Hydrologic Evaluation of the Lefthand Creek Watershed

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 Lefthand Creek Watershed was prepared by us, or under our 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 September 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.

This report documents the hydrologic evaluation for the Lefthand Creek Watershed.

Prior to September 2013, the last major flooding event in the Lefthand Creek Watershed upstream of Highway 36 was in 1969 and was caused by a combination of heavy snow and rain. The effective regulatory flow rates documented by the Federal Emergency Management Agency (FEMA) in the 2012 Flood Insurance Study (FIS) were developed in various studies between 1978 and 1983. The effective peak discharges for Lefthand Creek, James Creek and the downstream portion of Little James Creek through Jamestown were originally developed in an unpublished U.S. Army Corps of Engineers (USACE) Flood Hazard Report using the EPA Storm Water Management Model (SWMM). Peak discharges on the upstream portion of Little James Creek were based on regression equations from the USGS Technical Manual No 1. The 2012 FIS includes peak discharges for James Creek and Little James Creek based on this study; however, the peak discharges for Lefthand Creek upstream of Highway 36 were not published in the 2012 FIS. Fortunately, a 1983 Floodplain Information Report by Simons, Li and Associates includes documentation of the peak discharges determined by the USACE. Therefore, these peak discharges have been referenced in this report for comparison purposes.

In the current evaluation, a rainfall-runoff model was developed to transform the recorded rainfall to stream discharge using the U.S. Army Corps of Engineers' HEC-HMS hydrologic model (USACE, 2010). The hydrologic model was calibrated through adjustment of model input parameters that represent land cover and soil conditions. The model was calibrated to provide a best fit with peak discharges estimated for the 2013 event at a number of locations in the watershed.

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 that may be associated with this particular storm event.

Loss parameters in the rainfall-runoff model were then uniformly adjusted to provide an overall best fit with the estimated September peak discharges based on the peak 24 hours of the September rainfall rather than the entire multi-day storm. Loss parameters were also adjusted to create the same overall ratio of runoff to rainfall during the peak 24-hours of rainfall as the multi-day event. The latter adjustment was used for developing predictive estimates of 10, 4, 2, 1, and 0.2 percent annual chance peak discharges (10-, 25-, 50-, 100-, and 500-year storm events) based on a 24-hour Soil Conservation Service (SCS) Type II storm distribution and the recently released 2014 National Oceanic and Atmospheric Administration (NOAA) Atlas 14 rainfall values. It should be noted that in general, the model focuses on peak discharge estimation along the main stem channel within relatively large watershed areas. A "critical storm duration" analyses was not undertaken on smaller tributary areas to determine whether shorter, more intense rainstorms would produce greater discharges in individual tributaries.

The resulting modeled peak discharges for the various return periods were compared to the results of an updated flood frequency analysis for Lefthand Creek as well as to current regulatory discharges. The modeled peak discharges were 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. This information is shown on a log-log scale in Figure ES-1. This figure, including legend abbreviations, is discussed in more detail in the body of the report along with a comparison to other adjacent watersheds. From this figure, several observations can be made:

1. The predictive model peak discharges are slightly lower than the current regulatory peak discharges for James Creek, although typically within 20 percent.
2. The predictive model peak discharges closely match the current regulatory peak discharges and updated flood frequency analysis for the reach of Lefthand Creek downstream of the James Creek confluence.
3. The predictive model peak discharges are less than the current regulatory peak discharges for Lefthand Creek upstream of the James Creek confluence.

Figure ES-1. Comparison of 100-year Discharges in Lefthand Creek and Adjacent Watersheds

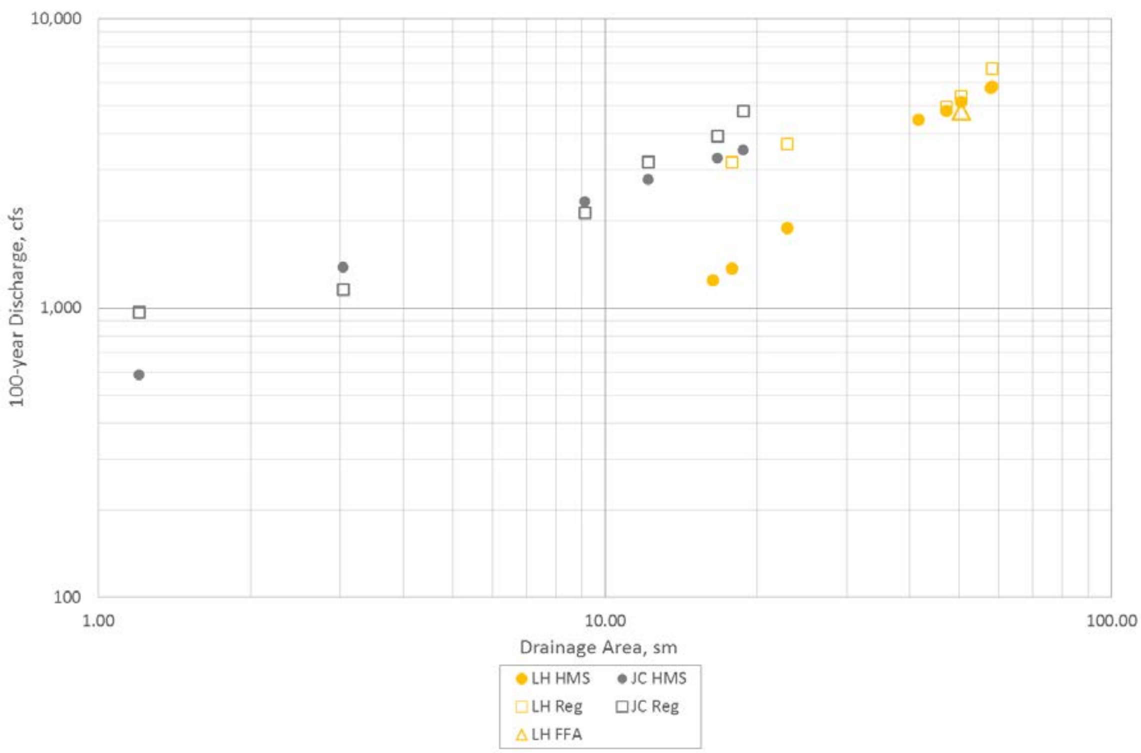

Table ES-1 summarizes the predictive model results for the 100-year event compared to current regulatory discharges. On James Creek and the lower reach of Lefthand Creek (below the confluence with James Creek), the predictive model results are generally within 20 percent of the current regulatory discharges. However, on Lefthand Creek upstream of the confluence with James Creek, the results are 50 to 60 percent lower than the current regulatory discharges. The primary reason for the large difference in 100-year unit discharges on James Creek and Upper Lefthand Creek is that the model was calibrated to the 2013 Flood estimates which varied dramatically between these two locations. The unit discharges on Upper Lefthand Creek are reasonable when compared with modeled unit discharges in the St. Vrain and Big Thompson watersheds as shown on Figure 6 in the body of the report. In contrast, the unit discharges for James Creek are significantly higher than those modeled in adjacent watersheds. The high regulatory peak discharges in the headwater areas (Upper Lefthand Creek and James Creek) are due to the critical storm duration method used in the 1983 Study.

