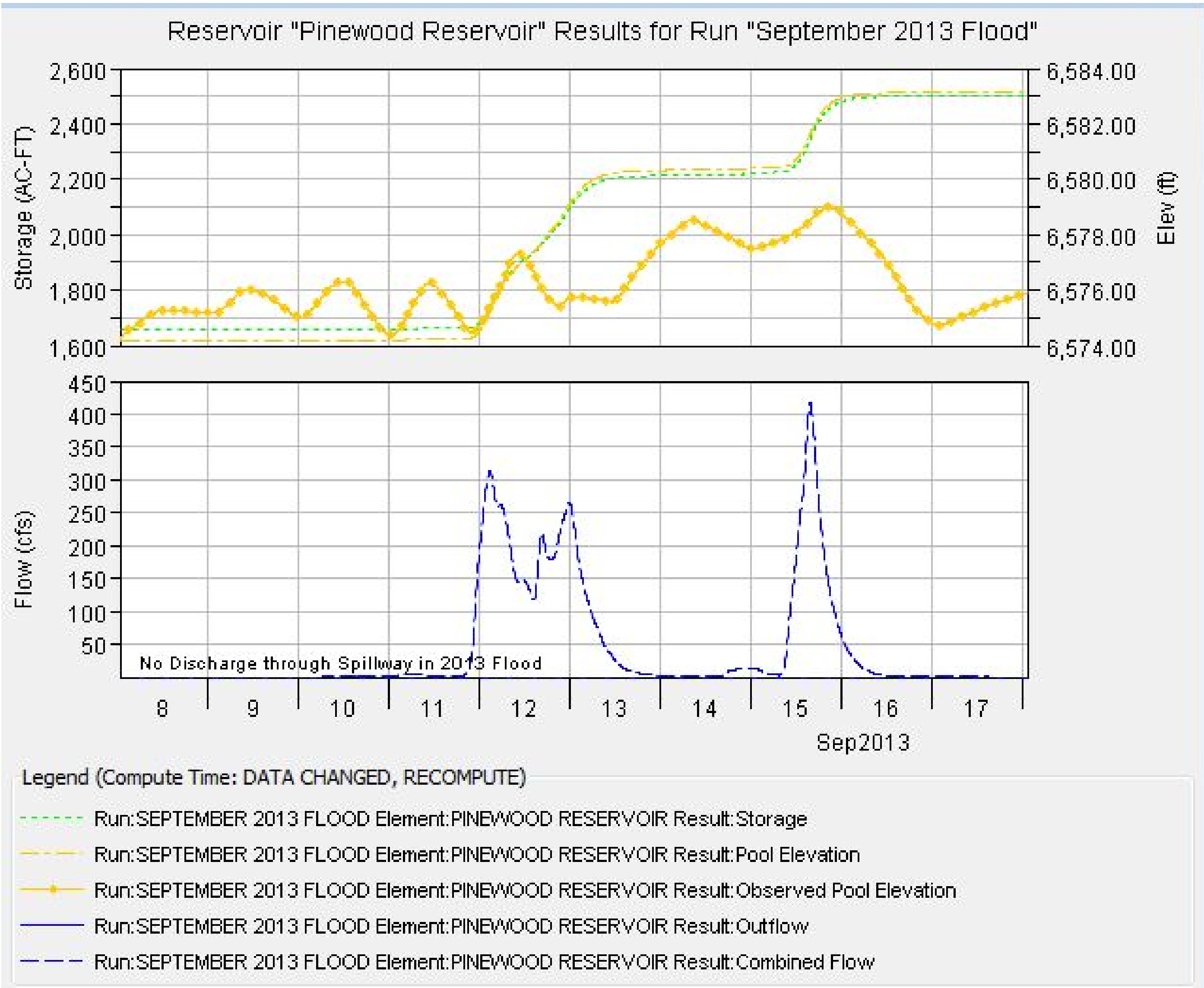
Big Thompson River at Highway 257

Big Thompson River at La Salle Gage

Big Thompson River at County Road 27 1/2

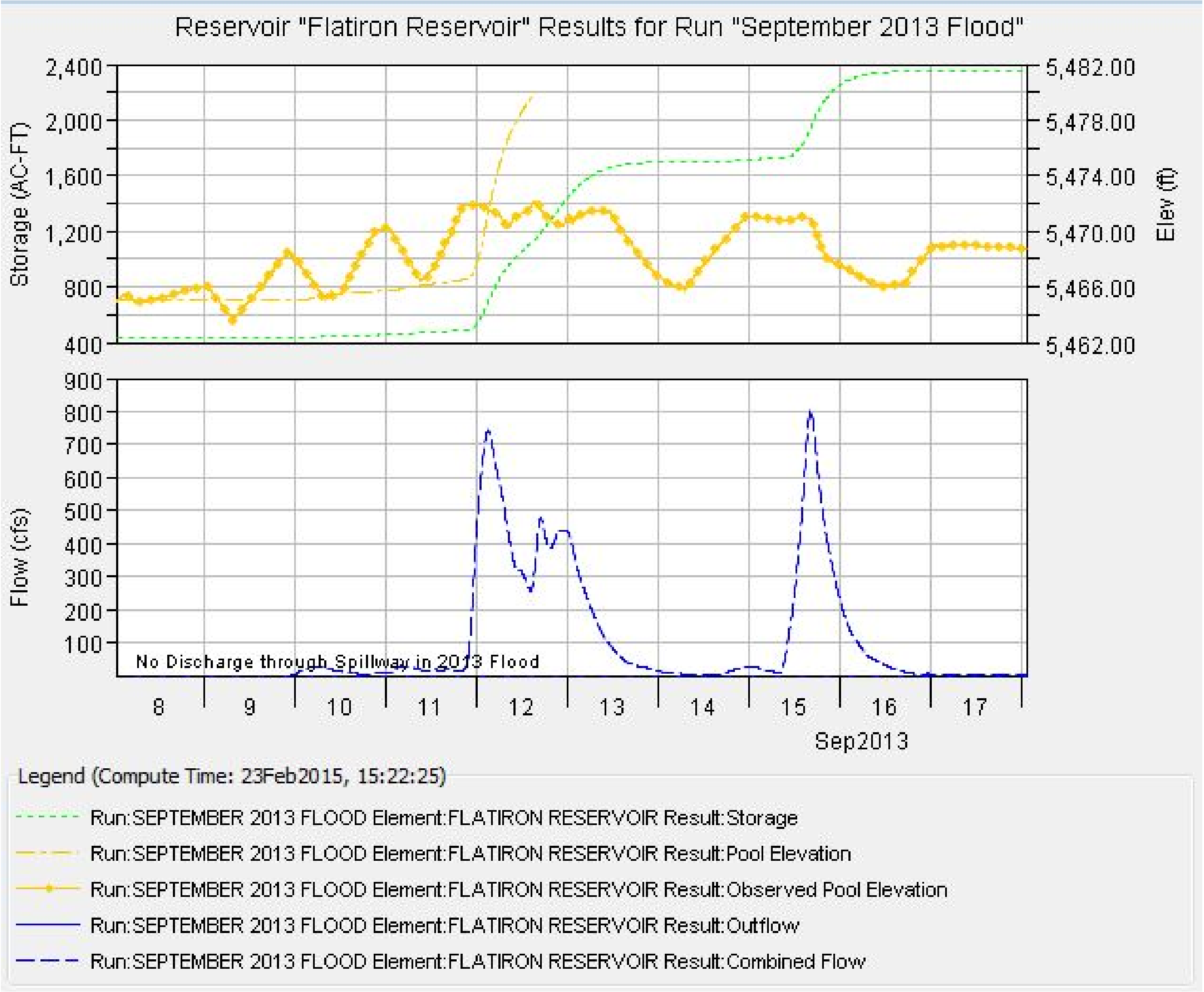
Appendix D.9

Pinewood Reservoir - Observed sawtooth pattern due to Colorado-Big Thompson Project Operations



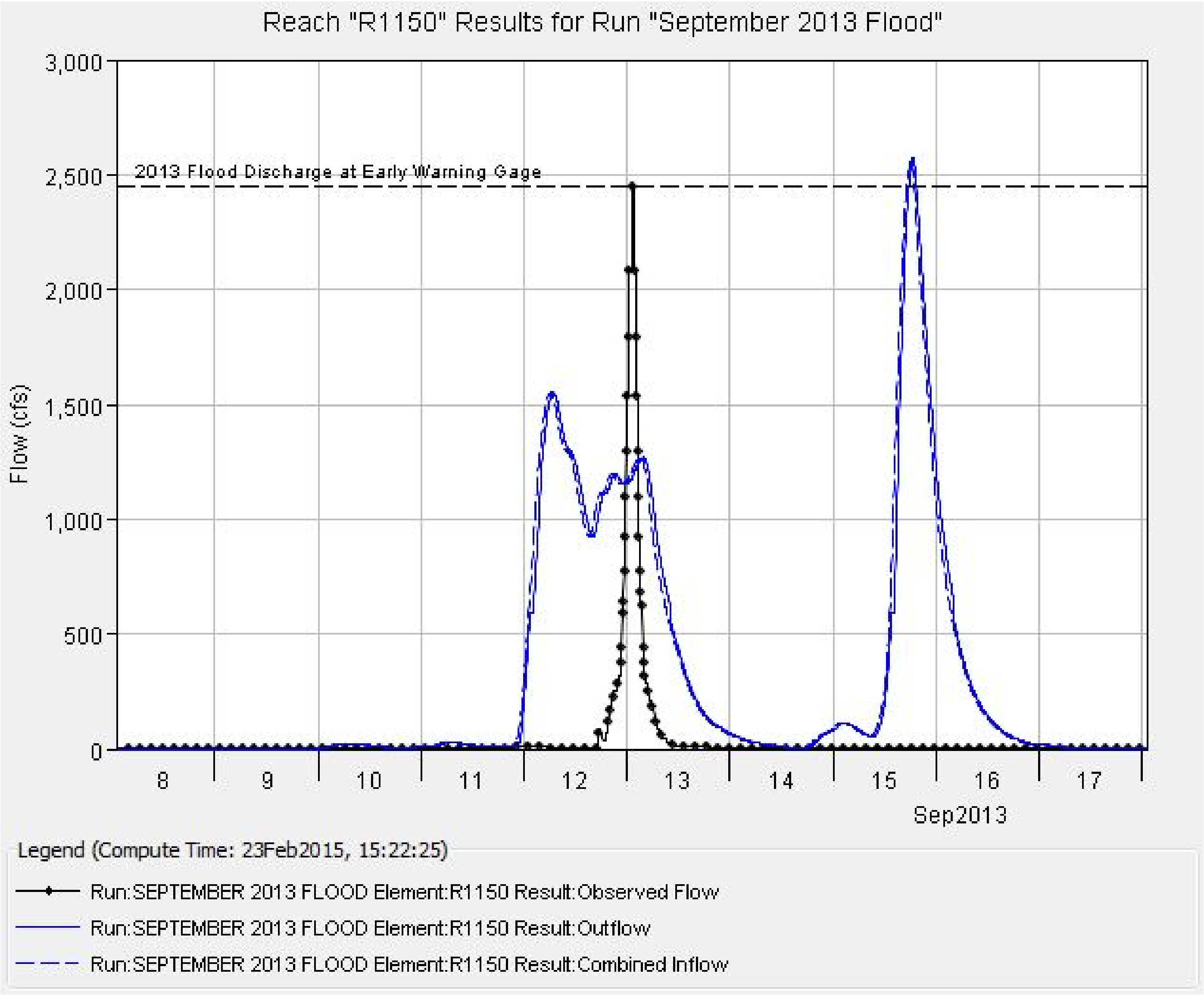
Appendix D.9 (continued)

Flatiron Reservoir – Observed sawtooth pattern due to Colorado-Big Thompson Project Operations



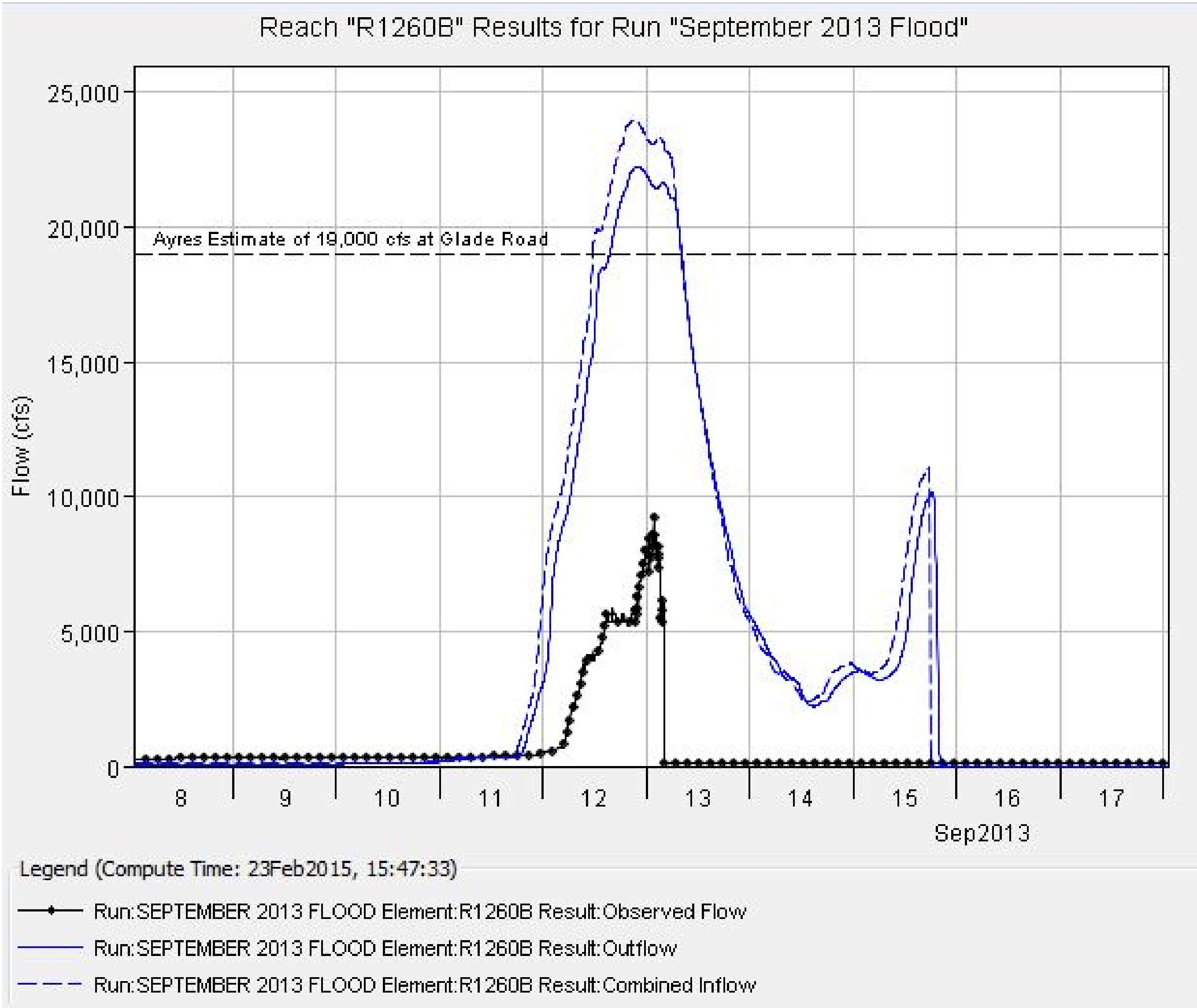
Appendix D.9 (continued)