Table ES-1. 100-year Modeled Peak Discharges Compared to Current Regulatory Discharges

Location	Current Regulatory Discharge (cfs)	Modeled Discharge (cfs)	Percent Difference
Little James Creek at upstream limit of detailed study	970	590	- 40%
Little James Creek upstream of confluence with James Creek	1,160	1,390	+ 19%
James Creek upstream of confluence with Little James Creek	2,140	2,340	+ 9%
James Creek at confluence with Little James Creek	3,205	2,780	- 13%
James Creek below Jamestown	3,930	3,300	- 16%
James Creek above confluence with Lefthand Creek	4,810	3,510	- 27%
Lefthand Creek at Lickskillet Gulch	3,180	1,370	- 57%
Lefthand Creek upstream of confluence with James Creek	3,690	1,890	- 49%
Lefthand Creek at Old Stage Road	4,940	4,800	- 3%
Lefthand Creek at Spruce Gulch	5,420	5,150	- 5%
Lefthand Creek at Highway 36	6,700	5,820	- 13%

Based on the predictive model discharges for the return periods analyzed, the peak discharges observed in the Lefthand Creek watershed during the September 2013 Flood event had an estimated recurrence interval ranging from approximately a ~~1~~ 2 percent annual peak discharge to a 0.2 percent annual peak discharge, or from a ~~100~~ 50-year to a 500-year storm event as shown in Table ES-2.

Table ES-2. Estimate of September 2013 Peak Discharge Recurrence Interval

Location	Estimated Discharge (cfs)	Annual Chance Peak Discharge (cfs)					Estimated Recurrence Interval (yr)
		10%	4%	2%	1%	0.2%	
Upper Little James Creek	1,050	187	316	442	587	1,010	~ 500
Lower Little James Creek	1,800	423	730	1,036	1,386	2,425	100 to 500
James Creek above Little James Creek	2,900	776	1,278	1,772	2,339	4,003	100 to 500
James Creek in Jamestown	4,800	912	1,502	2,095	2,777	4,834	~ 500
James Creek below Jamestown	3,300	954	1,647	2,395	3,304	6,195	~ 100
Lefthand Creek at Lickskillet Gulch	1,300	~~200~~ 364	~~410~~ 666	~~650~~ 988	~~960~~ 1,371	~~2,000~~ 2,581	~~100 to 500~~ ~ 100
Lefthand Creek below Old Stage Road	3,520	~~840~~ 1,260	~~1,640~~ 2,336	~~2,490~~ 3,438	~~3,550~~ 4,804	~~6,990~~ 9,220	~~~ 100~~ 50 to 100

Based on these comparisons, 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 at Highway 36 and further downstream where these model results will be incorporated into a separate model for the entire St. Vrain watershed. The results are available for local floodplain administrators to consider using for regulatory discharges in the Lefthand Creek watershed; however, as mentioned, the modeling effort documented herein does not focus specifically on the determination of critical storms and associated discharges in smaller tributary areas. In this case, current regulatory discharges in upper Lefthand Creek are higher than the predictive peak flows documented herein.

1.0 BACKGROUND

1.1 Purpose and Objective

In September 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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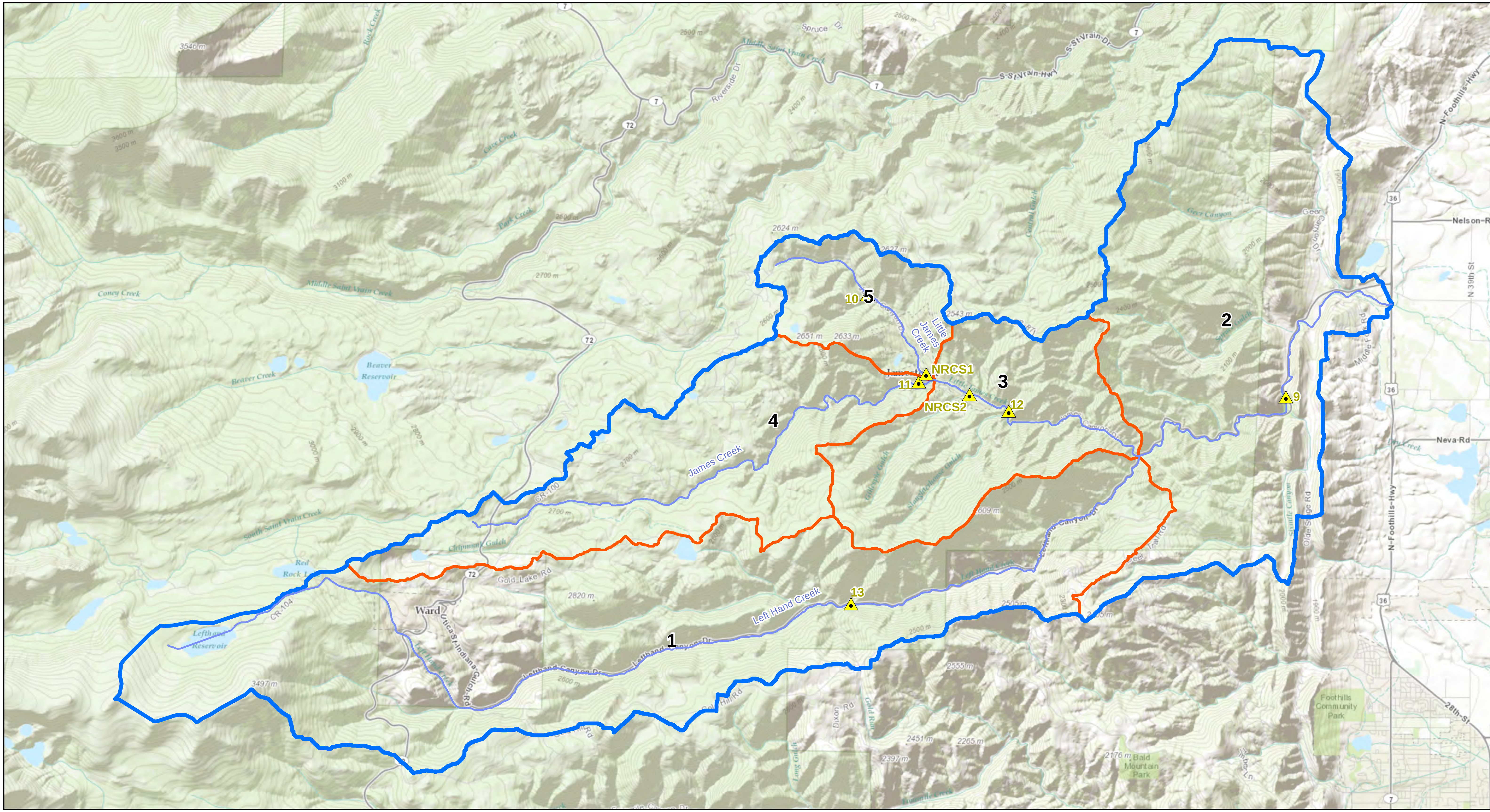
1.2 Project Area Description

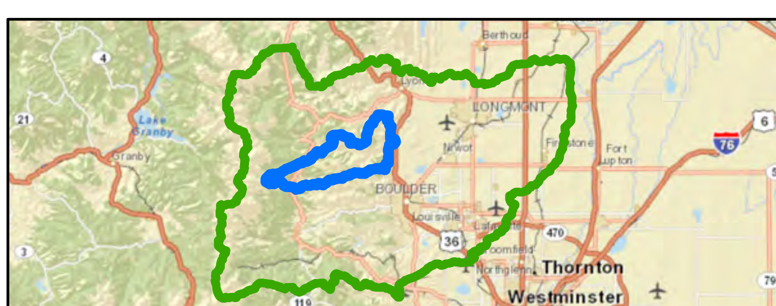
The Lefthand Creek watershed extends approximately 30 miles eastward from its headwaters in the Roosevelt National Forest near Lefthand Reservoir to its confluence with St. Vrain Creek in the City of Longmont. Most of the watershed lies in the mountains and varies in elevation from 5,600 feet to 11,000 feet. The remainder of the watershed lies in the high plains. Until recently, the floodplain was devoted entirely to agriculture. Now, because of expanding population and industrialization, urban development has begun at both ends and in the middle of the watershed. This study focuses on the upper 15 miles of Lefthand Creek with U.S. Highway 36 near the mouth of Lefthand Canyon serving as the downstream study limit. The study area encompasses approximately 58 square miles. Figure 1 provides an overview map of the study area within the much larger St. Vrain Watershed which also includes St. Vrain Creek and Boulder Creek.

Lefthand Creek flows east from Lefthand Reservoir near the headwaters and crosses State Highway 72 just south of the Town of Ward. Lefthand Creek then flows east paralleling Lefthand Canyon Drive and has two major tributaries including Spring Gulch and James Creek. The total drainage area of Lefthand Creek above the confluence with James Creek is approximately 23 square miles. Below the confluence with James Creek, Lefthand Creek flows east and north to the mouth of the canyon near U.S. Highway 36 with a total tributary drainage area of approximately 58 square miles.

Little James Creek, draining an area of approximately three square miles, flows into James Creek in Jamestown from the north, through mostly vacant land. James Creek upstream of Jamestown, drains an area of approximately nine square miles, and flows into Jamestown from the west. Existing development in Jamestown is located on both sides of James Creek, from the confluence with Little James Creek at Ward Street to 13th Street. Downstream of Jamestown, James Creek has a total drainage area of approximately 19 square miles and is a tributary to Lefthand Creek. The terrain is mountainous with steep slopes. The barren areas are predominately exposed bedrock that consists of mixed materials, including granite, sandstone, shale and limestone. The dominant land cover species is Ponderosa pine.

The climate of Boulder County is classified as semiarid. The average annual precipitation is 18.3 inches, which includes an average annual snowfall of 83 inches (2012 FIS). The average annual rainfall in the James Creek watershed is approximately 24 inches (2012 FIS). The occurrence of precipitation varies; however, most of the rainfall is concentrated in May. Thunderstorms also occur irregularly throughout the summer months.





Left Hand Creek Study Area

CDOT Flood Recovery Hydrologic Evaluation

Service Layer Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), TomTom, 2013
Sources: Esri, DeLorme, NAVTEQ, TomTom, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, and the GIS User Community

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- Investigation Sites
- Tributaries
- **Left Hand Creek Study Area**
- **Lefthand Creek Major Basins**
- **St. Vrain Watershed**

Major Basins

1: Lefthand Creek Upstream of Confluence with James Creek
2: Lefthand Creek Downstream of Confluence with James Creek
3: James Creek (Confluence with Little James Creek to Confluence with Lefthand Creek)
4: James Creek (Headwaters to Confluence with Little James Creek)
5: Little James Creek (Headwaters to Confluence with James Creek)

1.3 Mapping

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

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

1.4 Data Collection

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

1.5 Flood History

Unlike the September 2013 Flood, historical floods in Boulder County are due mainly to snowmelt combined with heavy rainfall, although heavy rainfall, especially in the form of cloudbursts, is also capable of causing flooding (2012 FIS).

Historic flooding occurred in the Lefthand Creek watershed in 1864, 1876, 1894, 1921, 1938, 1949 and 1951. There is little known regarding floods of record on Lefthand Creek within Lefthand Canyon with the exception of the James Creek watershed. The USACE Floodplain Information Report for Lefthand Creek (USACE 1969) provided some historic peaks. In 1938, a peak of 812 cfs was estimated at U.S. Highway 287 near Longmont. In June 1949 a peak of 1,140 cfs was estimated several miles above State Highway 7. In 1951, a peak of 785 cfs was estimated in the foothills (exact location not noted). The 1969 report also indicates that a stream gage was established 2.5 miles upstream of State Highway 7 in May 1929. This gage was operated between May 1929 and September 1931, between October 1947 and December 1953, between October 1955 and September 1957, and again between 1977 and 1980.

Downstream in Longmont, Lefthand Creek produced a large flood on May 7-8, 1969, with the primary damage being done to the South Pratt Parkway Bridge, which was ultimately destroyed by the floodwater. There are no existing stage data for the floods on Lefthand Creek in Longmont later than May 1957. The largest flood on record for Lefthand Creek in Longmont was the one that occurred in June 1949.

Floods in the Jamestown area usually occur during the period of May through September. Mountain snowmelt in May and June contributes significant runoff, but serious flooding does not occur unless rainfall accompanies the snowmelt (2012 FIS). Peak flooding will usually occur within a few hours after a single rainfall event. Flooding is generally of short duration, but may be prolonged significantly by snowmelt runoff.

In Jamestown, the steep stream slopes create swift currents during a flood, which produce added damages. Debris carried by the fast-moving water not only threatens bridges and culverts, but batters houses and other structures in the floodplain. The bridge and culvert crossings often result in channel restriction, raising the water surface elevations. Erosion undercuts and destroys structures that would otherwise receive little damage from inundation. Large quantities of rock are often deposited in portions of the channel.

In June 1894, a flood roared down James Creek and washed away much of the low-lying area of the town. Heavy rains accompanied by heavy spring runoff caused the flood. Most of the houses on the north side of Main Street were ruined or washed away, as was much of the road.

A similar flood occurred in August 1913, damaging or destroying almost every house along James Creek. All wagon bridges and footbridges were destroyed, and it took two weeks to open the road to traffic.

In August 1955, a brief cloudburst, lasting approximately 30 minutes, damaged four bridge and culvert crossings along James Creek and deposited several inches of mud in local residences.

Jamestown was also flooded in 1965, and again in May 1969, following three days of heavy snow and rain. The floodwaters left the normal channel, destroying a number of buildings and the town water supply.

2.0 HYDROLOGIC ANALYSIS

2.1 Previous Studies

The effective Boulder County Flood Insurance Study (FIS) was published by the Federal Emergency Management Agency (FEMA) on December 18, 2012. Therefore, the information included in the FIS was up to date and there are no known relevant studies that occurred between the FIS effective date and the September 2013 flood event. The effective regulatory flow rates documented in the 2012 FIS were developed in various studies between 1978 and 1983. The effective peak discharges for Lefthand Creek, James Creek and the downstream portion of Little James Creek through Jamestown were originally developed in an unpublished U.S. Army Corps of Engineers (USACE) Flood Hazard Report using the EPA Storm Water Management Model (SWMM). Peak discharges on the upstream portion of Little James Creek were based on regression equations from the USGS Technical Manual No 1. The 2012 FIS includes peak discharges for James Creek and Little James Creek based on this information; however, the peak discharges for Lefthand Creek upstream of Highway 36 were not published in the 2012 FIS. Fortunately, a 1983 Floodplain Information Report by Simons, Li and Associates includes documentation of the peak discharges determined by the USACE. Therefore, these peak discharges have been referenced in this report for comparison purposes. Table 1 provides a summary of the relevant peak discharges provided in the 2012 FIS and the 1983 Floodplain Information Report.