Dry Creek – Early Flood Warning Gage at Marianna Butte Golf Course



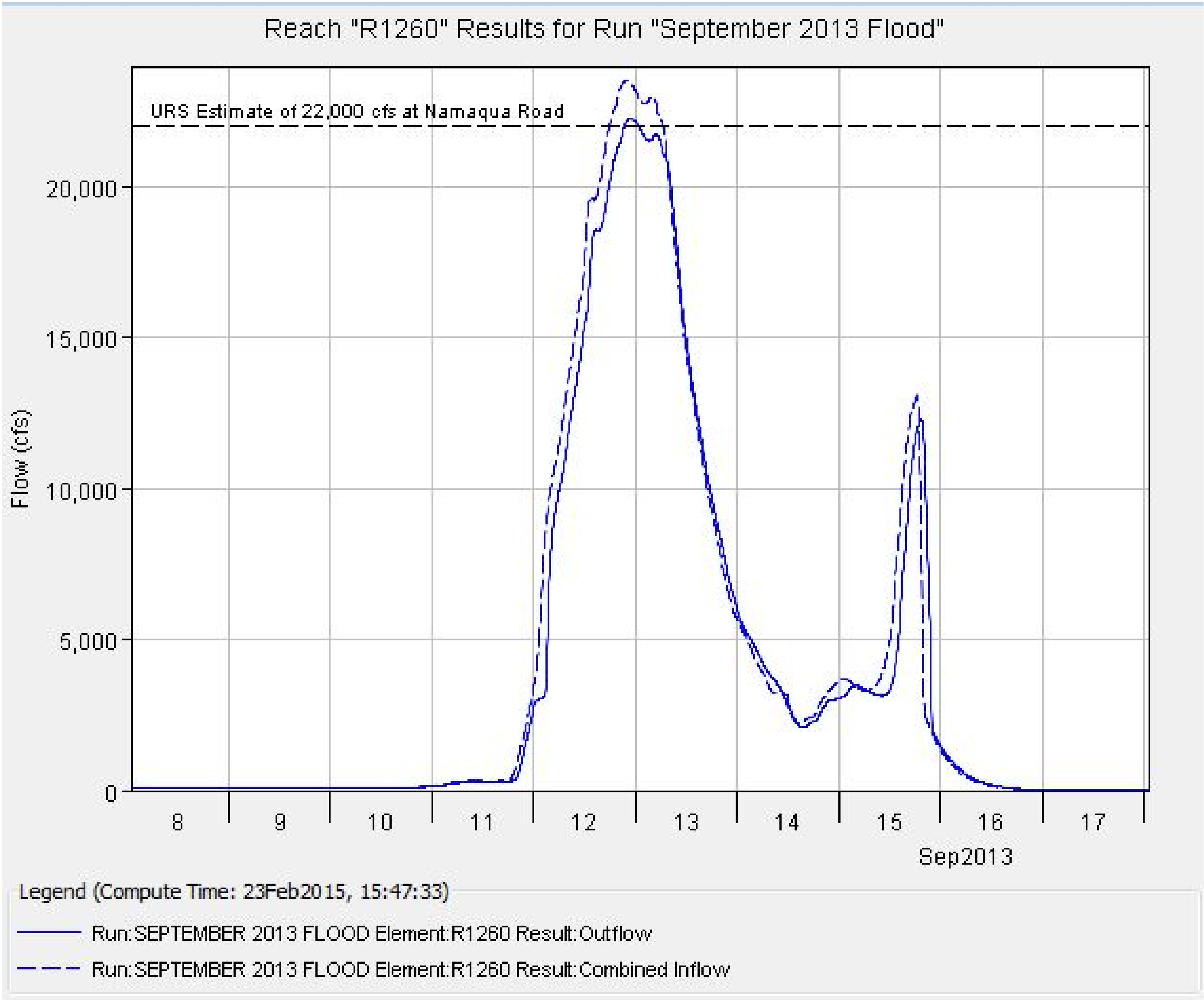
Appendix D.9 (continued)

Big Thompson River – Early Flood Warning Gage at Glade Road



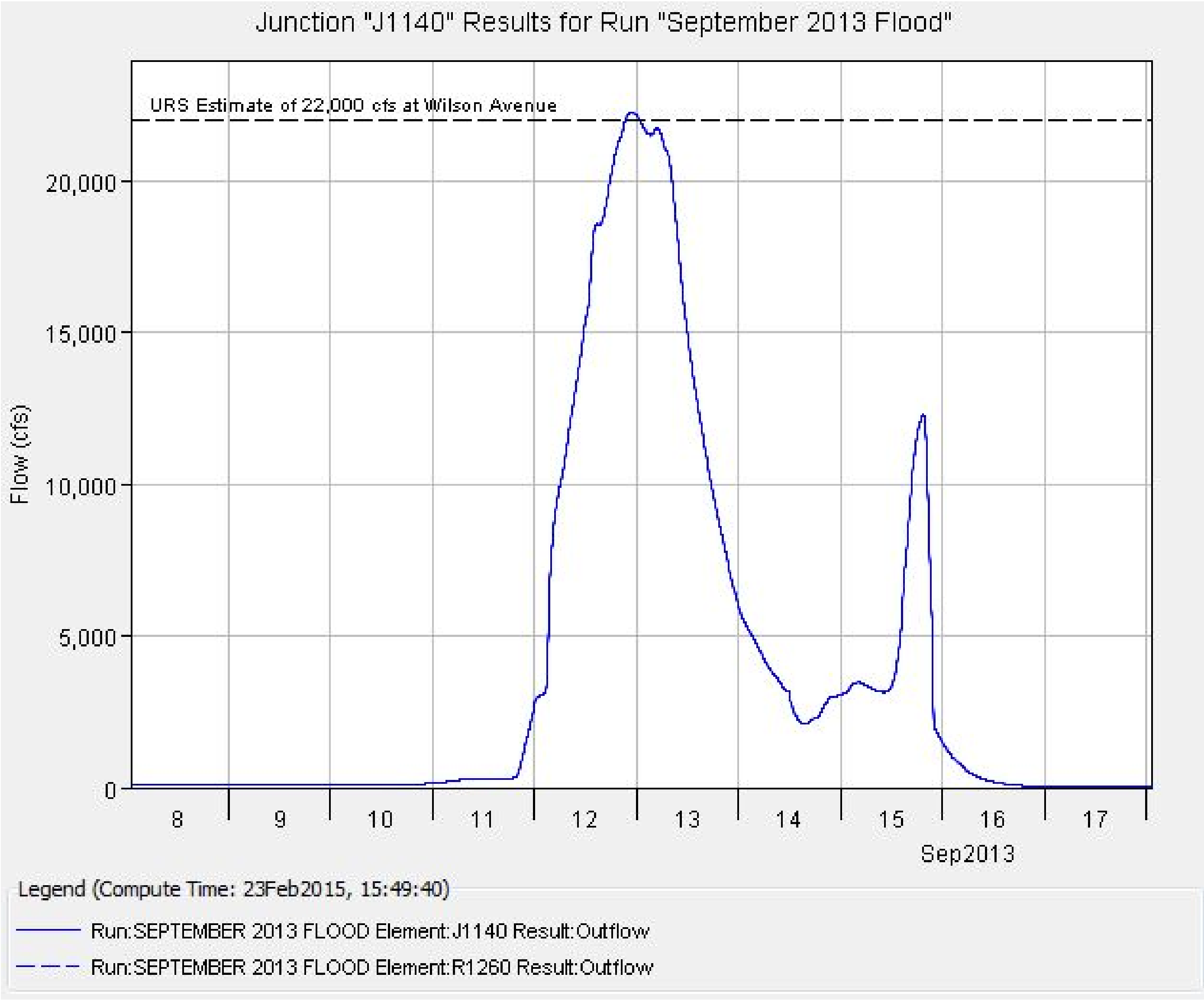
Appendix D.9 (continued)

Big Thompson River – Namaqua Road



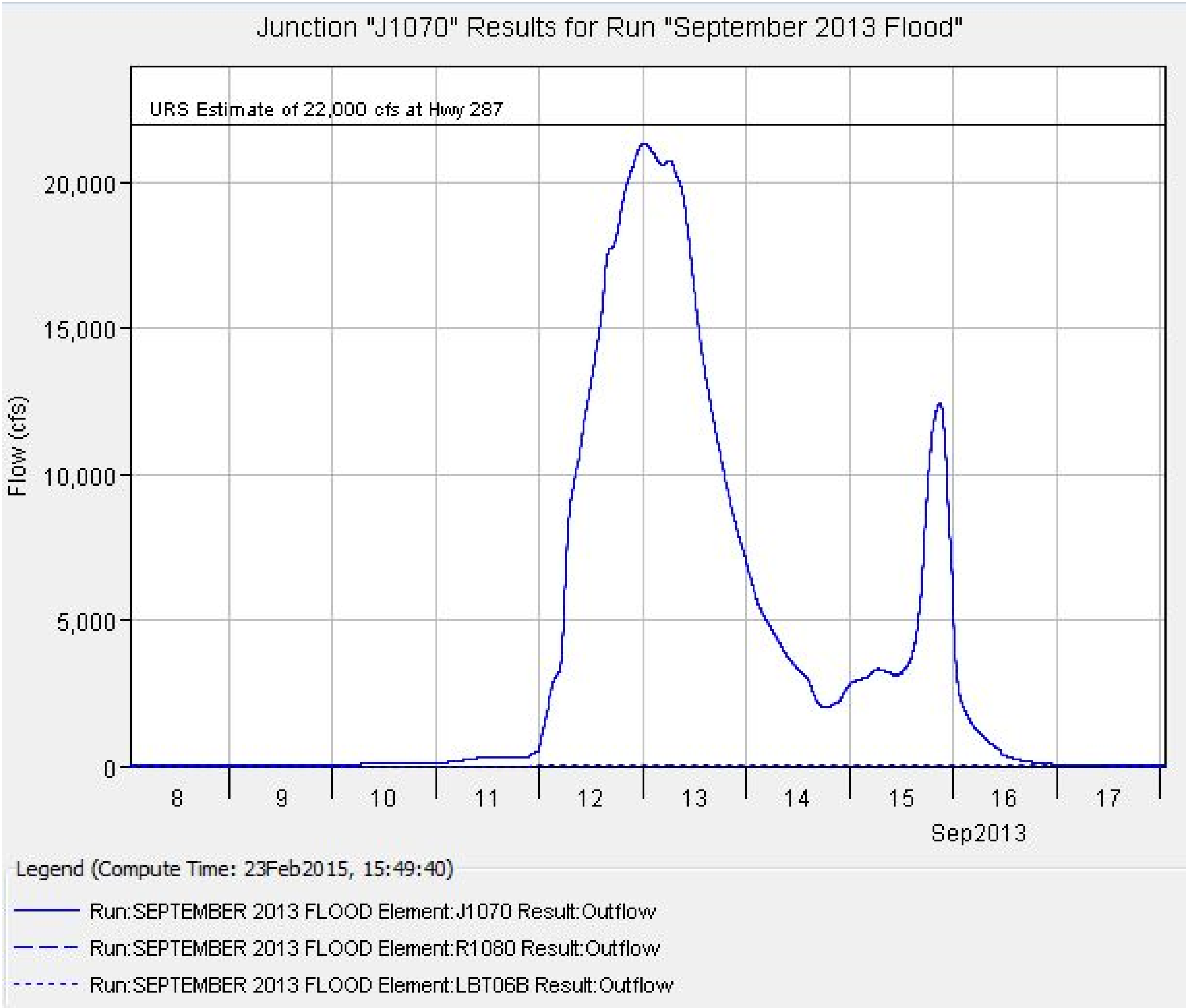
Appendix D.9 (continued)

Big Thompson River – Wilson Avenue



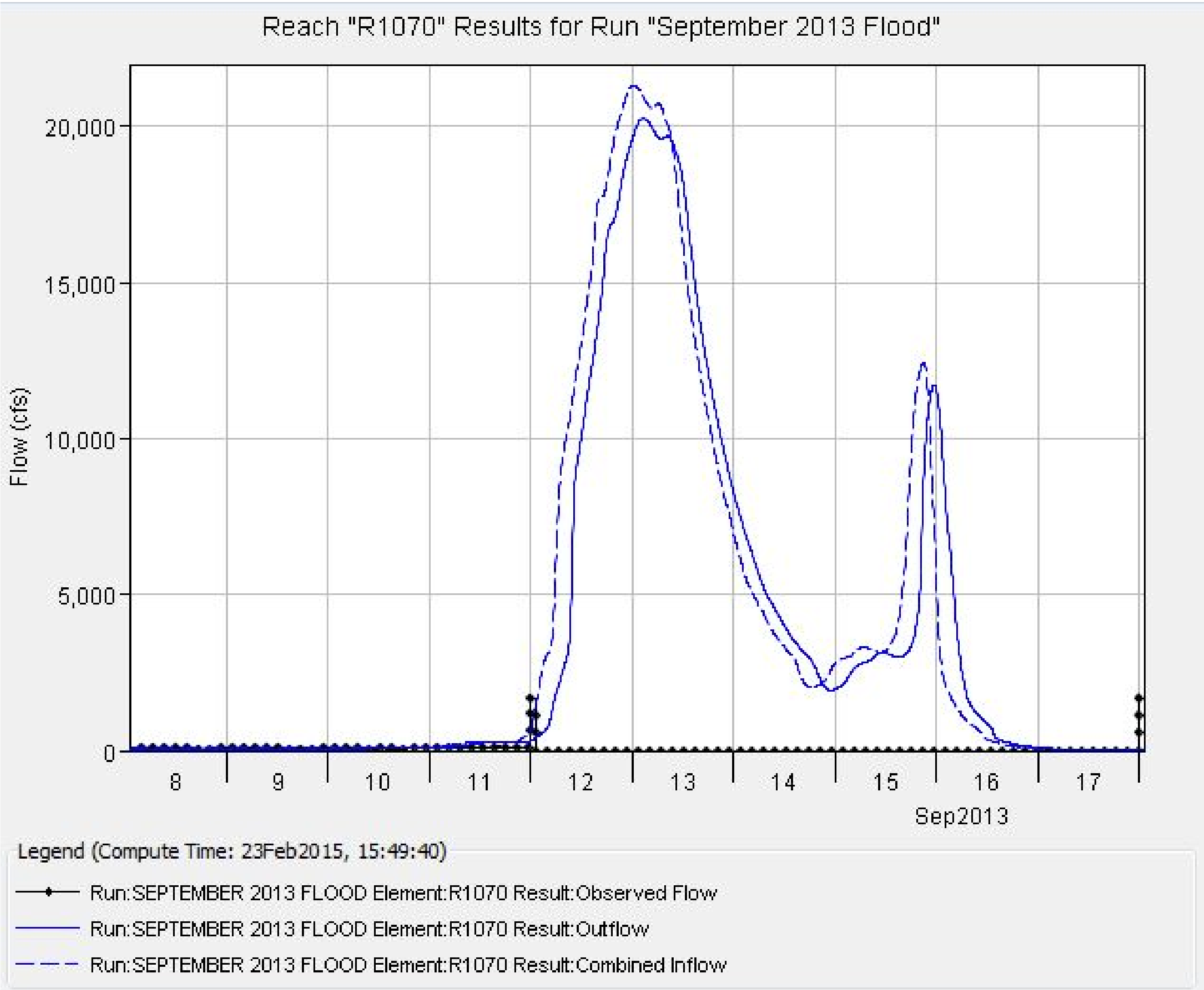
Appendix D.9 (continued)

Big Thompson River – Highway 287



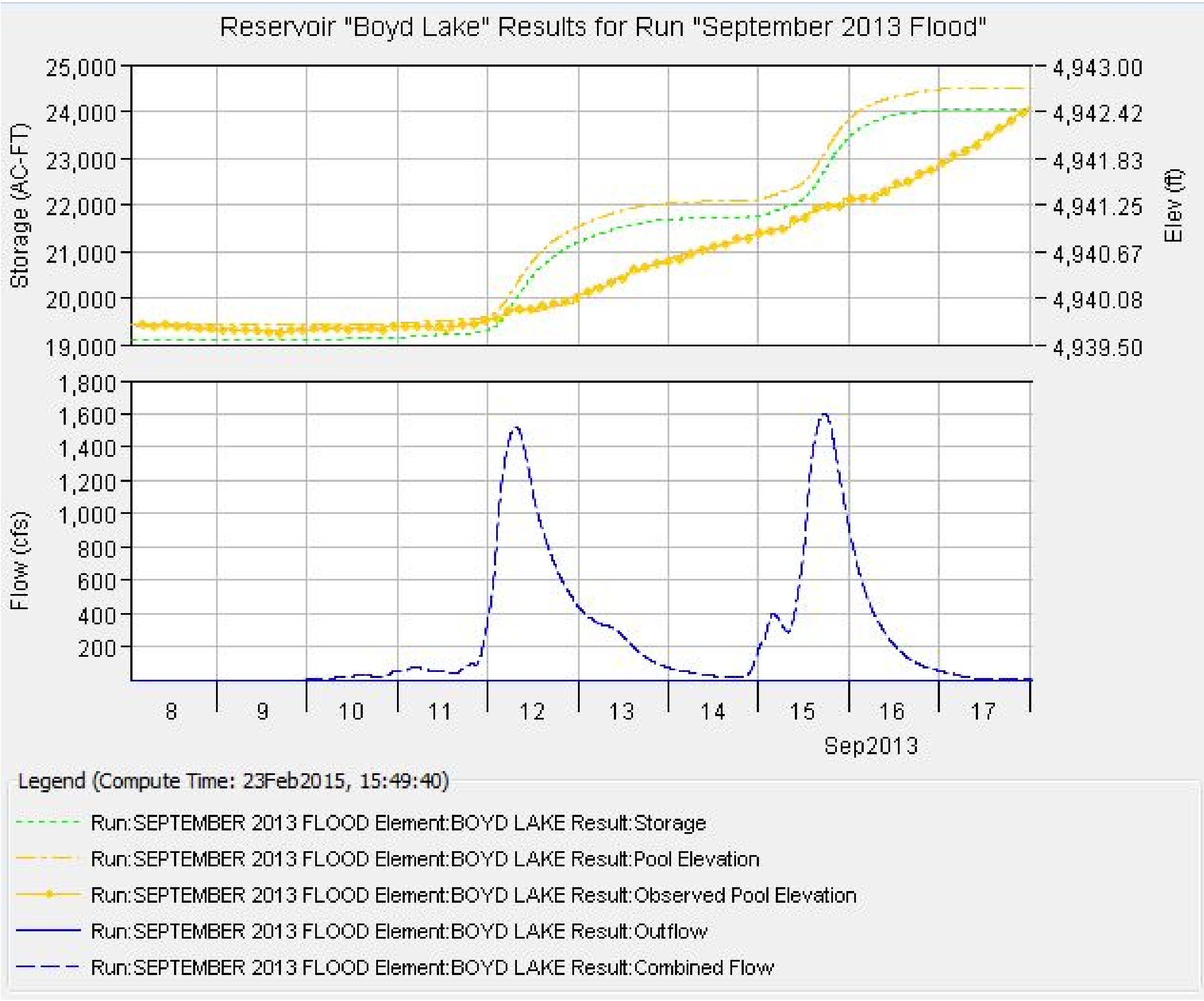
Appendix D.9 (continued)

Big Thompson River – Loveland Gage, USGS 06741510, CDWR BIGLOVCO



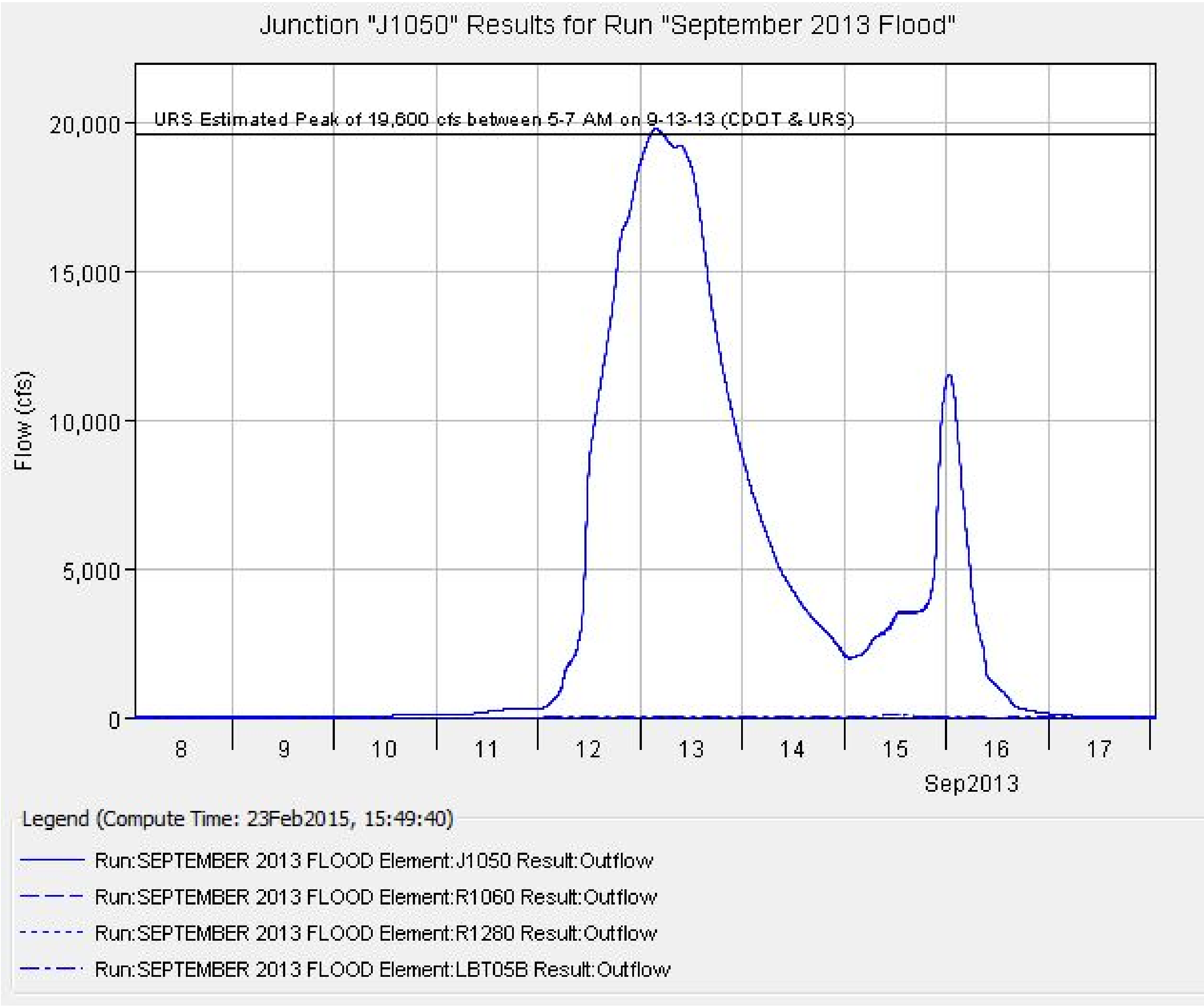
Appendix D.9 (continued)

Boyd Lake – Loveland & Greeley Irrigation Company



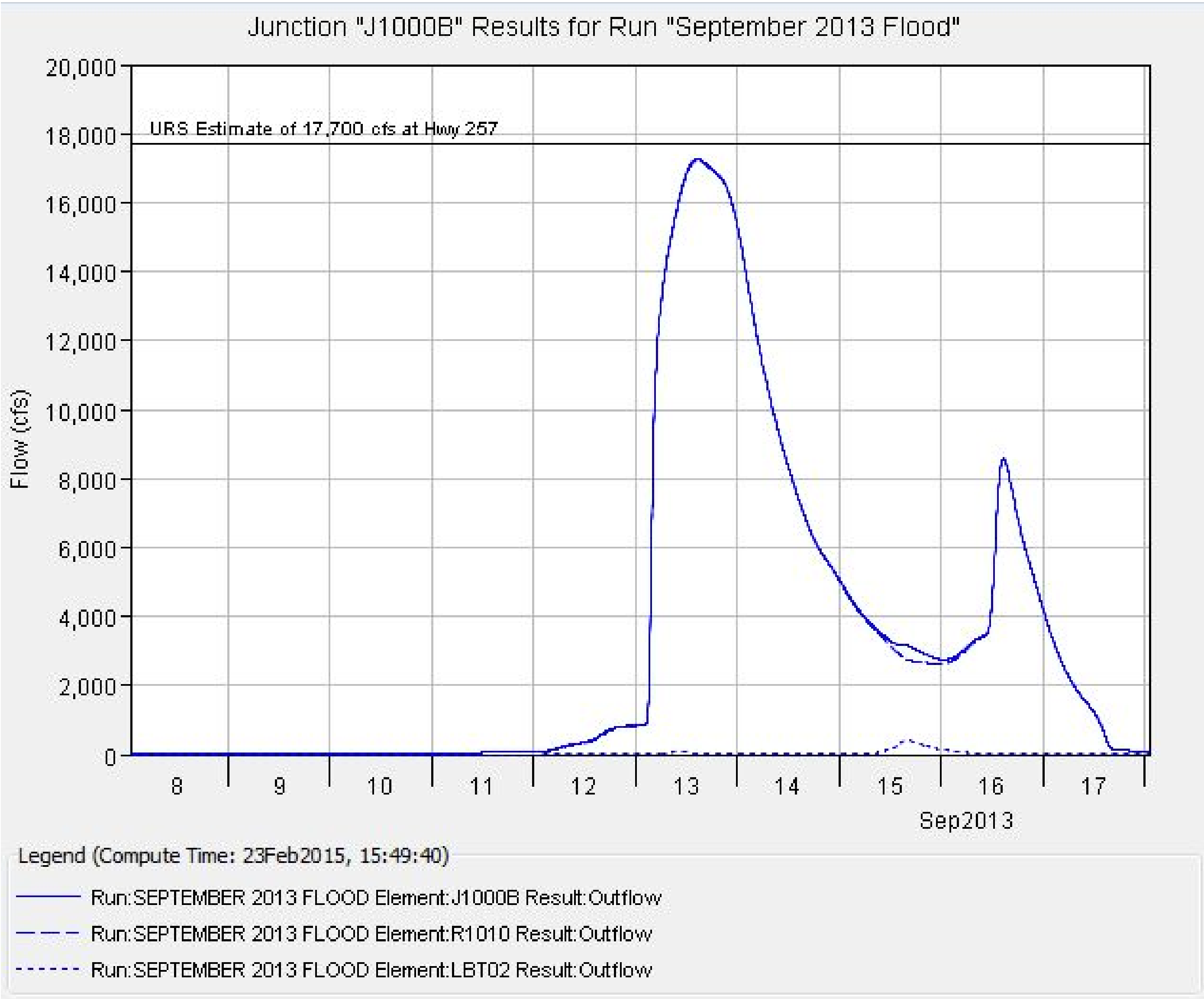
Appendix D.9 (continued)

Big Thompson River – I-25



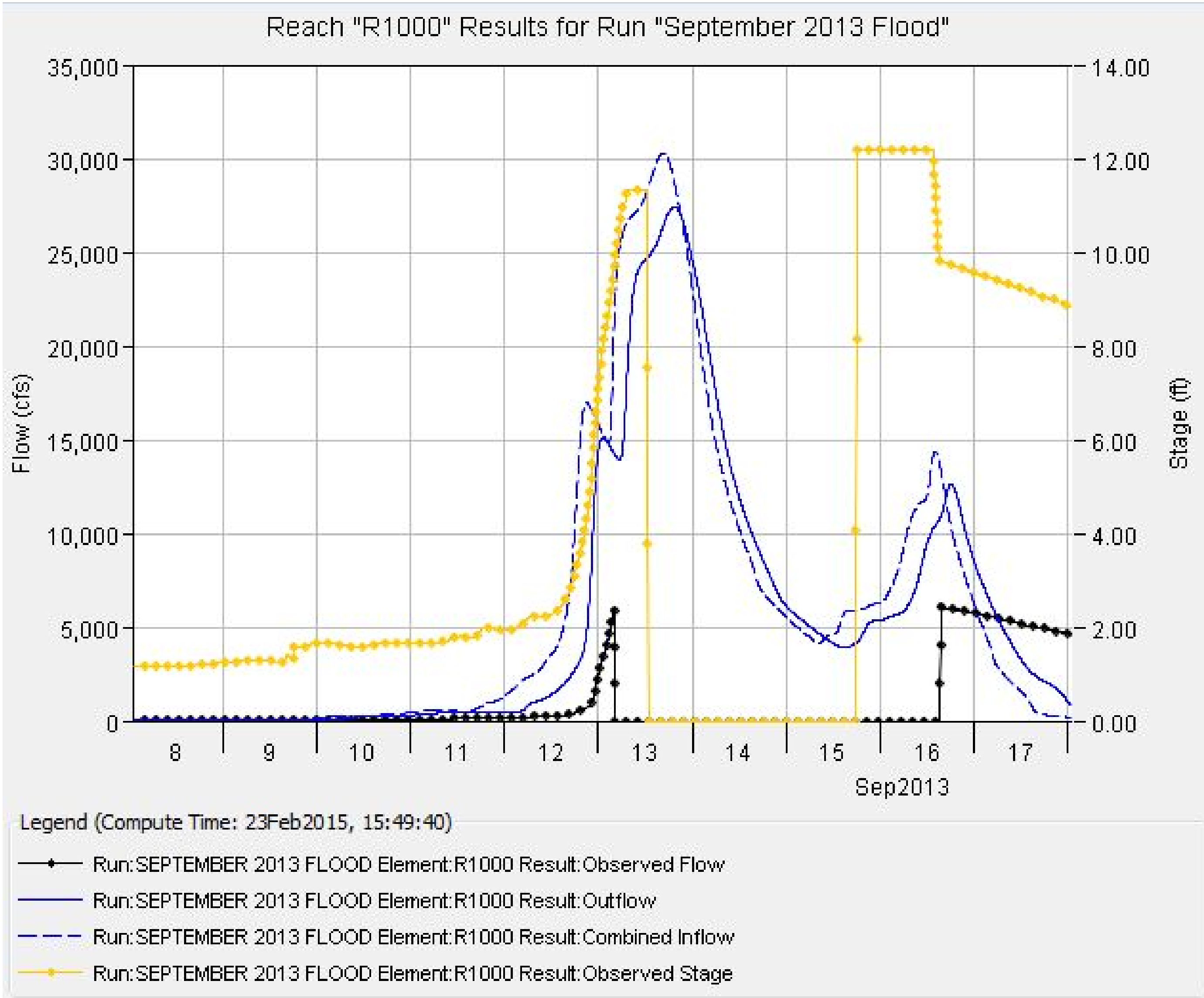
Appendix D.9 (continued)

Big Thompson River – Highway 257



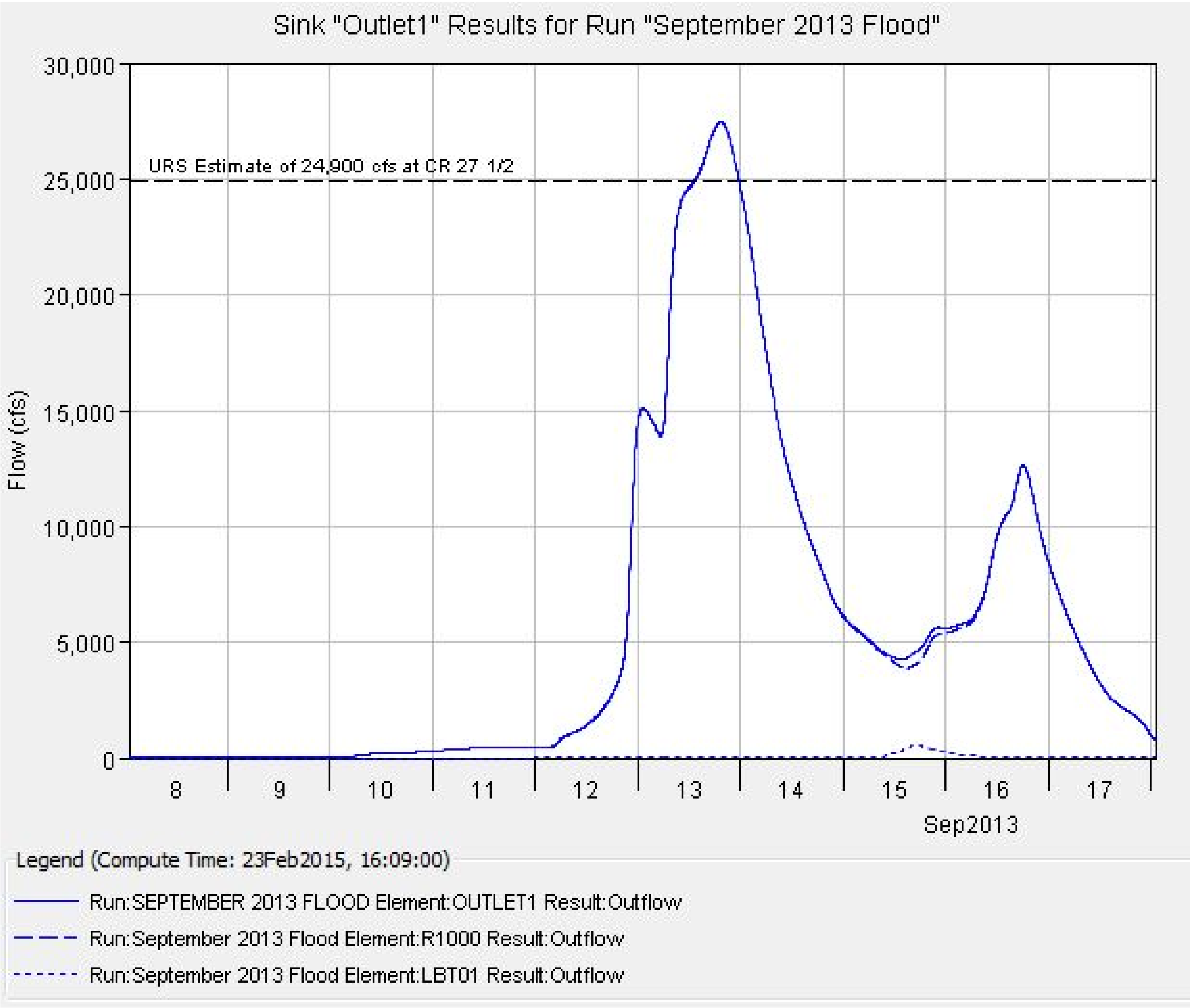
Appendix D.9 (continued)

Big Thompson River – La Salle Gage, USGS 06744000, CDWR BIGLASCO



Appendix D.9 (continued)

Big Thompson River – County Road 27 1/2



Appendix E

Little Thompson Watershed Calibration

Little Thompson River 10 Day Calibration Documentation

PREPARED FOR: Colorado Department of Transportation

COPY TO: Colorado Water Conservation Board

Jacobs Engineering Group

PREPARED BY: CH2M HILL

DATE: February 16, 2015

PROJECT NUMBER: 494613

This memorandum documents the development and calibration of a 10-day hydrologic model of the Little Thompson River above its confluence with the Big Thompson River. The hydrologic model was calibrated to a 10-day period encompassing the September 2013 Front Range rainfall event peak discharge estimated collected along the Little Thompson River. The hydrologic model and resultant output hydrograph were developed for use as input to the Big Thompson River calibrated hydrologic model by others.