Table 1. Select Peak Discharge Values from 2012 FIS and 1983 Floodplain Information Report

Flooding Source and Location	Drainage Area (sq.mi.)	Peak Discharge (cfs)			
		10-yr	50-yr	100-yr	500-yr
Lefthand Creek					
At Highway 36	--	1,035	4,145	6,700	14,990
Upstream of Spruce Gulch	--	890	3,190	5,420	12,540
Upstream of Six Mile Canyon	--	830	2,850	4,940	11,630
Upstream of James Creek	--	430	2,100	3,690	10,430
Upstream of Lickskillet Gulch	--	400	1,800	3,180	9,060
At Peak to Peak Drive	--	60	360	600	1,460
James Creek					
At Confluence with Lefthand Creek	18.7	390	2,660	4,810	13,500
At Cross Section A	14.5	355	2,180	3,930	10,880
At Main Street Bridge	12.2	300	1,785	3,205	8,850
At Confluence of Little James Creek	12.1	300	1,760	3,160	8,725
At Upstream Limit of Detailed Study	8.9	200	1,190	2,140	6,010
Little James Creek					
At Confluence with James Creek	2.8	130	650	1,160	3,220
At Confluence of Balarat Creek	2.25	130	650	1,160	3,220
At Upstream Limit of Detailed Study	1.8	109	544	970	2,690

2.2 September 2013 Peak Flow Estimates

Estimates of peak discharges associated with the September flood event based on field observations were undertaken by Bob Jarrett of Applied Weather Associates (AWA) as documented in the report *Peak Discharges for the September 2013 Flood in Selected Foothill Region Streams, South Platte River Basin, Colorado*. Over a long career with the USGS, Bob has developed techniques for making peak discharge estimates based on observations of high water marks and paleoflood evidence. Some of the important elements involved in making appropriate estimates include finding a suitable location on the river, accounting for the high hydraulic roughness that can develop during large floods, and factoring in the influence of sediment and debris. A brief description of the observation and discharge estimation techniques is included in Appendix A.

Key locations in the Lefthand Creek Watershed were identified, mapped, and prioritized for use by Bob in the field observations and estimates. The discharge estimates provided by Bob Jarrett, as well as any other available discharge estimates in the watersheds, were compared to the current regulatory discharges to provide an initial assessment of the relative magnitude of the September floods. This information is documented in a memo entitled *CDOT/CWCB Hydrology Investigation Phase One – 2013 Flood Peak Flow Determinations*, dated January 21, 2014 and revised on July 16, 2014. This memo is also included in Appendix A. Peak discharge estimates indicated in the July 16 memo are preliminary and subject to revision based on subsequent evaluations and comparisons.

Some of the discharge estimates were greater than what would be expected given the tributary drainage basin characteristics, rainfall amounts and rainfall intensities measured during the storm. This information along with field observations by Bob Jarrett have led us to the hypothesis that dam failures (including woody debris dams, road-embankments, beaver dams, stock ponds, and landslides) may have played a role in this flood. Post-flood aerial imagery in the study watersheds showed evidence of dam failures, mostly from debris flows, associated temporary debris dams, and catastrophic/sudden failures including the release of groundwater in landslides. These various dam failures may have resulted in dramatic peak flows, but because these dams have so little volume, attenuation of these peak flows downstream can also be dramatic. A USGS report (Godt et al., 2013) discussing landslides caused by the 2013 rainfall states that:

> *"debris flows exacerbated flooding by supplying sediment to stream valleys. This sediment was mobilized by floods and in some cases caused surging flood pulses that destroyed buildings and infrastructure."*

2.3 Updated Flood Frequency Analysis

Flood frequency analyses were performed to supplement the hydrologic evaluation in the Lefthand Creek watershed. 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 September 1981. This document is commonly known as *Bulletin 17B*.

Following the Bulletin 17B methods within the computer program HEC-SSP, Ayres Associates conducted the analysis using the annual peak flow records at the Lefthand Creek stream flow gage. Only 17 years of data were available for the gage including the 2013 estimate. In 2012, the CDWR took over operation of the gage but no peaks were recorded in 2012 or during the 2013 flood.

Lefthand Creek near Boulder

- USGS Gage 06724500 (1929 – 1980, broken)
- CDWR Gage LEFCRECO (2012 – 2012, no peak included)

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 (for the analyses reflected in the draft report) 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.

However, external review of the 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.

The detailed input to and output from HEC-SSP based on the revised approach using station skew is included in Appendix B. The results are summarized in Table 2 below. Based on these results, the estimated 2013 discharge of 3,520 cfs near Old Stage Road was between a 50-year and 100-year event.

Table 2. Results of Flood Frequency Analysis for Lefthand Creek near Boulder

Exceedence Recurrence Interval (years)	Lefthand Creek near Boulder (cfs)	5% and 95% Confidence Limits (cfs)	
2	338	243	456
5	676	499	1,008
10	1,091	768	1,862
50	3,110	1,830	7,709
100	4,810	2,595	14,090
500	12,965	5,685	56,120

The gage record for Lefthand Creek was limited to only 17 years of data and the analysis resulted in very wide confidence limits for storms greater than a 10-year event; therefore the flood frequency analysis for Lefthand Creek was not heavily relied on as a basis of calibration in this study. Reliable flood-frequency relations are difficult to estimate when using short gage record lengths, 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 2013).

2.4 Rainfall / Runoff Model for September, 2013 Event

2.4.1 Overall Modeling Approach

A hydrologic analysis was performed on the Lefthand Creek 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 within Lefthand Creek, James Creek, and Little James Creek.

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. 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 later in this report, 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 in the Appalachian Mountains 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 following sections discuss the steps undertaken during the rainfall/runoff modeling. Associated information is included in Appendix C, 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. DEMs are 3-D base maps, which HEC-GeoHMS uses 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 South St. Vrain Creek watershed to the north, the Continental Divide to the west, and the Boulder Creek watershed to the south. With the downstream limit of the study set at U.S. Highway 36, basins were delineated around all reaches and confluences. The overall watershed was divided into 18 basins ranging from 0.25 square miles to 15 square miles. Basins were manually subdivided where necessary in order to compare peak discharge estimates at investigation sites with results from the hydrologic model. The seven peak discharge estimation locations for the 2013 flood are:

1. Upper Little James Creek
2. Lower Little James Creek above confluence with James Creek
3. James Creek upstream of confluence with Little James Creek
4. James Creek in Jamestown
5. James Creek below Jamestown
6. Lefthand Creek upstream of Rowena near Lickskillet Gulch
7. Lefthand Creek downstream of Old Stage Road

2.4.3 Basin Characterization

The Lefthand basin is rural, mountainous and mostly wooded with steep valley sides draining laterally toward central stream features. James Creek is the other major creek that contributes to the watershed. The total watershed for the study area is approximately 58 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 30 to as high as 90. 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 uses a peaking coefficient and the standard lag time as required input parameters. A default peaking coefficient of 0.4 was initially selected for all basins as being representative of mountain areas. The lag time was calculated using Equation CH9-510 and Table CH9-T505 in the CWCB Floodplain and Stormwater Criteria Manual. A default Kn value of 0.15 for evergreen forests was 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.

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, slope and 8-point cross-section station-elevation data of the channel reaches were acquired using the HEC-GeoHMS program and the 10-meter DEM and DRG datasets. The Manning's n values were initially set to a default of 0.05 for the channels and 0.10 for the overbank areas. Reservoir storage was not accounted for anywhere in the Lefthand Creek model.

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 base maps 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 base maps 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.