Hydrologic Analysis

Project Area Description

The Little Thompson River watershed was modeled in two phases. Phase 1 in the uppermost portion of the Little Thompson River watershed has a total of 18 subbasins totaling 43.8 square miles. This portion of the model was completed in August 2014 (CDOT, 2014). Phase 2 extends to the confluence with Big Thompson River. Phase 2 consists of 13 additional subbasins totaling 152.6 square miles for a total watershed study area of 196.4 square miles. The increase in the scale of the project and the changes in topography and land use allowed the average size of subbasin to increase as the model was extended from Phase 1 to Phase 2. Phase 1 consists entirely of mountain topography while Phase 2 consists of mountain and plains topography (Phase 2 Mountains and Phase 2 Plains). **Figure 1** shows the location of the Little Thompson River watershed and the extent of Phase 1 and 2.

Overall Modeling Approach

U.S. Army Corps of Engineers' (USACE's) Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) version 3.5 (USACE, 2010) was selected to model the hydrologic conditions within the Little Thompson River 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. A calibrated hydrologic model was developed to model the September 2013 event. Hydrologic conditions unique to the September 2013 event (e.g., measured rainfall) were used to calibrate remaining model parameters to match modeled peak discharges to observed peak discharges observed following the September 2013 event by indirect measurements. **Figures 2a through 2c** depicts the HEC-HMS model components for Phase 2 of the Little Thompson River model.

Calibration Data

Peak flow estimates were provided by several sources: Applied Weather Associates (AWA) and its subconsultant Bob Jarrett (Jarrett, in press), URS (URS,2015), and Colorado Division of Water Resources Dam Safety Branch (CDWR, 2014) to estimate peak discharges for the Little Thompson River watershed as summarized in **Table 1**. These locations are shown in **Figure 1**.

Table 1
Little Thompson Physical-based Peak Flow Observations

Model Phase	Site Description	Calibration Source	Peak Discharge (cfs)
Phase 1	Little Thompson River Midpoint of Watershed	Jarrett, In Press	2,470
Phase 1	Little Thompson River Upstream of Confluence with West Fork Little Thompson River	Jarrett, In Press	2,680
Phase 1	Little Thompson River Downstream of Confluence with West Fork Little Thompson R.	Jarrett, In Press	7,800[a]
Phase 1	West Fork Little Thompson River Upstream of Confluence with Little Thompson River	Jarrett, In Press	6,200
Phase 2	Little Thompson River at X Bar 7 Ranch	CDWR, 2014	15,731
Phase 2	Little Thompson River at South County Line Road	URS, 2015	13,400
Phase 2	Little Thompson River at Interstate 25	URS, 2015	15,700
Phase 2	Little Thompson River at County Road 17	URS, 2015	18,000[b]

[a] - This flow was inaccessible and the observed peak discharge was estimated based on observations along similar, adjacent watersheds.

[b] – Bridge overtopped, (URS, 2015)

cfs = cubic feet per second

The Little Thompson River at Interstate 25 was observed by the CDOT to peak in the afternoon of September 12th for the September 2013 flood event (CDOT, In Press).

Subwatershed Areas

The Little Thompson River Study area upstream of the confluence with the Big Thompson River was delineated using 31 subbasins with drainage areas as shown in **Table A-1 in Appendix A**. Phase 1 consists entirely of mountain topography (LT-1 through LT-5 and WF-1 through 6). Phase 2 consists of with a few sub-basins with mountain topography (LT-6 through LT-8 and NF-1 through NF3) while the remaining subbasins (DC-3 and LT-9 through LT-14) have plains topography.

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 measurement collected during the event to the centroid of each subbasin. The total rainfall for each subbasin is depicted in **Figure 3** for Phase 2. There was significantly more rainfall in the mountain region during this storm event. The Phase 1 and Phase 2 Mountain subbasins received on average 12.6 inches over the 10 days as compared to an average of 5.6 inches for Phase 2 Plains subbasins.

Loss Method

Consistent to the Phase I hydrologic model, the NRCS (formerly SCS) method was selected to convert input rainfall to infiltration losses and runoff. Antecedent moisture condition (AMC) II was used to represent “normal” conditions. Technical Release 55: Urban Hydrology for Small Watershed provided Curve Numbers (CNs) based on land cover description and hydrologic soil group. Two GIS-based data sources, *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 scale. Delineation of hydrologic soil groups was accomplished using the USDA’s Web Soil Survey (USDA, 2013). After comparison to recent aerials, the land cover was adjusted in urbanized areas to account for development since 2006. 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.

Aerial review indicated that subbasins LT-10 and LT-11 had several large reservoirs that have storage impacts greater than what would be calculated using an initial abstraction ratio of 0.2. In these two subbasins, the open water areas were removed from the CN calculations and basin areas. As a result, the Little Thompson River watershed was reduced in size from 196.4 square miles to 194.6 square miles. The Phase 1 and Phase 2 Mountain subbasins had an average CN of 64 while the Phase 2 Plains subbasins had an average CN of 82.

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, Cp, and the lag time representative of the time elapsed between the centroid of a hyetograph and the peak of resultant hydrograph. Snyder's Cp can range from 0.4 to 0.8. 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, L_c 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 NED digital elevation model (USGS, 2013). The K_n parameter was assigned to values between 0.08 and 0.15 depending on the land use along the flow path (Table CH9-T505, CWCB, 2008). The parameter C_p was varied during the calibration process. Lag times for each individual subbasins are provided in **Table A-2 in Appendix A**. The subbasins with plains topography had longer lag times as compared to subbasins with mountain topography. Phase 2 Plains subbasins had an average lag time of 3.5 hours, and Phase 1 and Phase 2 Mountain subbasins had an average lag time of 2.4 hours.

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. The location of the Phase 2 model reach locations are provided in the connectivity maps **Figures 2a through 2c** and the model eight-point cross sections are provided in **Figure 4a and 4b**. 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 U.S. Geologic survey National Elevation Dataset (NED) 1/3 arc-second data (USGS, 2013) was utilized to develop cross sections along the West Fork Little Thompson River and post flood LiDAR (NOAA, 2013) was used to develop cross sections along the North Fork and Little Thompson River. 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. Civil Air Patrol photos taken immediately after the 2013 storm confirmed that the floodplain was approximately 500 to 2,500 feet wide during the storm event for subbasins in Phase 2 Plains subsection.

For Phase 2 Plains subbasins, a Manning's roughness value of 0.1 was used for the main channel and overbank due to heavier amounts of brush along the waterway as indicated by review of aerial photographs (Chow, 1959). The subbasins in Phase 2 with mountain topography (Phase 2 Mountains) were assigned an intermediate value of 0.08, which was used for the main channel and overbank as a transitional value between the mountains subbasins in Phase 1 and Phase 2 Plains subsections of the model.

Calibration Process

Model calibration is the iterative process of adjusting model parameters so that simulated results match real-world observations (measurements). Several parameters were varied throughout the calibration process. 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 flow paths.

Calibration of the model should also consider the sensitivity of model results to 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. In general, model parameters were adjusted in groups according to the three main sections of the model: Subbasins in Phase 1, Subbasins in Phase 2 Mountains, and Subbasins in Phase 2 Plains. As a result of this process, the following assessment of the sensitivity of the model results to the following parameters were made:

- *Snyder’s Peaking Factor:* No effect on modeled runoff volume; moderate effect on modeled peak discharge and time-of-peak. Decreased Snyder’s peaking factor lengthened the duration of the hydrograph, decreased peak discharge, and resulted in later time-of-peak discharge.
- *CN:* Only parameter that affected modeled runoff volume; significant effect on modeled peak discharge and negligible impact on time-of-peak discharge.

The Snyder peaking coefficient, Cp, was varied from 0.4 to 0.8 and the values settled on the lower range of 0.4 for all sections of the model.

The curve number land cover condition adjustment was needed to increase the discharge at several calibration points. CNs for each of the subbasins were adjusted iteratively from “fair” condition to “poor” condition until modeled discharges matched peak discharge estimates for the September 2013 event.

The final model parameters after calibrating the model to the 10 day September 2013 storm event are provided in **Table A-3 in Appendix A**.

Results

Comparison of modeled discharge to observed discharges is provided in **Table 2**. The calibrated 10 day model had a time-of-peak at LT-J12 (Little Thompson River at Interstate 25) of 2:15 p.m. on September 12, 2013 which compares favorably to CDOT’s estimate of the afternoon of September 12, 2013. As shown in **Table 2**, the percent difference between the observed peak discharge estimates and calibrated model estimates are less than 20% at all comparison locations. The discharge hydrograph at the Phase 2 Outfall location (Little Thompson River Upstream of Confluence with Big Thompson) is presented in **Figure 5**. The Little Thompson River modeling results are presented in **Table A-4 in Appendix A**.

Table 2

Little Thompson River Comparison of 10 Day Modeled Discharges to Observed Discharges

Site Number	HMS Node	Location	Drainage Area (sq miles)	Observed Peak Discharge (cfs)	Modeled Peak Discharge (cfs)	% Difference	Runoff Volume (in)
#61	LT-J3	Little Thompson River Midpoint of Watershed	13.8	2,470	2,119	-14%	6.08
#59	LT-J4 Without WF	Little Thompson River Upstream of Confluence with West Fork Little Thompson River	17.8	2,680	2,653	-1%	6.30
#60	LT-J4	Little Thompson River Downstream of Confluence with West Fork Little Thompson River	43.2	7,800[a]	8,639	11%	8.08
#64	LT-J4 Without LT	West Fork Little Thompson River Upstream of Confluence with Little Thompson River	25.4	6,200	6,077	-2%	9.33
N/A	LT-J6	Little Thompson River at X Bar 7 Ranch	81.8	15,731	13,196	-16%	7.72
LT-2	LT-J10	Little Thompson River at South County Line Road	131.2	13,400	15,453	15%	6.41
LT-3	LT-J12	Little Thompson River at Interstate 25	163.6	15,700	15,996	2%	5.78
LT-4	LT-J13	Little Thompson River at County Road 17	184.7	18,000[b]	16,179	-10%	5.55

[a] - This flow was inaccessible and the observed peak discharge was estimated based on observations along similar, adjacent watersheds.

[b] – Bridge overtopped, (URS, 2015)

cfs = cubic feet per second

References

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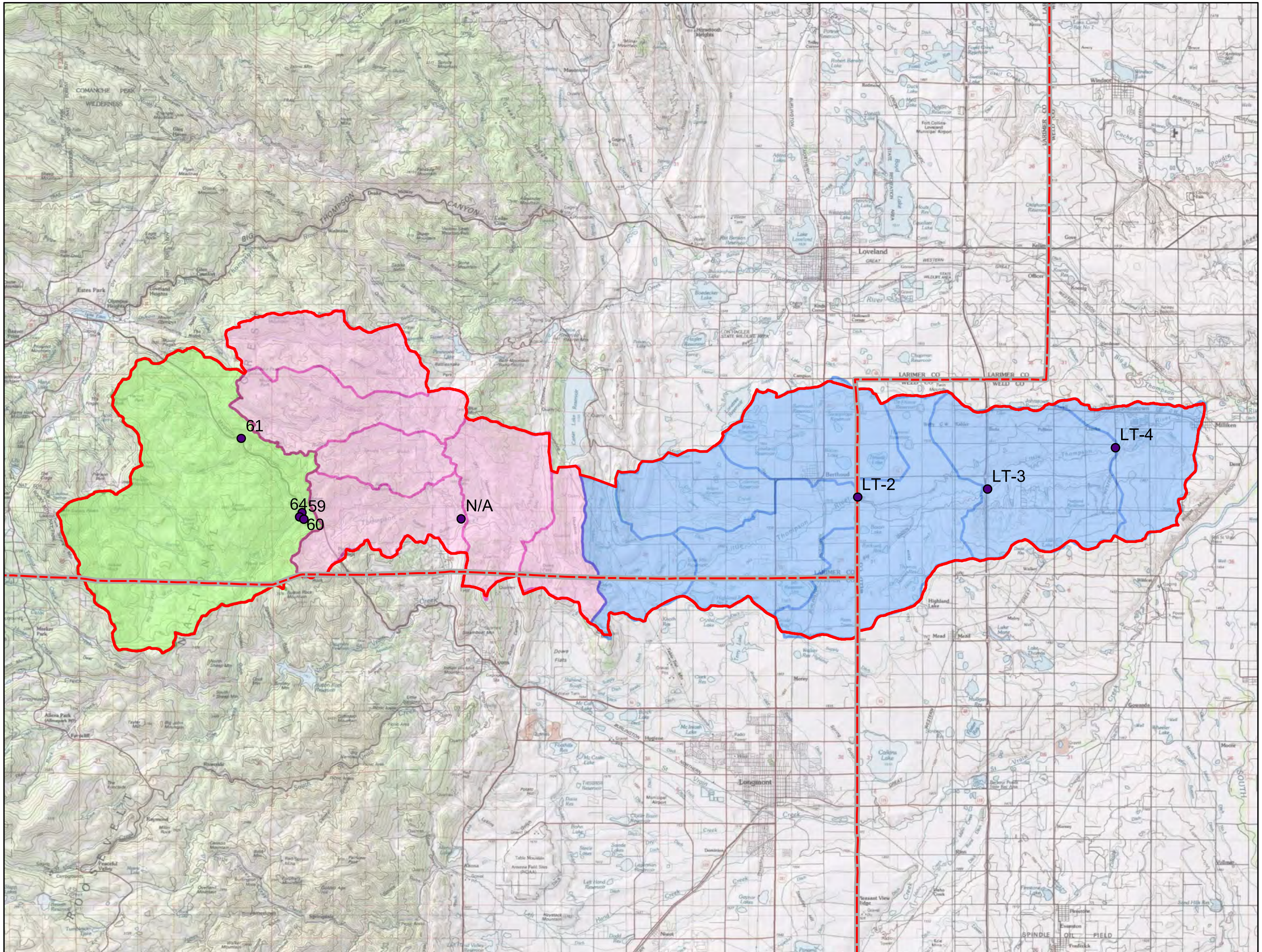
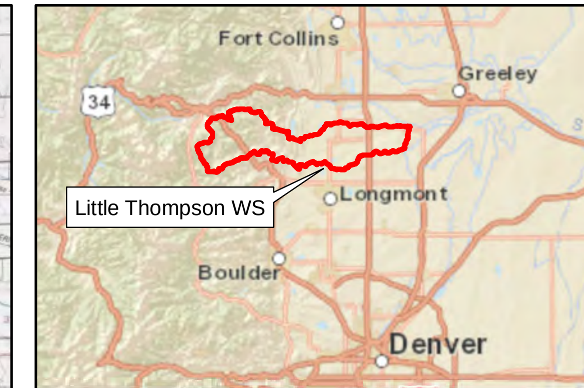
Figures





LEGEND

- Calibration Point
- Phase 1
- Phase 2 Mountains
- Phase 2 Plains
- Little Thompson River Watershed
- Counties

N

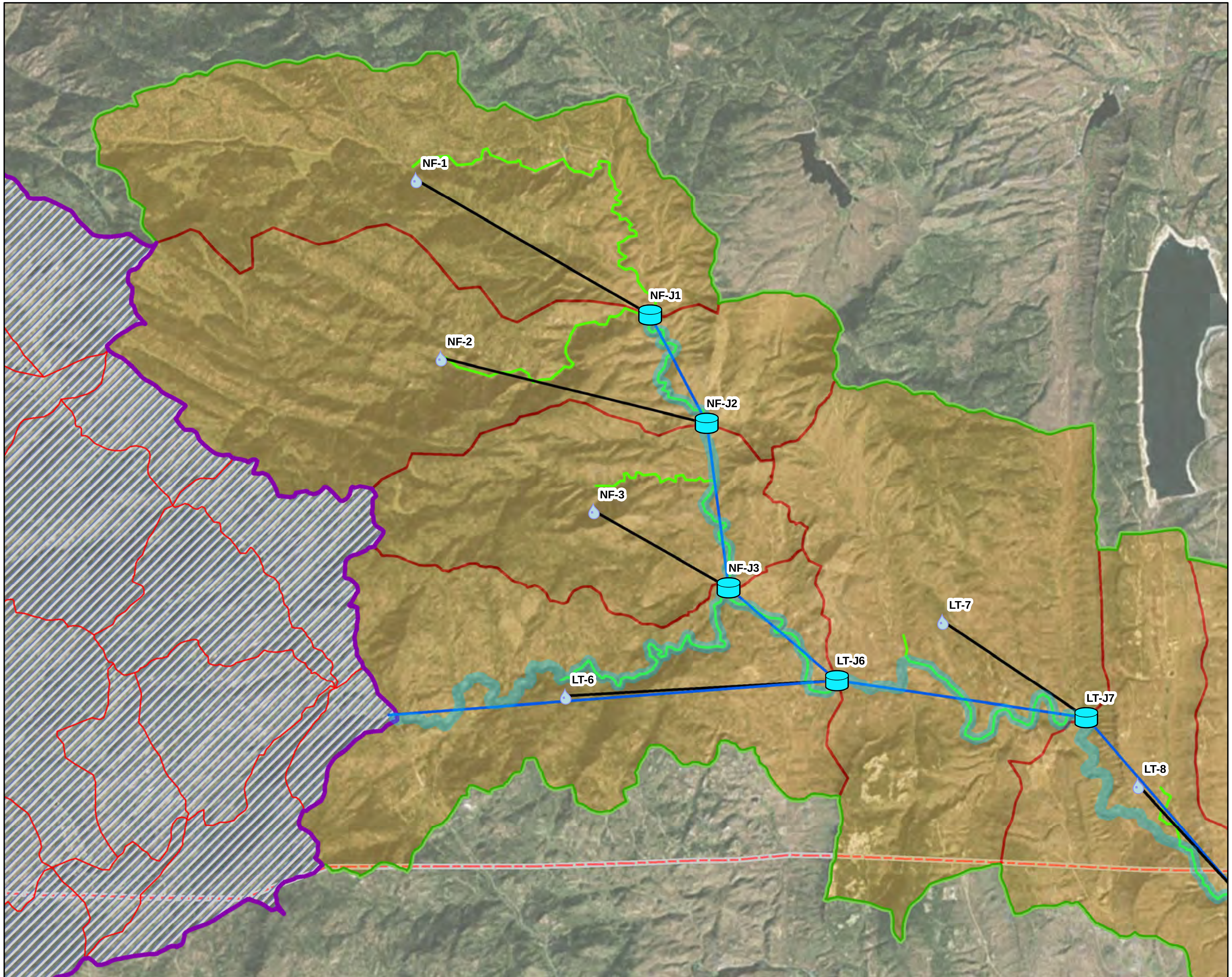
0 1.5 3 4.5 6 Miles

1 inch = 3 miles

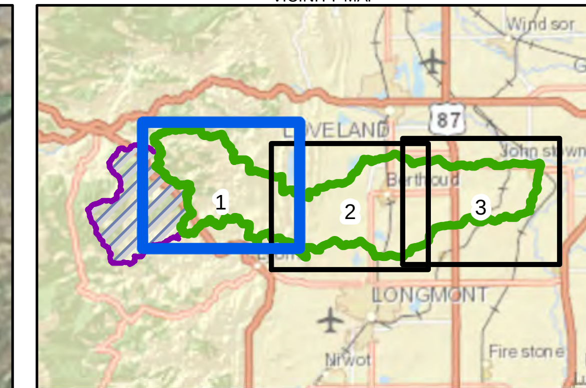
FIGURE 1
Little Thompson River Overview Map

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL



VICINITY MAP



LEGEND

Model Precipitation Gage

Subbasin Routing

Reach Segment

Basin Connection

Model Junctions

Model Reach

Model Phase 1 Subbasins

Model Phase 2 Subbasins

Flow Paths to Subbasin Centroid

Model Phase 1 Watershed

Model Phase 2 Watershed

Counties

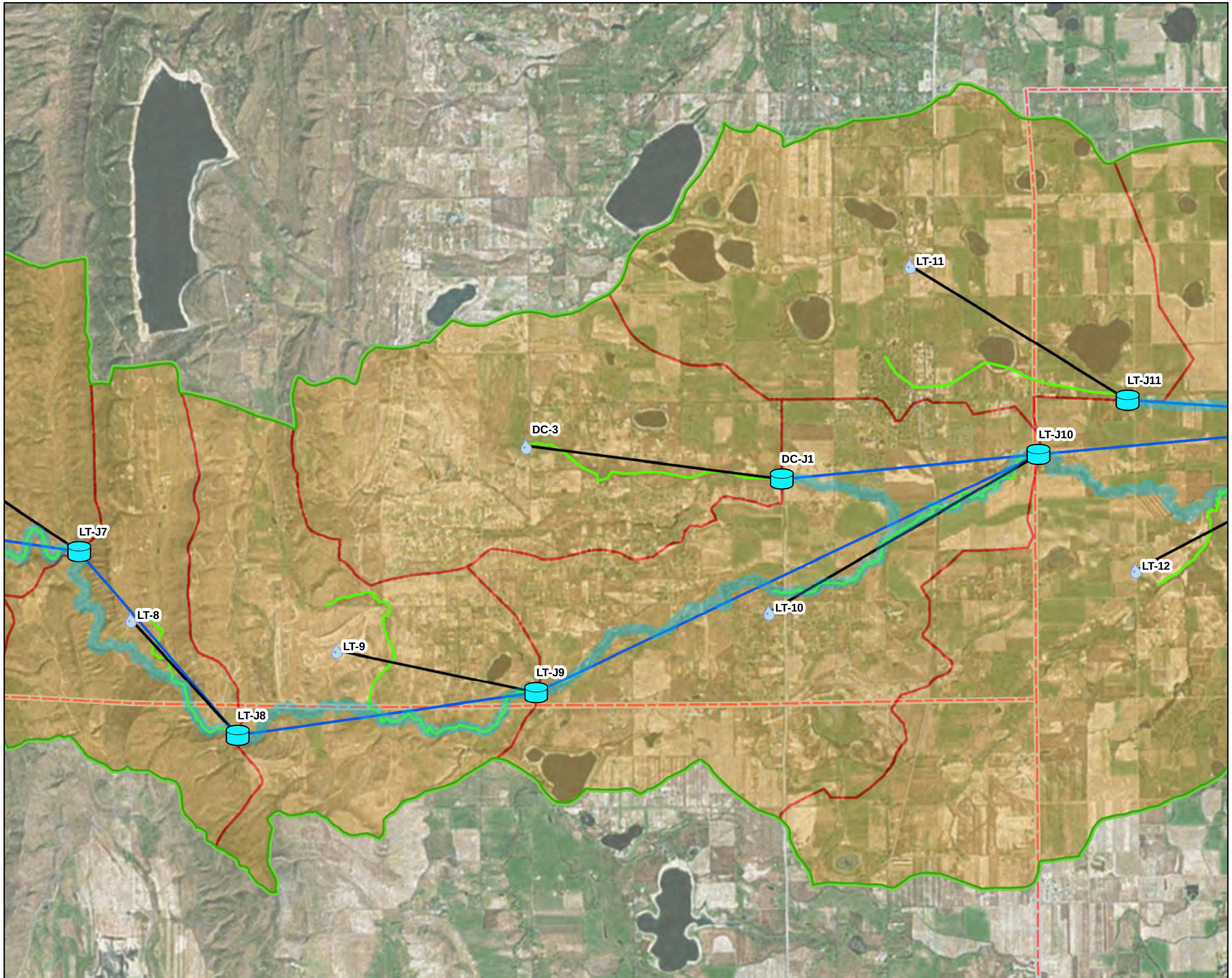
N

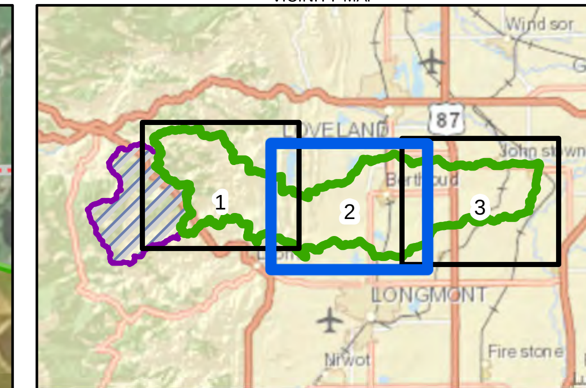
0 0.5 1 2 Miles

1 inch = 1 miles

FIGURE 2a
Phase 2 Connectivity Map - Page 1 of 3

CDOT Flood Recovery Hydrologic Evaluation





LEGEND

- Model Precipitation Gage

Subbasin Routing

- Reach Segment
- Basin Connection
- Model Junctions
- Model Reach
- Model Phase 1 Subbasins
- Model Phase 2 Subbasins
- Flow Paths to Subbasin Centroid
- Model Phase 1 Watershed
- Model Phase 2 Watershed
- Counties

0 0.5 1 2 Miles

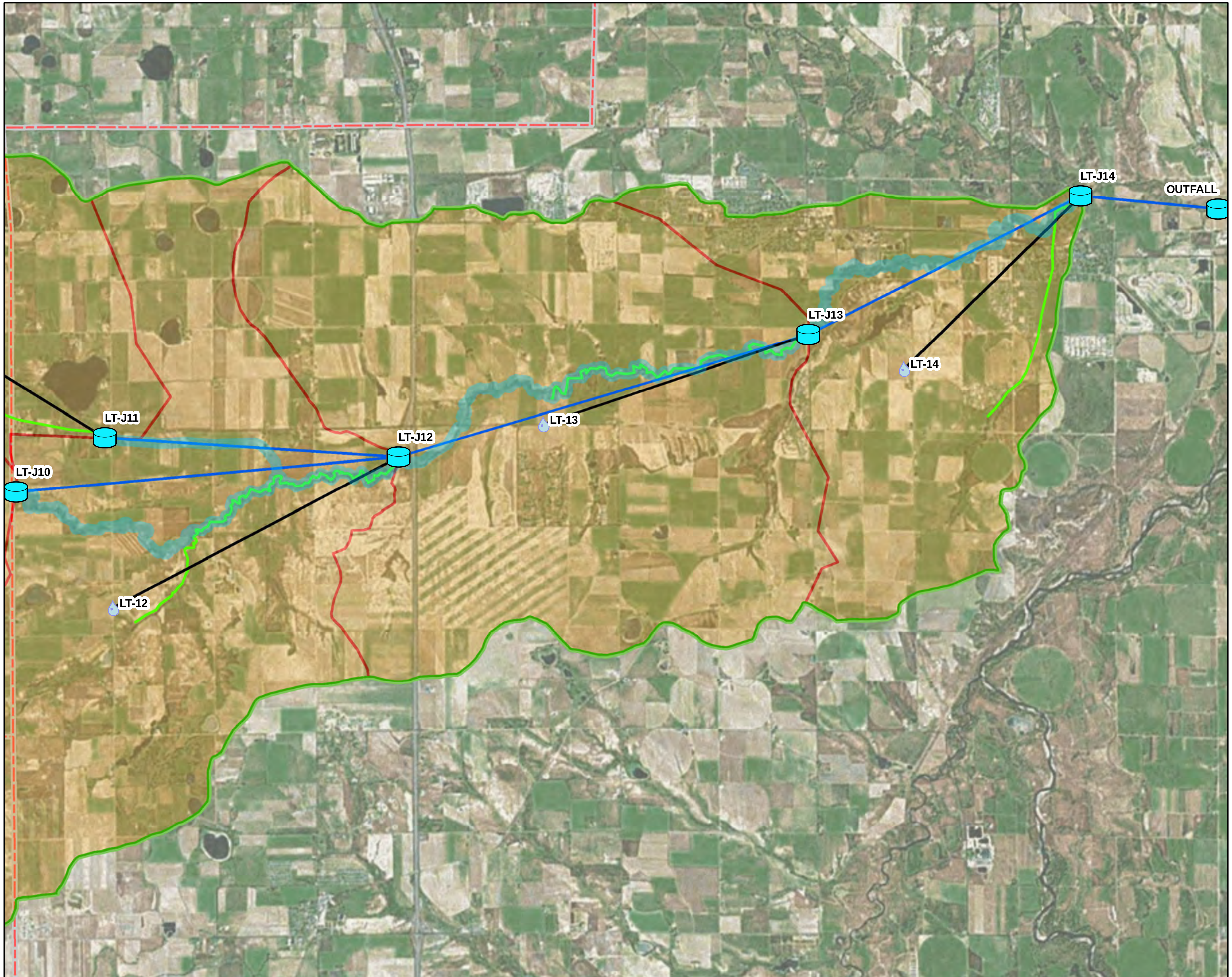
1 inch = 1 miles

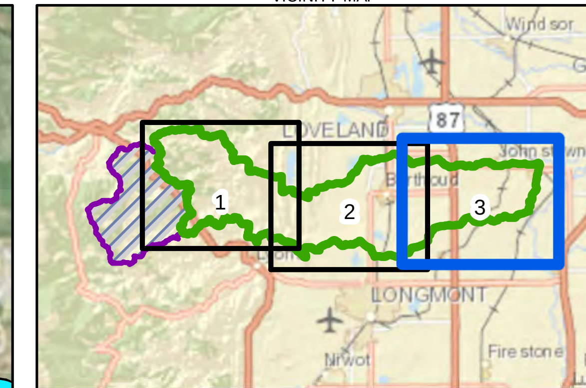
FIGURE 2b

Phase 2 Connectivity Map - Page 2 of 3

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL.





LEGEND

- Model Precipitation Gage

Subbasin Routing

- Reach Segment
- Basin Connection
- Model Junctions
- Model Reach
- Model Phase 1 Subbasins
- Model Phase 2 Subbasins
- Flow Paths to Subbasin Centroid
- Model Phase 1 Watershed
- Model Phase 2 Watershed
- Counties

N

0 0.5 1 2 Miles

1 inch = 1 miles

FIGURE 2c

Phase 2 Connectivity Map - Page 3 of 3

CDOT Flood Recovery Hydrologic Evaluation

UNK T:\481085-482330 CDOT EMERGENCY RESPONSE SERVICES\390_DESIGN_ELEMENTS\DRAINAGE\GIS_PHASE_II\MAPFILES\REPORT\LITTLETHOMPSONCONNECTIVITYMAP_PHASE_II_CBM2.MXD CMCDONOU 11/5/2014 10:18:33 AM

CH2MHILL.

Figure 3 – AWA 10 Day Precipitation (Phase 2)