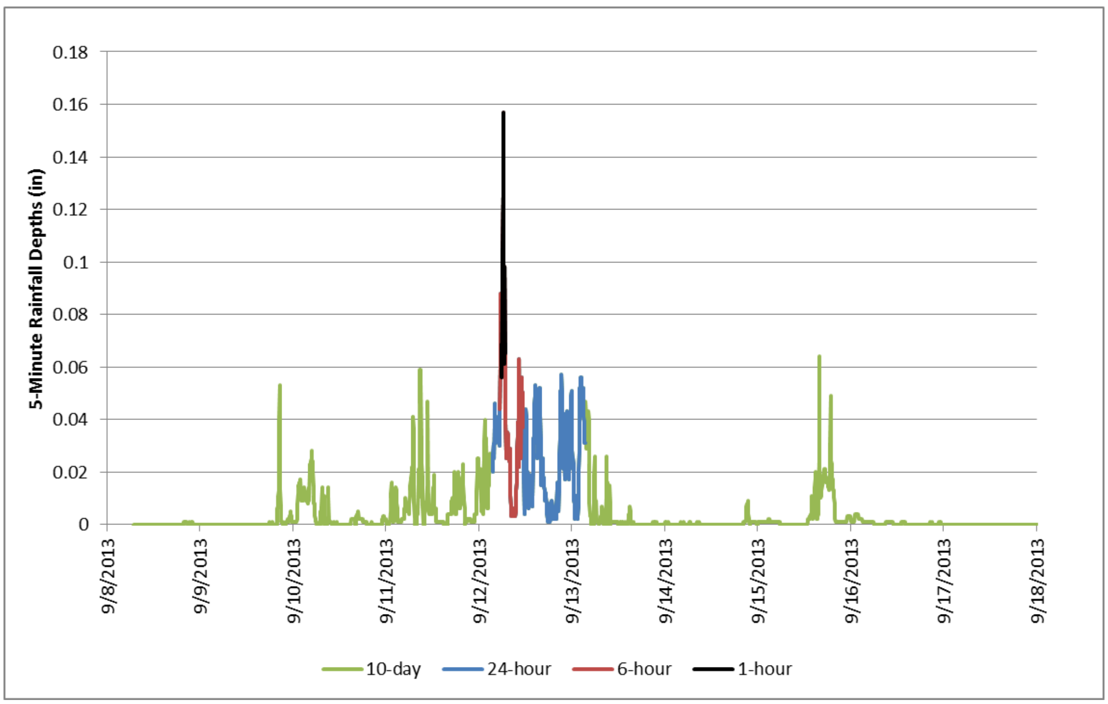
In data sparse regions where there are a limited number of rain gages, there can be increased uncertainty in traditional rainfall analyses, especially in topographically significant regions. For the September 2013 storm, this was not the case in most places. There was excellent weather radar coverage 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 highly 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 was 14.20 inches, ranging from as low as 8.89 inches up to 17.17 inches for the individual basins. However, the majority of this rainfall fell within a 24-hour period starting around 3 A.M. on Thursday, September 12, 2013. The average 24-hour rainfall depth for all of the basins was 7.67 inches, ranging from 3.21 inches up to 9.73 inches for the individual basins. The average 24-hour rainfall depth of 7.67 inches roughly corresponds to between a NOAA 500-year and 1000-year 24-hour rainfall depth. Table 3 shows the September 2013 rainfall depths for various durations in five representative basins from the study area. It also shows the associated NOAA Atlas 14 recurrence interval for each depth-duration pair.

Figure 2 shows a hyetograph for Basin LH260 on James Creek from Jamestown to the confluence with Lefthand Creek. The incremental depths are based on a 5-minute time step. As shown in Table 3, Basin LH260 experienced average rainfall totals and intensities for the 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 exceeds a 1000-year event, the maximum 24-hour rainfall total is approximately a 1000-year event, the maximum 6-hour rainfall total is approximately a 50-year event, and the maximum 1-hour rainfall total is only a 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 extreme.

Figure 2. September 2013 Rainfall Hyetograph on James Creek below Jamestown



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.

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

Location	Little James Creek (LH220)		James Creek Headwaters (LH240)		James Creek below Little James Creek (LH260)		Sixmile Canyon at Old Stage Road (LH280)		Lefthand Creek at Headwaters (LH300)	
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	13.87	> 1000	10.94	500	14.52	> 1000	17.17	> 1000	8.89	200
24-hour	7.43	1000	5.36	200	7.96	1000	9.73	> 1000	3.21	25
6-hour	2.69	50	2.12	25	2.78	50	4.05	200	1.23	2 to 5
1-hour	0.91	5	0.55	1	1.10	5	1.29	10	0.38	< 1

2.4.6 Model Calibration and Validation

The first step in the model development and calibration process was to setup a working base model. 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. Once the base model was up and running correctly with the default input parameters, the second step was to begin calibrating the model to the estimated peak discharges for the 2013 Flood event.

Many of the model input parameters for the model 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. Four 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), and Channel Roughness (Manning's n).

In order to determine the sensitivity of each of the four calibration parameters, attempts to calibrate the entire watershed using only one parameter at a time were conducted. From this analysis, it was determined that the peak flows and timing of peaks were most sensitive to the CN value selected for each basin as explained 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 multiple 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. Various cross-section shapes were also evaluated with little effect. After some additional research, it was concluded that the Muskingum-Cunge method, as well as several of the other HEC-HMS routing options, are highly sensitive to channel slope. The relatively steep mountain slopes within the study area were therefore the predominant factor in channel routing calculations and limited the effect the roughness calibration as a calibration parameter for adjusting travel times and coincidental peaks. Further review of literature, specifically reports by Jarrett (1985) and Barnes (1967) regarding the appropriate Manning's n values for mountain streams was conducted and it was determined that a value of 0.15 was appropriate for the channels in this watershed.

After conducting the sensitivity analysis on the individual calibration parameters, additional attempts were made to get a best fit to the 2013 Flood peak discharge estimates by calibrating the CN, Cp, and Kn values simultaneously. However, it was subsequently determined that focusing the calibration effort on the CN value while holding the other parameters at reasonable default values was the most justifiable method. This decision was supported by calibration efforts being performed concurrently in the St. Vrain Creek Watershed and Big Thompson River Watershed. While the combined calibration approach was being conducted, the U.S. Bureau of Reclamation provided a stage-storage relationship for Lake Estes in the Big Thompson Watershed, along with stage-storage-discharge time-series data during the 2013 Flood event. This valuable information allowed better calibration and optimization routines to be run on the Big Thompson Watershed upstream of Lake Estes with respect to timing, volume and peak discharges based on a calculated inflow hydrograph to Lake Estes. Unfortunately, no real-time hydrographs were available in the Lefthand Creek Watershed. The Lake Estes optimization results showed that the CN value was the dominating calibration factor. The Lake Estes optimization results for the Big Thompson Watershed are included in Appendix C.5 for reference.

Calibrating the model to match the peak discharge estimates was relatively straightforward at most locations. 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 runoff produced from a single basin prior to any channel routing would only be a small fraction of the peak discharge estimate at that same location. In these cases, all attempts were made to double check measured input parameters for errors including basin area, length, length to centroid, slope, and associated rainfall data.

Attempts were also made to maximize peak discharges by raising composite CN values to 98, increasing the peaking coefficient and shortening the lag time. Even with all of these calibration parameters maximized, the modeled peak discharges were often still not close to the estimated peak discharge. There were also locations where peak discharge estimates (and associated unit discharges cfs/sq.mi.) fluctuated up and down within short reaches when moving downstream through the watershed. Upon further discussion with the project team and review of available field data, it was hypothesized that several locations in the watershed experienced some form of a dam failure (possibly from woody debris dams, road-embankments, beaver dams, stock ponds, or landslides) that generated peak discharges significantly higher than the rainfall/runoff process alone would have produced. Evidence of these types of dam failures and resulting high, short peak discharges from the 2013 Flood was documented by a USGS report (Godt et al., 2013).

Additional analysis was undertaken to develop expected unit discharges (cfs/sq.mi.) at the estimation locations for all watersheds being studied by CDOT. These unit discharges were then compared against one another as well as against model results throughout each of the study watersheds. In addition, unit discharges were also normalized with respect to the peak 1-hour rainfall experienced in the corresponding basins. Graphical curves were developed to provide a best-fit to this unit discharge data. This information and the best-fit curves helped to identify peak discharge estimates that were likely impacted by phenomenon other than the natural rainfall/runoff process. In these locations, attempts were made to calibrate the models while considering the “natural” flow that would be expected based on the unit discharge curves. After several iterations of calibrating the model, it was determined that a relatively good fit to the estimated peak discharges had been obtained. Calibration results for the 10-day 2013 Flood event are discussed in more detail in Section 3.0 of this report.

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, described below.

2.5.2 Design Rainfall

The NOAA Atlas 14, Volume 8 was used to determine point precipitation frequency estimates. Isopluvials for 24-hour precipitation depths were overlaid with the basin delineation maps to determine the variation in rainfall depths within the watershed. Based on the isopluvials, the Lefthand Creek Watershed was broken into two raingage zones corresponding with basin boundaries. Latitude and Longitude values were determined for the centroid of each raingage zone in order to obtain precipitation frequency estimates. Table 4 below and Appendix C.6 show the point precipitation values for each of the raingage zones and the basins included in each zone. Table 4 also shows the 90 percent confidence intervals on the rainfall depths which expresses some of the uncertainty associated with the rainfall data. Zone 1 included 6 basins along the western side of the watershed from approximately Rowena up to the headwaters. Zone 2 included 12 basins in the eastern part of the watershed. 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 4. Lefthand Creek Raingage Zones and Precipitation Depths

Zone	Zone 1 (West)	Zone 2 (East)
Latitude	40.08393003	40.11537159
Longitude	-105.468383	-105.34358
Model Basins	LH220A, LH220, LH240, LH300, LH290, LH270A	LH260, LH260A, LH270, LH250, LH280, LH230, LH210, LH200, LH160, LH170, LH180, LH190
10-yr, 24-hr	2.74 (2.23 – 3.35)	2.94 (2.42 – 3.56)
25-yr, 24-hr	3.45 (2.76 – 4.51)	3.72 (3.00 – 4.78)
50-yr, 24-hr	4.08 (3.16 – 5.38)	4.40 (3.44 – 5.71)
100-yr, 24-hr	4.76 (3.56 – 6.46)	5.14 (3.87 – 6.85)
500-yr, 24-hr	6.62 (4.55 – 9.55)	7.13 (4.91 – 10.0)

Due to the size of the Lefthand Creek Watershed (approximately 58 square miles) it was necessary to consider area correction of the rainfall depths as described in NOAA Atlas 2. For the 24-hr storm duration, rainfall depths are reduced by as much as 4% 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. Between 30 and 58 square miles, a 4% reduction was applied. Appendix C.6 shows the reduction at each design point in the model. To evaluate the area corrections, the entire watershed was run with three different sets of rainfall depths for each return period corresponding to the different levels of area correction. The appropriate peak discharge result at each location in the watershed was then selected based on its relative location with respect to total tributary area. This results in unadjusted rainfall depths being used to generate peak flows in the headwater areas, while the area corrected rainfall depths are used as you move progressively downstream. This is described in more detail in Appendix C.6.

2.5.3 Model Calibration

Initial results from the NOAA rainfall predictive model produced peak discharges that were considerably lower than the current regulatory discharges and expected unit discharges. Further analysis of the predictive model results showed that a large percentage of the rainfall in the SCS 24-hour distribution was being removed by the initial abstraction component of the CN infiltration method. This large initial abstraction was resulting in limited rainfall becoming runoff. This raised questions regarding the differences between the SCS 24-hr rainfall distribution and the 2013 storm event which had a long duration with a lower intensity. After some consideration, it became apparent that the calibrated CN values for the 10-day storm were highly dependent on the rainfall early in the storm that saturates the soil prior to the peak rainfall occurring. This also raised some concerns about the applicability of the CN infiltration method. Known limitations 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 also evaluated to see if they responded differently to the 10-day vs. 24-hr rainfall duration.

In order to most efficiently evaluate the different infiltration methods, the optimization routines in HEC-HMS were utilized in the 10-day model representing the September 2013 storm. The optimization feature allows the user to specify which model input parameters will be optimized in an attempt to produce runoff that matches an observed hydrograph. Unfortunately, there were no observed hydrographs in the Lefthand watershed during the 2013 Flood event. However, since the Big Thompson watershed had available time-series data at Lake Estes from the 2013 Flood, an inflow hydrograph to Lake Estes was developed based on the observed stage-storage and stage-discharge information provided by the U.S. Bureau of Reclamation. Therefore, the Big Thompson 10-day model was used to test the various infiltration methods. Within HEC-HMS, the Nedler and Mead search method was utilized with a Peak Weighted Root Mean Square objective function. This means that the infiltration parameters for basins upstream of Lake Estes were iteratively adjusted in an attempt to match the above average peak flow values in the observed hydrograph. The parameters were iteratively adjusted using a scaling factor so that all basin parameters were affected in a consistent manner. Several optimization scenarios were run for the different infiltration methods including:

- Constant Loss Method – optimizing Initial Loss and Constant Loss
- CN Method – optimizing CN value and Initial Abstraction
- CN Method – optimizing CN value only
- Exponential Loss Method – optimizing Initial Range, Initial Coeff, Coeff Ratio, and Exponent
- Exponential Loss Method – optimizing Initial Range, Initial Coeff, and Coeff Ratio
- Green-Ampt – optimizing Initial Loss, Moisture Deficit, Wetting Front Suction, and Hydraulic Conductivity

After reviewing results for the optimization scenarios outlined above (included in Appendix C.5 for reference), it was apparent that the CN Method was actually able to produce the best fit to the observed inflow hydrograph at Lake Estes. Although the CN method has its limitations, it is suitable for large return period storm events. Additionally, since it is being used as a calibration parameter, the actual selection of a default value for forested areas is not critical. To further support the continued use of the CN method, the other infiltration methods had limitations which deterred their use for this project.

After deciding to stay with the CN Method for all watersheds being studied, it was still necessary to address the 10-day storm vs. NOAA 24-hour rainfall duration. Therefore, it was decided to extract the maximum 24-hour period of rainfall from the 10-day period of data and re-calibrate the model. The goal 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. At a conceptual level, the idea of adjusting the CN values for all basins in the St. Vrain Watershed to Antecedent Runoff Condition (ARC) 3 seemed like a good starting point in the calibration process since the early wetting period of the storm had been removed. Chapter 10 of the National Engineering Handbook Part 630 was used to determine the ARC 3 CN value for each basin.

For ARC 3, the average CN value for the watershed (average of all individual basin CN values) increased from 64 for the 10-day model to 80 for the Max24hr model. The model results after adjusting the CN value to ARC 3 were still lower than the estimated 2013 Flood peak discharges and the 10-day model results. Therefore, further calibration was conducted to match the estimated peak discharges resulting in an average CN value of 87 for the watershed. However, when initial attempts to use the Max24hr CN values with the SCS 24-hour rainfall distributions were made, the resulting peak discharges were extremely high and did not agree with expected unit discharges or the updated flood frequency analysis. Further investigation revealed that the average 24-hour maximum rainfall for Lefthand Creek was a smaller percentage of the average 10-day rainfall than in the Big Thompson watershed and thus curve numbers calibrated for the 24-hour rainfall would have to be inordinately high to compensate for the lower rainfall. The difference between the average 10-day rainfall (14.20 inches) and the average 24-hour maximum rainfall (7.67 inches) for the Lefthand Creek Watershed was 6.53 inches. Therefore, it should be expected that the high CN values would be necessary to produce the same peak discharges when only using roughly half of the rainfall total.