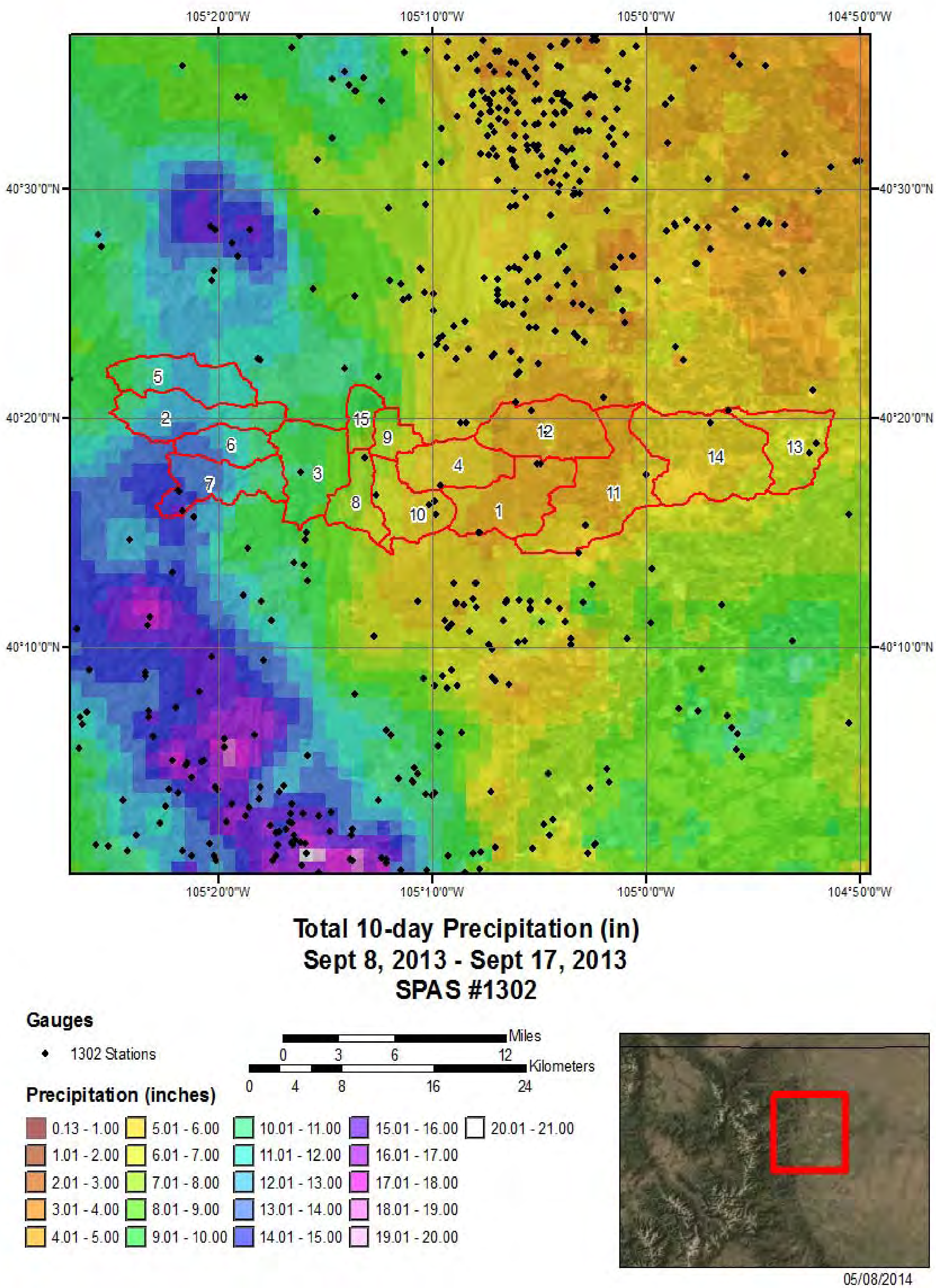


Figure 4a - Phase 2 Muskingum-Cunge Eight-Point Routing Cross Sections

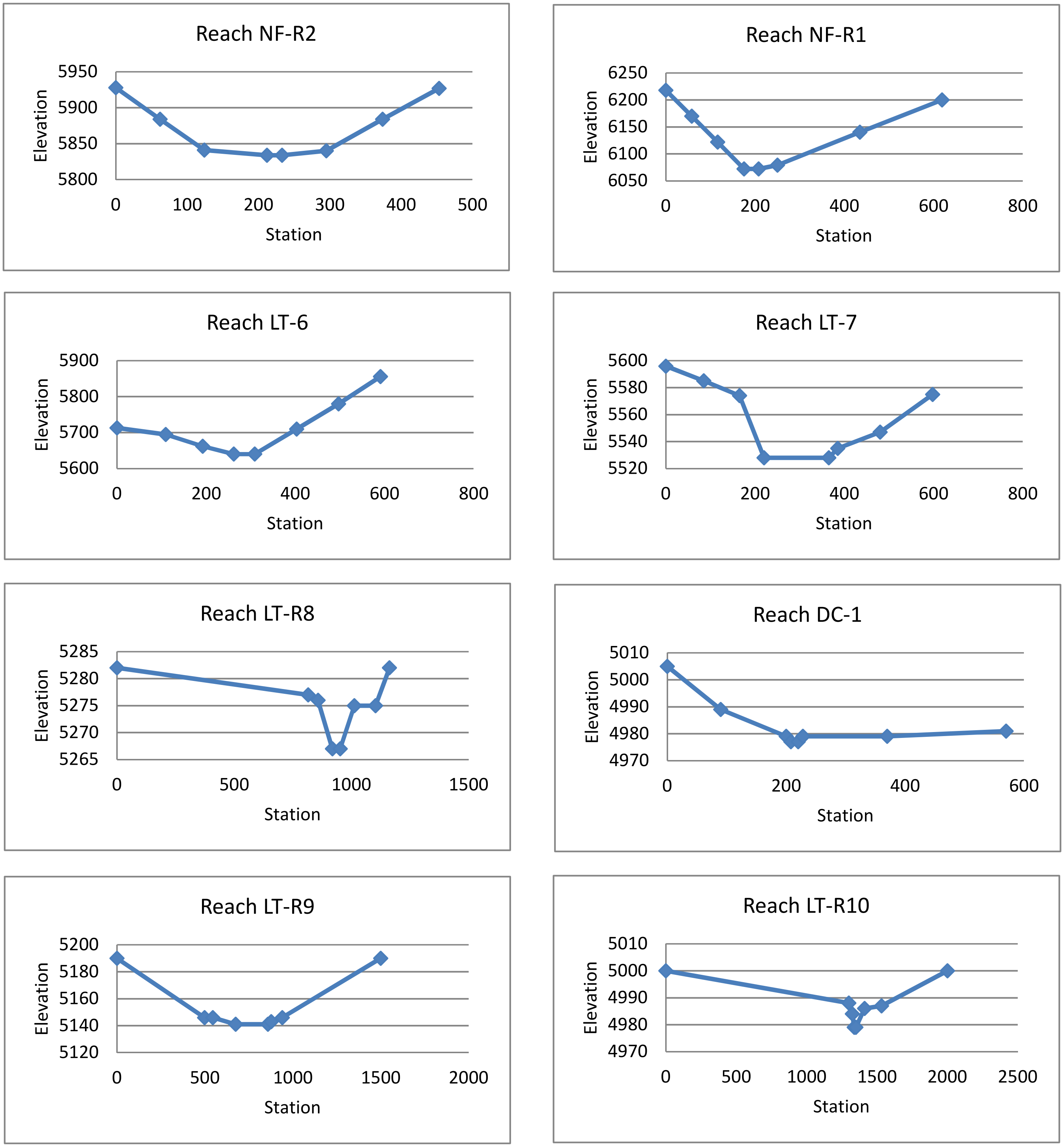
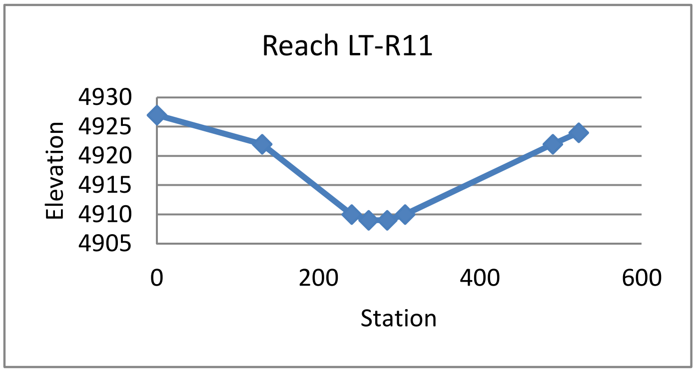
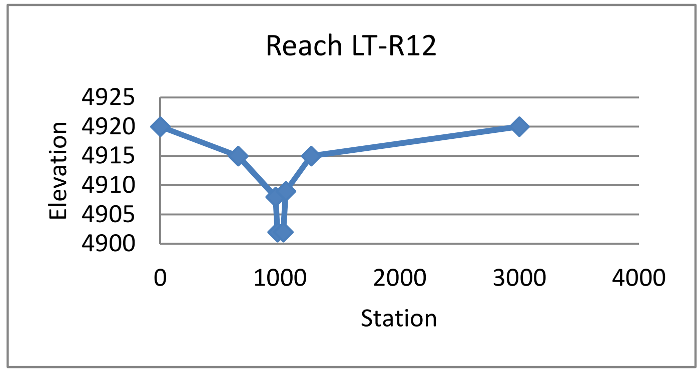
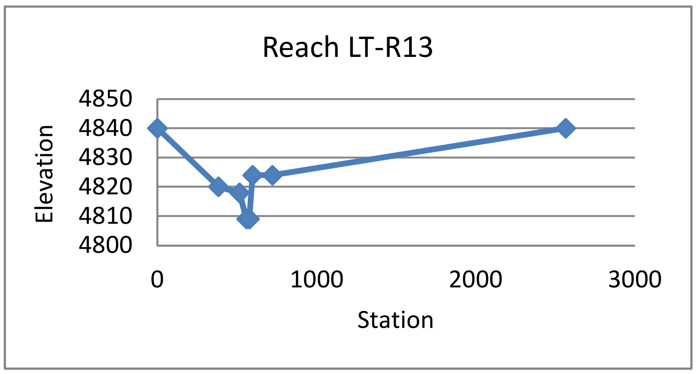
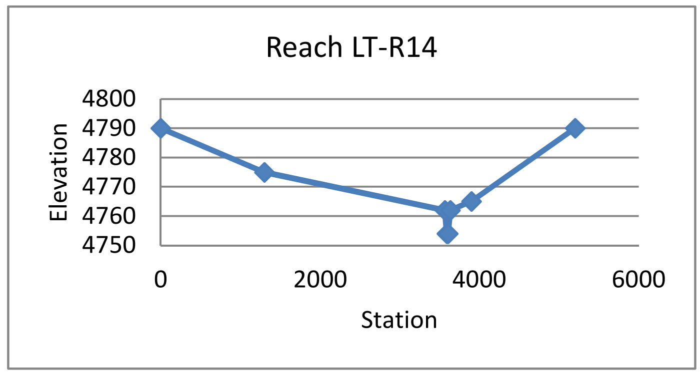


Figure 4b - Phase 2 Muskingum-Cunge Eight-Point Routing Cross Sections (continued)









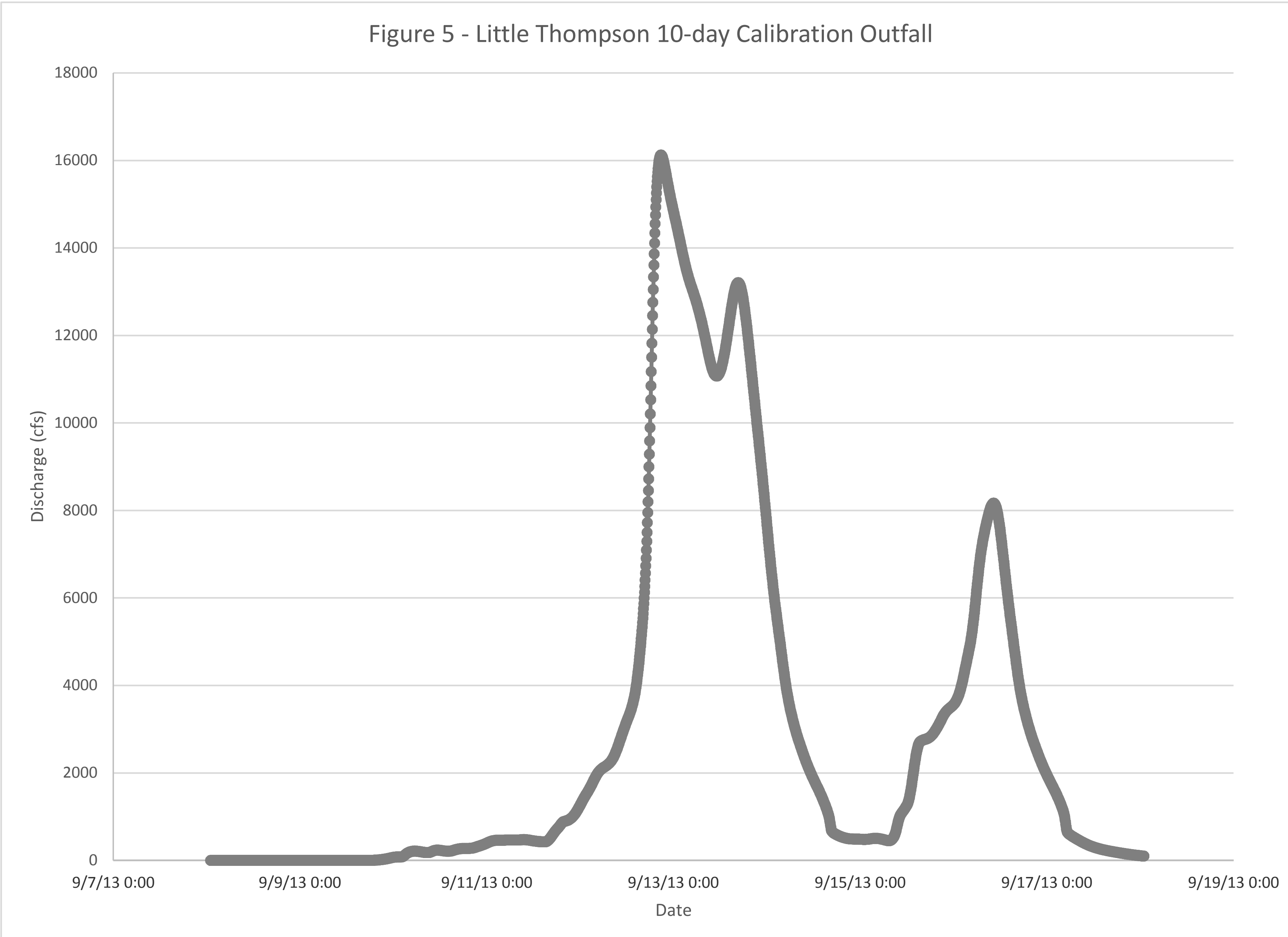


Figure 5 - Little Thompson 10-day Calibration Outfall
Discharge (cfs)
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
9/7/13 0:00
9/9/13 0:00
9/11/13 0:00
9/13/13 0:00
9/15/13 0:00
9/17/13 0:00
9/19/13 0:00
Date

Appendix A – Hydrologic Analysis

Table A-1

Little Thompson River Basin Area

Basin ID	Area (mi^2)	Subsection
DC-3	9.69	Phase 2 Plains
LT-10	13.00	Phase 2 Plains
LT-11	11.97	Phase 2 Plains
LT-12	20.46	Phase 2 Plains
LT-13	20.63	Phase 2 Plains
LT-14	9.95	Phase 2 Plains
LT-1A	2.89	Phase I
LT-1B	2.87	Phase I
LT-2	2.99	Phase I
LT-3A	4.09	Phase I
LT-3B	1.00	Phase I
LT-4A	2.06	Phase I
LT-4B	1.89	Phase I
LT-5	0.62	Phase I
LT-6	10.18	Phase 2 Mountains
LT-7	12.04	Phase 2 Mountains
LT-8	6.31	Phase 2 Mountains
LT-9	8.79	Phase 2 Plains
NF-1	10.27	Phase 2 Mountains
NF-2	11.45	Phase 2 Mountains
NF-3	6.13	Phase 2 Mountains
WF-1A	2.84	Phase I
WF-1B	4.00	Phase I
WF-2	4.31	Phase I
WF-3A	2.10	Phase I
WF-3B	1.72	Phase I
WF-4	4.66	Phase I
WF-5A	1.19	Phase I
WF-5B	0.60	Phase I
WF-6A	2.02	Phase I
WF-6B	1.96	Phase I
total:	**194.62**	

Table A-2
Little Thompson Lag Time Parameters

Basin ID	K_{n*}	L *mi*	L_c *mi*	S *ft/mile*	TLAG *hours*
DC-3	0.10	6.19	3.71	40	3.4
LT-10	0.10	8.09	3.59	40	3.7
LT-11	0.10	5.96	2.66	30	3.1
LT-12	0.10	9.43	4.50	30	4.3
LT-13	0.10	9.57	3.63	30	4.1
LT-14	0.10	5.57	2.57	40	2.9
LT-1A	0.15	3.61	2.31	370	2.5
LT-1B	0.15	2.69	1.38	150	2.2
LT-2	0.15	3.10	2.72	450	2.4
LT-3A	0.15	3.63	1.76	400	2.3
LT-3B	0.15	2.34	0.93	770	1.4
LT-4A	0.15	3.43	2.25	990	2.1
LT-4B	0.15	3.18	1.77	170	2.5
LT-5	0.15	1.79	0.79	750	1.2
LT-6	0.15	9.59	5.30	130	5.4
LT-7	0.10	6.23	3.30	130	2.7
LT-8	0.09	4.67	1.81	130	1.8
LT-9	0.08	7.07	4.18	70	2.7
NF-1	0.10	8.26	4.75	360	2.8
NF-2	0.15	8.88	4.55	410	4.2
NF-3	0.15	5.83	2.86	510	3.0
WF-1A	0.15	4.34	1.88	490	2.0
WF-1B	0.15	3.68	1.57	560	2.1
WF-2	0.15	3.37	1.73	690	2.0
WF-3A	0.15	2.96	1.91	200	2.4
WF-3B	0.15	2.59	1.90	390	2.1
WF-4	0.15	5.29	3.31	320	3.3
WF-5A	0.15	2.93	1.68	180	2.4
WF-5B	0.15	1.19	1.45	730	1.3
WF-6A	0.15	2.39	1.51	1340	1.5
WF-6B	0.15	4.08	2.55	280	2.8