This information made it apparent that in order to develop a calibrated model based on the maximum 24-hour rainfall period, it was necessary 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 CN values necessary to produce the expected runoff depths for each individual basin. The end result was an average CN value of 66 for the watershed with individual basin CN values ranging from 48 to 77. Appendices C.1 through C.4 include the model results for the Max24hr rainfall period utilizing the CN values that match the peak discharge estimates as well as the runoff/rainfall ratio determined CN values. The peak discharges for the runoff/rainfall ratio determined CN values are approximately 40% lower than for the 24-hour CN values required to match the 10-day peak discharge estimates.

Using the calibrated Max24hr runoff/rainfall ratio model, the NOAA 24-hour rainfall depths and SCS Type 2 storm distributions were applied for each of the return periods. As a reasonableness check, the predictive model results were compared to expected unit discharges upstream of Highway 36. These reasonableness 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 5 (below) and the expanded table in Appendix C.1 show results at selected locations along the channels of Lefthand Creek, James Creek and Little James Creek. Location descriptions and tributary drainage areas are provided for each location. The table in Appendix C.1 also includes approximate river stationing and the corresponding model node for each location. Estimated peak discharge values from the 2013 Flood were developed by Bob Jarrett and the NRCS and are provided at a few locations. The next column presents the calibrated model results for the full 10-day rainfall period. 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 C.1 also includes the 2012 Effective FIS and 1983 Floodplain Information Report peak discharges 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. The expanded table in Appendix C.1 also includes the updated Flood frequency analysis results by Ayres Associates for the 10-, 50-, 100-, and 500-year recurrence intervals.

Table 5. Hydrologic Model Peak Discharge Results

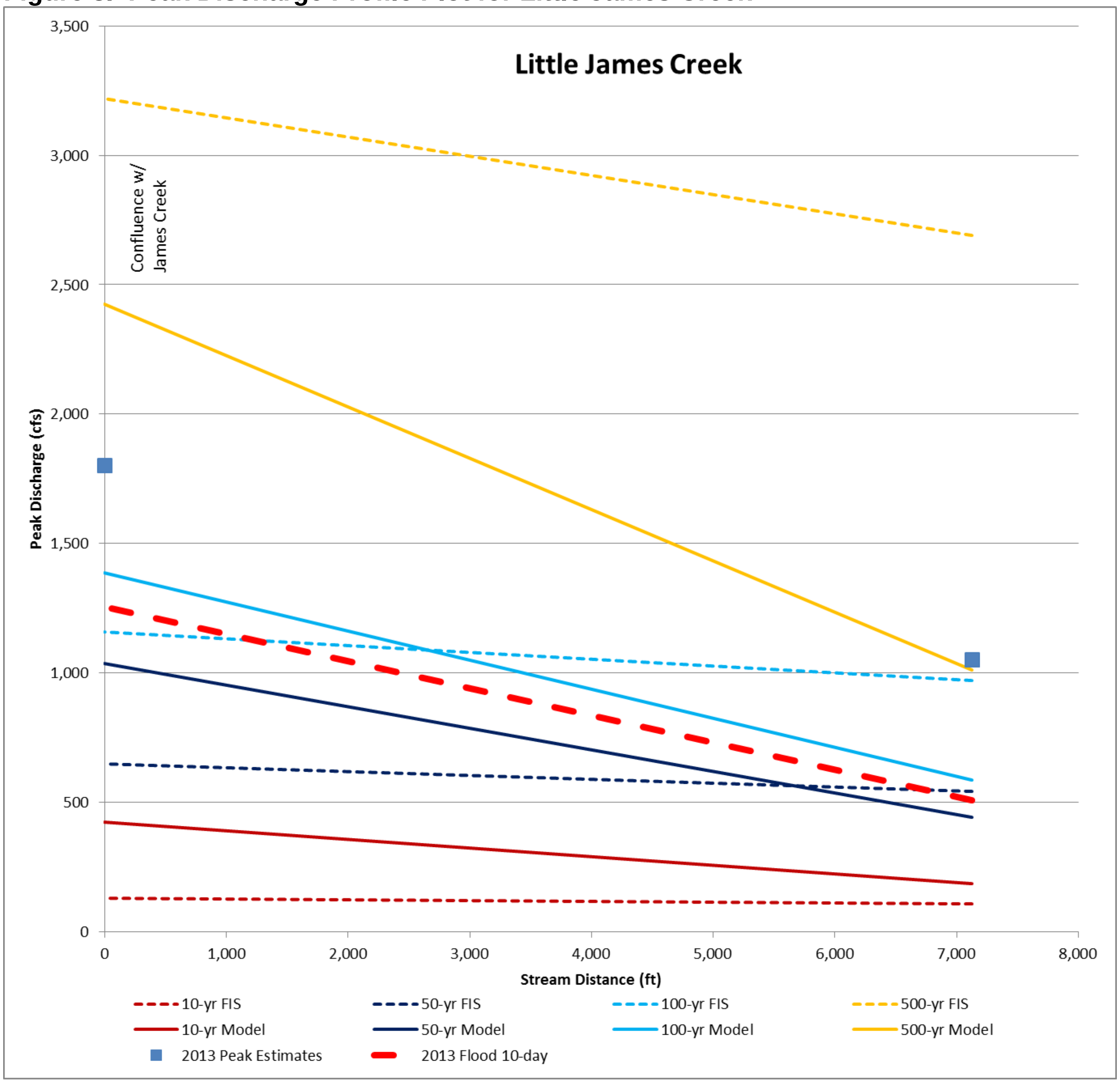
Description	Drianage Area (sq. mi.)	2013 Flood Estimated Peak Discharge (cfs)	2013 Flood 10-day Calibrated (cfs)	NOAA Design Storms (Depth-Area Adjusted) 10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)
Upper Little James Creek (Jarrett #10)	1.20	**1,050**	**508**	**187**	**316**	**442**	**587**	**1,010**
Lower Little James Creek (NRCS 1)	3.04	**1,800**	**1,256**	**423**	**730**	**1,036**	**1,386**	**2,425**
James Creek above Little James Creek (Jarrett #11)	9.13	**2,900**	**2,197**	**776**	**1,278**	**1,772**	**2,339**	**4,003**
Confluence of James and Little James (NRCS 2)	12.17	**4,800**	**3,250**	**912**	**1,502**	**2,095**	**2,777**	**4,834**
James Creek below Jamestown (Jarrett #12)	16.69	**3,300**	**3,824**	**954**	**1,647**	**2,395**	**3,304**	**6,195**
James Creek at Confluence with Lefthand Creek	18.72		4,095	973	1,710	2,517	3,512	6,728
Confluence of LHC and Spring Gulch	16.35		1,325	333	606	898	1,246	2,349
Lefthand Creek at Rowena (Jarrett #13)	17.79	**1,300**	**1,548**	**364**	**666**	**988**	**1,371**	**2,581**
Lefthand Creek above James Creek	22.92		2,626	502	920	1,359	1,886	3,536
Confluence of Lefthand Creek and James Creek	41.64		6,687	1,212	2,214	3,227	4,472	8,439
Lefthand Creek below Old Stage Road (Jarrett #9)	47.29	**3,520**	**7,734**	**1,260**	**2,336**	**3,438**	**4,804**	**9,220**
LHC confluence with Spruce Gulch	50.54		8,461	1,340	2,493	3,678	5,151	9,975
Confluence of LHC and Geer Canyon	57.72		9,394	1,457	2,743	4,085	5,771	11,388
Lefthand Creek at Hwy 36	58.12		9,474	1,469	2,765	4,117	5,822	11,493

Three peak discharge profile plots are also provided on Figures 3, 4 and 5 for Little James Creek, James Creek, and Lefthand Creek, respectively. The Effective FIS peak discharges from the table 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. The estimated peak discharges and flood-frequency results are plotted as points on the profile plots.