Table A-3

Little Thompson River 10 Day Model Parameters

Sub Basin	Section	CN (AMC II)	Land Cover Condition	Peaking Coefficient	Kn	Manning's n Overbank	Manning's n Channel
DC-3	Phase 2 Plains	83	Fair	0.4	0.10	0.100	0.100
LT-10	Phase 2 Plains	82	Fair	0.4	0.10	0.100	0.100
LT-11	Phase 2 Plains	84	Fair	0.4	0.10	0.100	0.100
LT-12	Phase 2 Plains	84	Fair	0.4	0.10	0.100	0.100
LT-13	Phase 2 Plains	83	Fair	0.4	0.10	0.100	0.100
LT-14	Phase 2 Plains	80	Fair	0.4	0.10	0.100	0.100
LT-1A	Phase 1	75	Poor	0.4	0.15	0.080	0.045
LT-1B	Phase 1	72	Poor	0.4	0.15	0.080	0.045
LT-2	Phase 1	55	Fair	0.4	0.15	0.080	0.045
LT-3A	Phase 1	55	Fair	0.4	0.15	0.080	0.045
LT-3B	Phase 1	60	Fair	0.4	0.15	0.080	0.045
LT-4A	Phase 1	50	Fair	0.4	0.15	0.080	0.045
LT-4B	Phase 1	59	Fair	0.4	0.15	0.080	0.045
LT-5	Phase 1	55	Fair	0.4	0.15	0.080	0.080
LT-6	Phase 2 Mountains	62	Fair	0.4	0.15	0.080	0.080
LT-7	Phase 2 Mountains	71	Fair	0.4	0.10	0.080	0.080
LT-8	Phase 2 Mountains	74	Fair	0.4	0.09	0.080	0.080
LT-9	Phase 2 Plains	77	Fair	0.4	0.08	0.100	0.100
NF-1	Phase 2 Mountains	74	Poor	0.4	0.10	0.080	0.080
NF-2	Phase 2 Mountains	61	Fair	0.4	0.15	0.080	0.080
NF-3	Phase 2 Mountains	61	Fair	0.4	0.15	0.080	0.080
WF-1A	Phase 1	73	Poor	0.4	0.15	0.080	0.045
WF-1B	Phase 1	73	Poor	0.4	0.15	0.080	0.045
WF-2	Phase 1	53	Fair	0.4	0.15	0.080	0.045
WF-3A	Phase 1	76	Poor	0.4	0.15	0.080	0.045
WF-3B	Phase 1	57	Fair	0.4	0.15	0.080	0.045
WF-4	Phase 1	74	Poor	0.4	0.15	0.080	0.045
WF-5A	Phase 1	70	Poor	0.4	0.15	0.080	0.045
WF-5B	Phase 1	57	Fair	0.4	0.15	0.080	0.045
WF-6A	Phase 1	68	Poor	0.4	0.15	0.080	0.045
WF-6B	Phase 1	55	Fair	0.4	0.15	0.080	0.045

Table A-4
Little Thompson River Modeling Results

		2013 Peak Discharge Estimates			Calibrated Model			
HMS Node	Location	Drainage Area (sq miles)	Observed Peak Discharge (cfs)	Unit Discharge (cfs/sq miles)	Modeled Peak Discharge (cfs)	% Difference	Unit Discharge (cfs/sq miles)	% Difference
LT-J3	Little Thompson River Midpoint of Watershed	13.8	2,470	179	2,119	-14%	154	-14%
LT-J4 Without WF	Little Thompson River Upstream of Confluence with West Fork Little Thompson River	17.8	2,680	151	2,653	-1%	149	-1%
LT-J4	Little Thompson River Downstream of Confluence with West Fork Little Thompson River	43.2	7800[a]	181	8,639	11%	200	11%
LT-J4 Without LT	West Fork Little Thompson River Upstream of Confluence with Little Thompson River	25.4	6,200	244	6,077	-2%	239	-2%
LT-J5	Little Thompson River at Highway 36	43.8			8,730		199	
LT-J6	Little Thompson River at X Bar 7 Ranch	81.8	15,731	192	13,196	-16%	161	-16%
NF-J3 Without NF	Little Thompson River Upstream of North Fork Little Thompson River	43.8			8,727		199	
NF-J3 Without LT	North Fork Little Thompson River Upstream of Little Thompson River	27.9			3,610		130	
LT-J8	Little Thompson River at LTCANYO Gage	100.2			14,615		146	
LT-J10	Little Thompson River at South County Line Road	131.7	13,400	102	15,470	15%	118	15%
LT-J12	Little Thompson River at Interstate 25	164.1	15,700	95	16,012	2%	98	2%
LT-J13	Little Thompson River at County Road 17	184.7	18,000[b]	97	16,179	-10%	88	-10%
Phase 2 Outfall	Little Thompson River Upstream of Confluence with Big Thompson	194.6			16,153		83	

[a] - This flow was inaccessible and the observed peak discharge was estimated based on observations along similar, adjacent watersheds.
[b] – Bridge overtopped, (URS, 2015)
cfs = cubic feet per second

Appendix F

Project Correspondence and Response to Review Comments

CDOT Review Comments on Draft Lower Big Thompson 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. Figure ES-2 may be a little too busy to "squeeze" into such a small space. Same note for Figures 4 and 6. 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.

3. Table 6 table font is perhaps a bit too small. Could this table also enjoy its own page to enlarge the text?
   Table 6 (now Table 7) was increased slightly in the report by decreasing the margins on the page. However, text was added directing the reader to Appendix D.5 which is much larger (11x17 sheet) and provides additional information.

CWCB Review Comments on Draft Lower Big Thompson Phase 2 Report

Provided on April 8, 2015

Response to Review Comments by Jacobs Team

1. The page numbers are off. No page 4, which is supposed to be the Lower Big T study area. That figure is not labeled as Figure 1.
   The page numbers have been corrected. The mistake was a result of inserting the figure into the pdf as page 3 instead of page 4 where it was supposed to be. The figure has also been labeled as Figure 1.

2. Page 13 under hydrograph routing describes using 10-m DEM to measure the length and slope of the channel reaches. Was wondering why they didn't use the LIDAR to do that? They did use the Lidar to cut the cross sections.
   The HEC-GeoHMS program was used to define the basin delineations and the channel reaches as one combined process. This process was done early in the project to identify overall basin boundaries and identify locations of tributary confluences as discussed in Section 2.4.2 of the report. The LIDAR data were only available along the major channels and therefore not sufficient to determine the basins and smaller tributaries during the HEC-GeoHMS setup. Also, since the average channel reach length in the model is approximately 3 miles long and the average channel reach slope is approximately 50 feet per mile, it was determined that the 10-meter DEM was sufficient to represent the channels in the hydrologic model. The cross-section detail for each routing reach was then added to the model based on the LIDAR data.

3. 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 would help. I believe in Appendix D there is a sub basin map that they could reference that would either go with Table 4 or is the HMS basin delineations, but it was unclear to me.
   A reference to Figure D.1 in Appendix D was added to the text to direct the reader to the map showing the individual basin delineations.

4. A Stream gage location map would be helpful to reference in the section on FFA.
   The stream gage locations have been added to Figure D.1 in Appendix D. The figure is now referenced in Section 2.3.

5. 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.
   Labels have been added to the figures. The figure on page 4 has been labeled "Figure 1" and the figure in Appendix D has been labeled 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.

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.
   These 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 1.2:** Third paragraph, second sentence should read "Dry Creek, along with several other small unnamed tributaries in this area, drains east under Highway 287."
   The sentence was corrected as indicated.

10. **Sec 2.4.2:** First Paragraph second sentence should be revised to: HEC-GeoHMS uses DEMs to develop watershed boundaries and flow paths."
    The sentence was corrected as indicated.

11. **Sec 3.0**: Grammatical error pg 26, last paragraph, fifth sentence. "The 2013 peak discharge estimate of 24,900 cfs show that…are not sufficient **to** control large floods.."
    The sentence was corrected as indicated.

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

Review comments by Will Thomas, Michael Baker International, on behalf of FEMA

Response to Review Comments by Jacobs Team

Background

Hydrology analyses were performed on the Big Thompson River using a HEC-HMS model from the confluence with Buckhorn Creek near the mouth of the canyon to the confluence with the South Platte River. The watershed upstream of the confluence with Buckhorn Creek was studied in Phase 1 of this project. The HEC-HMS model was calibrated to estimates of the September 2013 flood event made at 6 locations along the Big Thompson River. 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, Volume 8, to be indicative of the rainfall over the respective watershed area.

The hydrologic modeling procedure that was developed 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,
- Adjusting the runoff curve numbers and using the maximum 24-hour rainfall for the 2013 flood to achieve a reasonable calibration, and
- Using the adjusted runoff curve numbers with the NOAA Atlas 14 rainfall to estimate the 10-, 4-, 2-, 1-, and 0.2-percent chance flood discharges.

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. Pages 9-10 – Bulletin 17B frequency analyses were performed at two gaging stations on the lower Big Thompson River at Loveland (515 square miles) and La Salle (828 square miles). The 1-percent chance flood was 21,180 cfs at Loveland and 12,310 cfs downstream at La Salle. The top 15 peaks (including the 2013 flood) are very similar at both stations. A major difference is

the historic record length used to adjust the 2013 flood. The recommendation is to use a historic period of 99 years for both analyses and then the frequency curves will be more similar as they should be. The 2013 flood is the highest flood on the lower Big Thompson River for at least the last 99 years.

We examined both the Loveland and La Salle gages in response to the comment and we agree that extending the historic period to 99 years at the Loveland gage is reasonable and advisable. This is based on the observation that during the period of overlap between the two gages, very large events at the Loveland gage show up as significant peaks at the La Salle gage. We can surmise, therefore, that if any major event had occurred in Loveland during the segments of the last 99 years in which there was no gage at Loveland, it would have shown up as a large event in the La Salle record. There is no such indication, so we agree with the commenter's conclusion that the 2013 flood was the highest peak in at least the last 99 years in Loveland. We have revised the analysis at the Loveland gage accordingly. No change is required at the La Salle gage.

2. 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. This will give a better idea of how well the log-Pearson Type III frequency curves fi t the plotting positions.

   The flood frequency graphs from HEC-SSP were added to Appendix B.

The conclusions of the Lower Big Thompson River 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 in the Big Thompson watershed.
- It is recommended that local floodplain administrators consider using the results of this hydrologic analysis to update and revise current regulatory discharges in the Big Thompson watershed.

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 for the Big Thompson watershed.

Will Thomas
Michael Baker International
April 3, 2015

CH2M-Hill Review Comments on Draft Lower Big Thompson 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 the Lower Big Thompson beyond a rough estimate of the peak timing at I-25. The two 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 the Big Thompson 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.
    The list of reservoirs and a brief description of each is provided in Section 1.2 – Project Area Description. As noted above, additional information regarding how the reservoirs were modeled for the 2013 Flood versus the predictive storms is discussed in Section 3.0 of the report. Stage-Storage-Discharge relationships have been provided in Appendix D.1 for the reservoirs included in the predictive model.

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.

Big Thompson River Comments

1. Is the Dry Creek (South) discussion necessary in the Executive Summary?
   Dry Creek (South) was raised by a reviewer in the Phase 1 Report comments as a comparison for Big Thompson. Therefore, it is viewed as an important with respect to the validity of the current model and will be left in the Executive Summary.

2. There is a high degree of correlation-bias in the modeled peak discharge results (true for any model) such that correlation between unit peak discharges should not be used as justification for adoption of the modeled peak discharges. Presenting this as an observation, as done in the St. Vrain Creek report, is more appropriate.
   The language has been changed to emphasize that the modeled peak discharges are more consistent downstream of I-25 when compared to the abrupt drop in the current regulatory peak discharges. The observation of the unit discharge comparison is simply referenced to provide a visual representation of the concept.

3. Section 1.2 – provided CH2M Hill memo documents the calibration of the *10-day* model of the Little Thompson River that should be considered independent of the 24-hour calibration that forms the basis of the *Little Thompson Hydrologic Evaluation*. Please revise the last sentence of this section to clarify.
   Clarification was added to Section 1.2.

4. In Section 1, it is mentioned that there is storage volume specifically appropriated for flood control in Boyd Lake, then on Page 23 it is stated that this is discounted and Boyd Lake was assumed full to the spillway elevation. Why was the dedicated flood storage not included in the predictive model?
   The text was revised to more clearly state that Boyd Lake was modeled assuming the reservoir was full to the principal spillway and that the dedicated 880 acres of flood storage between the principal spillway crest and the emergency spillway crest was utilized (depth of 6 inches). The peak discharge 6 inches above the principal spillway crest is only 250 cfs, above this depth the peak discharge increases dramatically.

5. Was the discharge of Boyd Lake to the Greeley and Loveland Canal accounted for in the hydrologic modeling?
   No. Boyd Lake is designed to spill to the Greeley and Loveland Canal for releases up to 250 cfs. Anything above this is designed to spill southeast toward the Big Thompson River upstream of I-25. The reservoir improvements were based on a 2-hour storm which has much less runoff volume than the 24-hour storms modeled in this study. Therefore, the assumption was made that the additional runoff volume in the watershed during a 24-hour storm would occupy any available capacity in the canal and all spills from Boyd Lake would follow the overflow path to the Big Thompson River.

6. Page 3 – Ryan Gulch: Verify that Carter Lake's outlet canal flows into Boulder Creek as opposed to Little Thompson that is next watershed south
   The outlet canal is named the St. Vrain Supply Canal and it is capable of delivering water to the Little Thompson, St. Vrain, and Boulder Creek watersheds. The report has been updated to reflect this correction.

7. Table 1 – global comment suggesting to not present information on Little Thompson River and reference CH2M HILL report instead.
   The Big Thompson hydrology is dependent on the inflows from the Little Thompson. To leave all information pertaining to the Little Thompson River out of the report would make it challenging for the reader to understand the effects on the Little Thompson on the Big Thompson River. Also, if peak discharges in the Little Thompson report change, it will be necessary to update the Big Thompson report anyway since the downstream results are dependent.

8. Page 7 – Is the drop in flood flow frequencies between County Road 9E and Interstate 25 in reference to results from the USACE, 1971 study or a general comment regarding the FIS (possibly two different studies?)
   The drop appears to be a relic from the 1971 USACE study. The upstream reach through Loveland was restudied in 2005 by Ayres and the peak discharges from the 1971 study were retained. Similarly, the downstream reach near Johnstown was restudied in 2005 by Anderson Consulting and the peak discharges from the 1971 study were retained.

9. I would add tributaries to the workmap, to correspond with discussion in the report.
   The named tributaries have been added to Figure D.1.

10. 0.15 Seems high for a Manning's N value for the overbanks (table in Appendix), the text in the report states a range of 0.05 to 0.1.
    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 Dry Creek (South) 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.

11. Page 26 – please add the "±" to the percent differences in parentheses to clarify these are percent differences, rather than mistyped annual percent chance exceedances.
    The "±" was added to each of the percent differences.

12. Appendix D.3 – why the sudden cessation of the hydrograph on 9/15?
    The 10-day hydrograph for the upper Big Thompson Watershed (Phase 1) was dependent on the observed peak discharges from Lake Estes. The time series for the discharge from Lake Estes provided by the USBR ended on the 15th.

13. Appendix D.7 – Does this relate to Dry Creek (South), I would add South to title.
    "South" was added to the title.

14. Appendix D.9 – The modeled peak discharge occurs on 9/16, rather than early AM of 9/13 when the peak discharge was observed. This is likely a relic of the SCS CN method and would suggest reporting the modeled peak discharge as the peak discharge between 9/12 and 9/13 (1,200 to 1,500 cfs), rather than the 2,600 cfs on 9/13.
    The modeled peak discharge of 2,600 cfs was reported.

15. Might be helpful to add HMS data for each reservoir and source in the appendix, ie, stage, storage, discharge relationships?
    Stage-Storage-Discharge relationships have been provided in Appendix D.1 for the reservoirs included in the predictive model.

Town of Johnstown Review Comments on 2nd Draft of Lower Big Thompson Phase 2 Report

Provided on June 3, 2015

Response to Review Comments by Jacobs Team

From: Greg Weeks
To: Holly M. Linderholm
Cc: John Franklin - Town of Johnstown (jfranklin@townofjohnstown.com); Tom Hellen
Subject: RE: REMINDER: CDOT/CWCB Big Thompson Watershed Phase 2 Draft Report Review and Comment
Date: Wednesday, June 03, 2015 10:34:09 AM
Attachments:

Holly

I read through the Draft Report (Executive Summary & Main Report Body – through page 35). I did not review the appendices per se. Following are my comments/observations:

- Overall, the report seems well written, readable, and understandable.
- The general conclusions/results appear reasonable and substantiated.
- I did happen to note a relatively few number of apparent typos and/or recommended corrections, as follows:
  - Pg. ES-5, first line following Table ES-2: "… consistent in terms peak discharge…" should have the missing "of" added – e.g. "… consistent in terms **of** peak discharge…". **The typo has been corrected in the final report.**
  - Pg. 5, 1st par:
    - 5th line: The abbreviation "PMP" is utilized, without clarification of what it stands for (e.g. "Probable Maximum Precipitation"). Outside of the hydraulics/hydrology community, PMP may be a foreign abbreviation. It might be good (at least for this first usage) to "spell it out" (e.g. "Probable Maximum Precipitation (PMP) event". **The acronym has been spelled out in the final report.**
    - 10th line: Missing space – e.g. "…andRyan…" should be "… and Ryan…". **The typo has been corrected in the final report.**
  - Pg. 32, 3rd par., 3rd line (same as comment above on ES-5) - "… consistent in terms peak discharge…" should have the missing "of" added – e.g. "… consistent in terms **of** peak discharge…". **The typo has been corrected in the final report.**

Thanks
Greg

Gregory A. Weeks, P.E. CFM, LEED ® AP
For TTG – As Town Engineer for Johnstown
TTG
'Celebrating 60 Years'

Weld County Review Comments on 2nd Draft of Lower Big Thompson 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.

Larimer County Review Comments on the Draft Phase 2 Reports for the Lower Big Thompson River and Lower Little Thompson River

A review comment regarding the boundary delineation between the Big Thompson and Little Thompson watersheds was provided directly to CHM2-Hill as part of the review for the Draft Lower Little Thompson Report. The CH2M-Hill response is included in the Final Lower Big Thompson Report for clarity.

From:	Holly M. Linderholm
To:	Gordon Gilstrap; tracyel@co.larimer.co.us; Mark Peterson
Cc:	Steven D. Humphrey; Griffin, Steven (Steven.Griffin@state.co.us); Houck - DNR, Kevin; Holly M. Linderholm; Morgan.Lynch@ch2m.com; James.Woidt@ch2m.com; Cory.Hooper@CH2M.com; Schram, Heidi; Jim T. Wulliman; drapp@peakstormwater.com
Subject:	Little Thompson and Big Thompson Hydrology Studies - Dry Creek Discussion/Findings
Date:	Monday, June 01, 2015 10:05:21 AM
Attachments:	LT_Overview_Map.pdf LarimerExhibit.pdf FIRM.PDF

Good Morning,

At the Little Thompson meeting there was discussion regarding the basin delineation of Dry Creek. We have looked into this in more detail and the CH2MHill team has provided the following response. Please respond by this Thursday the 4th by COB with either concurrence of our outlined approach or additional comments.

Respectfully,
Holly Linderholm

Holly M. Linderholm, MBA
Program Management Coordinator

Muller Engineering Company, Inc.
Consulting Engineers
777 S. Wadsworth Blvd. | Suite 4-100 | Lakewood, CO 80226

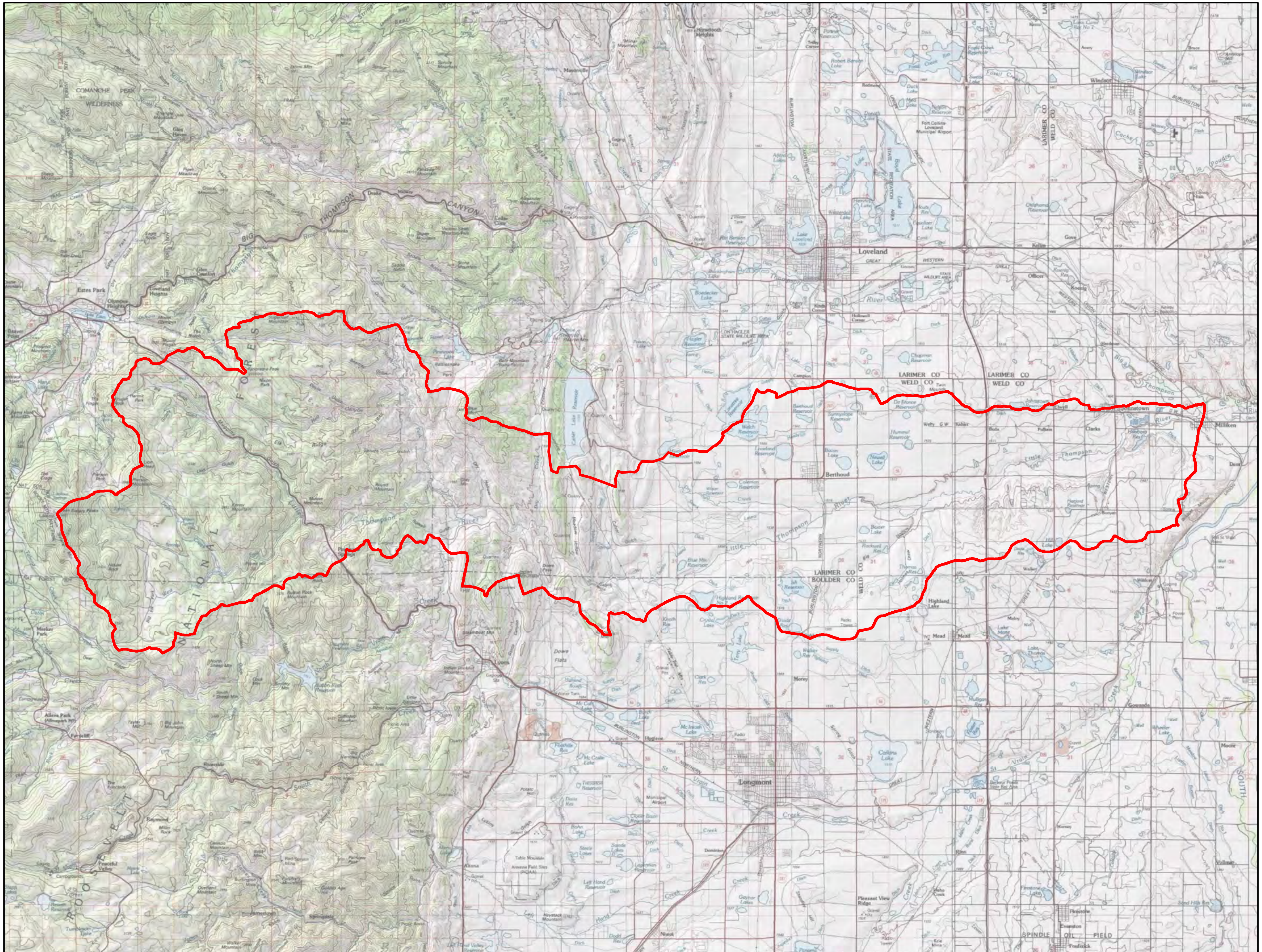
It is our understanding that the basin delineation concern between the Little Thompson and Big Thompson Watersheds originated due to the FEMA floodplain delineation along Dry Creek. Our team further examined the effective FIRM, see attached, and discovered that near County Road 8E and the headwaters of Dry Creek, the floodplain delineation in question follows "Unnamed Stream" rather than Dry Creek that originates Southeast of County Road 8E. It is our opinion that the approximate floodplain for Dry Creek to the north of County Road 8E was based on USGS quad topography (which identifies Dry Creek on the north side of County Road 8E, thus opposite the FIRM) versus what is observed in the field. It is our interpretation and understanding that there are two conveyance systems at County Road 8E, Unnamed Stream conveying water to the north and Dry Creek conveying water to the south. This corresponds with and supports Larry Lempka's assessment of Dry Creek.

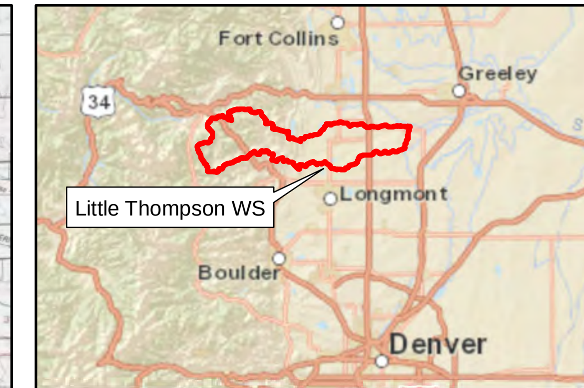
Unnamed Stream becomes part of the Dry Creek Lateral and Handy Ditch systems. We were able to confirm with the Dry Creek Lateral Ditch Rider, Larry's description of how water enters Welch and Lonetree Reservoirs. From here, some water flows east into the Little Thompson Watershed and some water flow north into the Big Thompson Watershed through a series of canals and waterways.

This reflects how we currently have the basins delineated. Please see the attached figure.

Based on these findings it is our recommendation that the Big Thompson and Little Thompson watershed boundaries remain unchanged from what is presented in the hydrology reports.

Morgan Lynch, PE, CFM
Water Resources Engineer





LEGEND

Little Thompson River Watershed

N

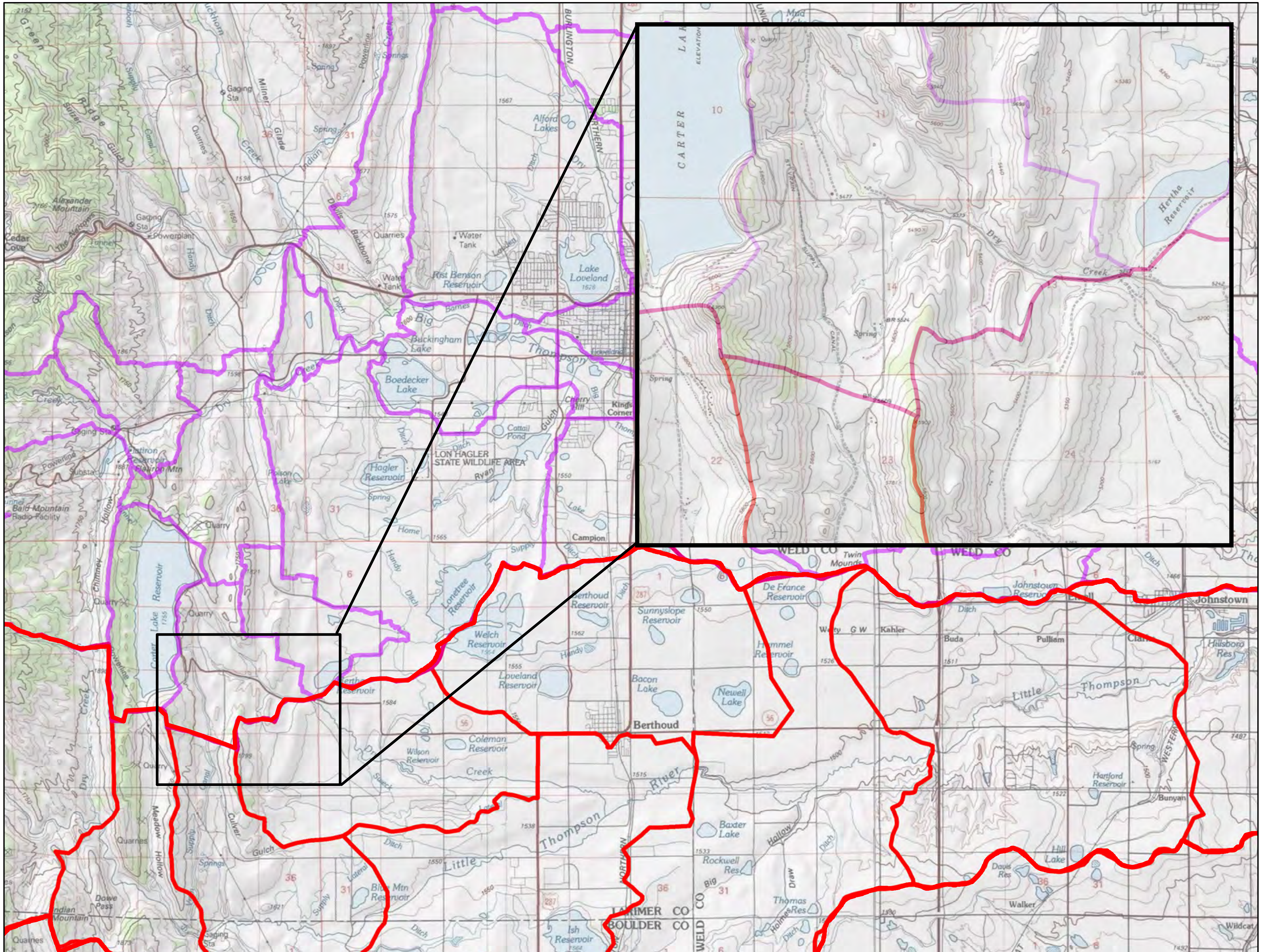
0 1.5 3 4.5 6
Miles

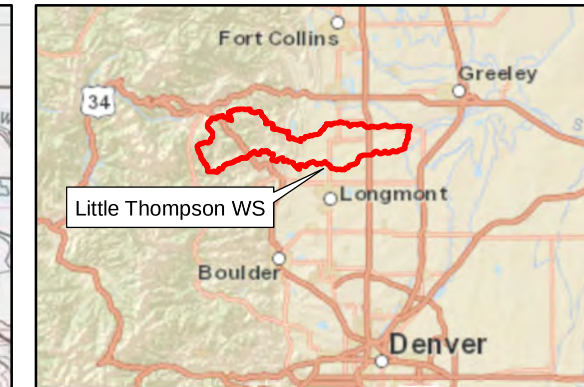
1 inch = 3 miles

FIGURE 1
Little Thompson River Overview Map

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL





LEGEND

Big Thompson

Little Thompson

N

0 0.95 1.9 2.85 3.8 Miles

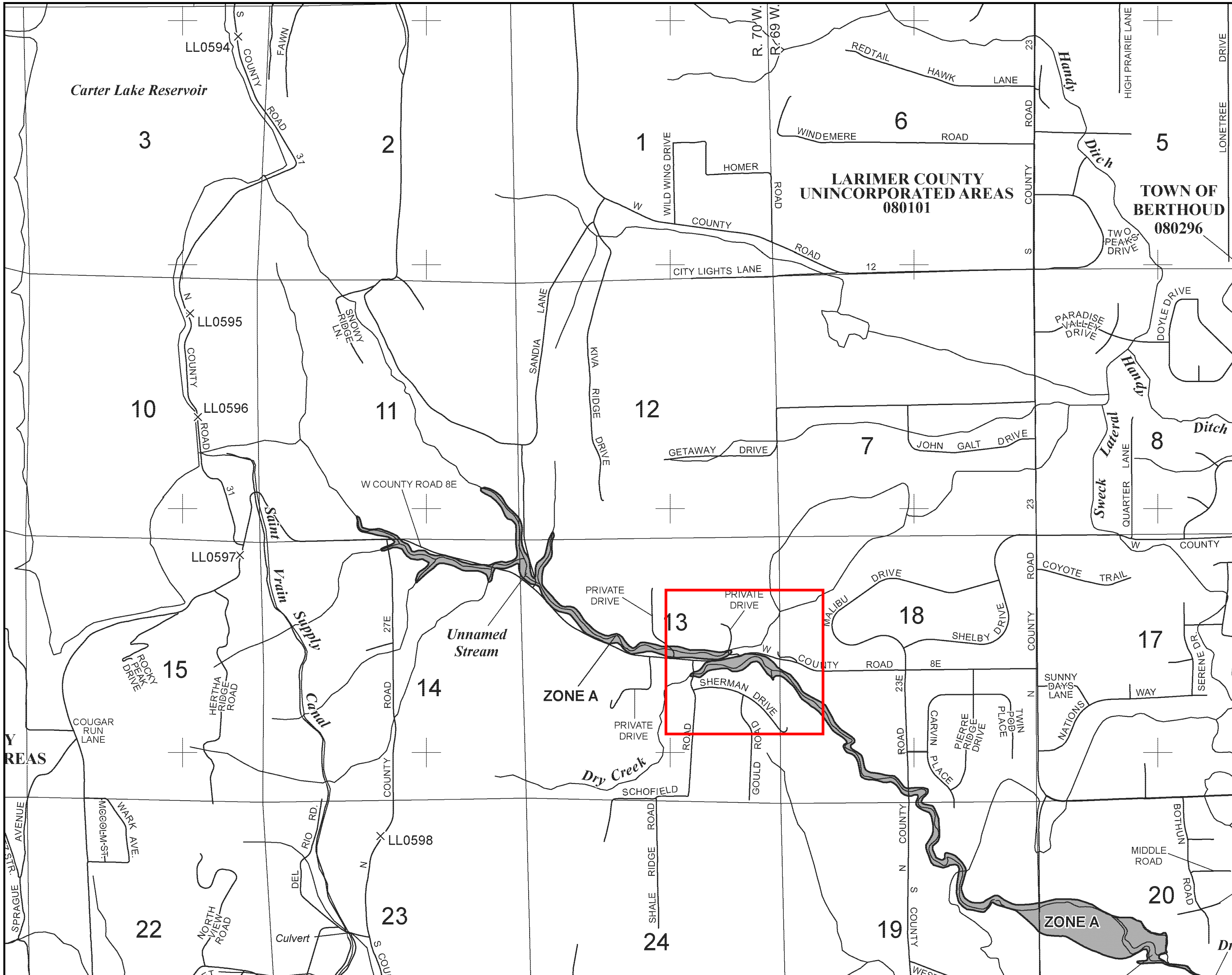
1 inch = 2 miles

Exhibit

Carter Lake Exhibit

CDOT Flood Recovery Hydrologic Evaluation

CH2MHILL



PANEL 1375G

FIRM
FLOOD INSURANCE RATE MAP

LARIMER COUNTY, COLORADO AND INCORPORATED AREAS

PANEL 1375 OF 1420
(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
LARIMER COUNTY	080101	1375	G
BERTHOUD, TOWN OF	080296	1375	G
LOVELAND, CITY OF	080103	1375	G

NATIONAL FLOOD INSURANCE PROGRAM

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER 08069C1375G

MAP REVISED FEBRUARY 6, 2013

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

Appendix G

Digital Data (Electronic Only)