On Little James Creek (Figure 3), the 10-day calibrated model was unable to generate peak discharges as high as the 2013 flood discharges estimated from field observations. The peak discharge estimates on Upper Little James Creek (1,050 cfs) and Lower Little James Creek (1,800 cfs) had unit discharges of 878 cfs/sq.mi. and 593 cfs/sq.mi., respectively. These are extremely high unit discharges and may be due to debris flow and/or possible landslides as described earlier in the report. Similarly on James Creek upstream of the confluence with Little James Creek (Figure 4), the estimated peak discharge of 2,900 cfs (318 cfs/sq.mi.) was higher than the rainfall/runoff model was able to produce. Additional discussions with Bob Jarrett revealed that these estimates had a higher range of uncertainty due to the debris evidence present during field investigations. Attempts to calibrate to these high peak discharges resulted in calibration parameters outside of acceptable ranges and produced peak discharges downstream in James Creek and Lefthand Creek that were much higher than estimates in those locations. The high unit peak discharges and debris flows experienced in Little James Creek and upper James Creek appear to have attenuated as they traveled downstream to the estimation locations below Jamestown and on Lefthand Creek near Old Stage Road. This is consistent with the fact that debris failures result in dramatic peak flow surges, but because these dams have little volume, attenuation of these peak flows downstream can also be dramatic.

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.

Figure 3. Peak Discharge Profile Plot for Little James Creek

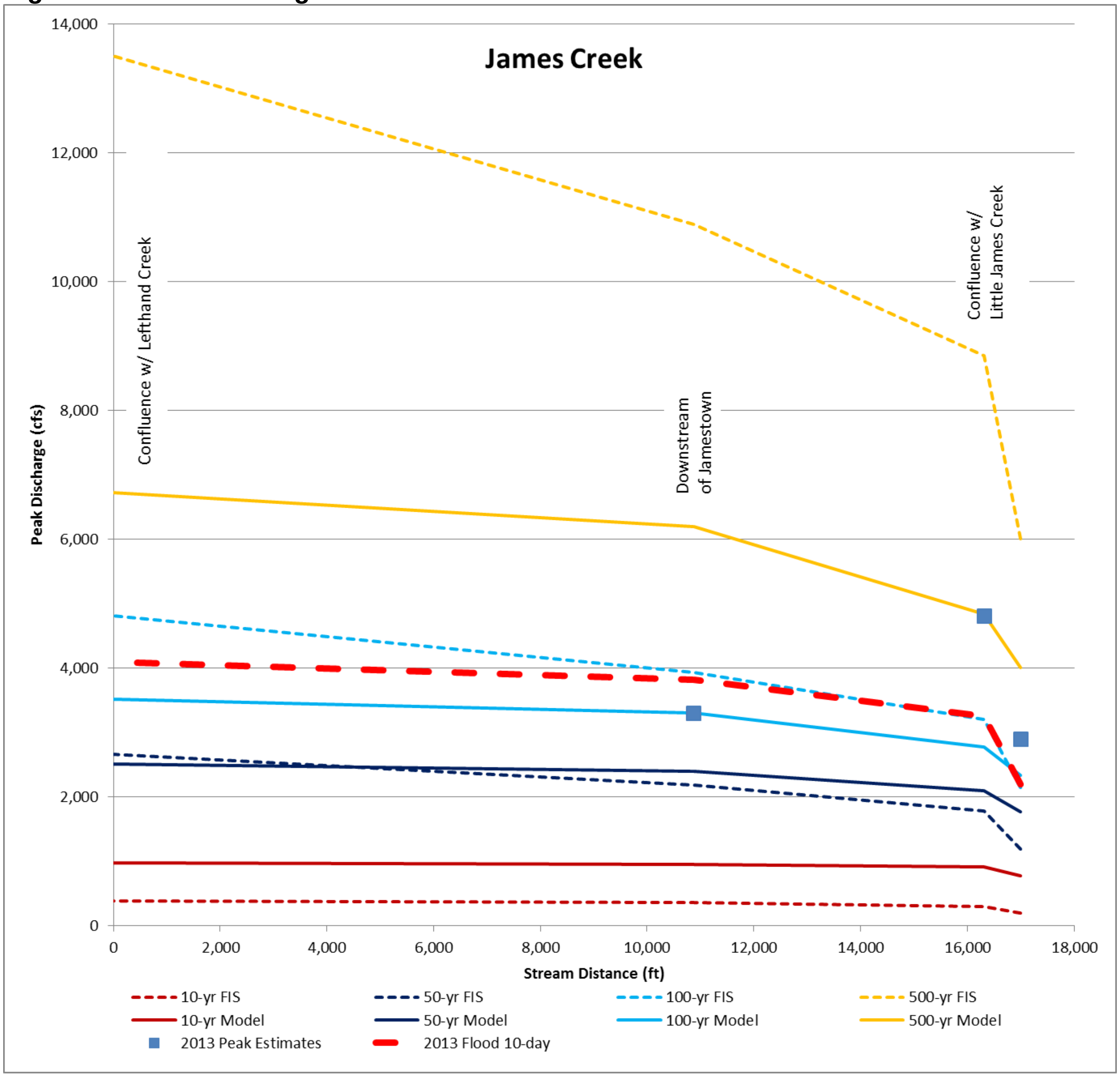


On Little James Creek, the calibrated 10-day model was approximately 30% to 50% lower than the estimated peak discharges. Similarly, James Creek above the confluence with Little James Creek was also 24% below the estimated peak discharge. At the confluence, the calibrated 10-day model peak discharge is approximately 32% lower than the NRCS estimate in Jamestown. Downstream of Jamestown, the 10-day model peak discharge is approximately 16% higher than the estimated peak discharge (3,820 cfs vs. 3,300 cfs). Therefore, it is hypothesized that the higher estimated peak discharges in the headwater reaches are due to debris dam failures and/or debris flows and that these surges are attenuated through Jamestown. As a result, the model is able to match the estimated peak discharge downstream of Jamestown within the range of hydrologic uncertainty.

On Little James Creek, it can be seen that the modeled 100-year storm is approximately 20% higher (1,390 cfs vs. 1,160 cfs) than the Effective FIS discharge value just upstream of the confluence with James Creek, yet it crosses below the effective FIS discharge profile approximately 0.5 miles upstream. However, it should be noted that the regulatory discharge on the upper reach of Little James Creek was determined using regression equations as opposed to the SWMM model. Similarly, the 50-year model results overlapped the Effective FIS discharges (higher downstream and lower upstream). The 10-year model results were consistently higher than the Effective FIS discharges. The 500-year model results were approximately 25% lower than the Effective FIS discharge values upstream of the confluence with James Creek; however, the effective peak discharges were based on extrapolated rainfall depths for the 500-year event.

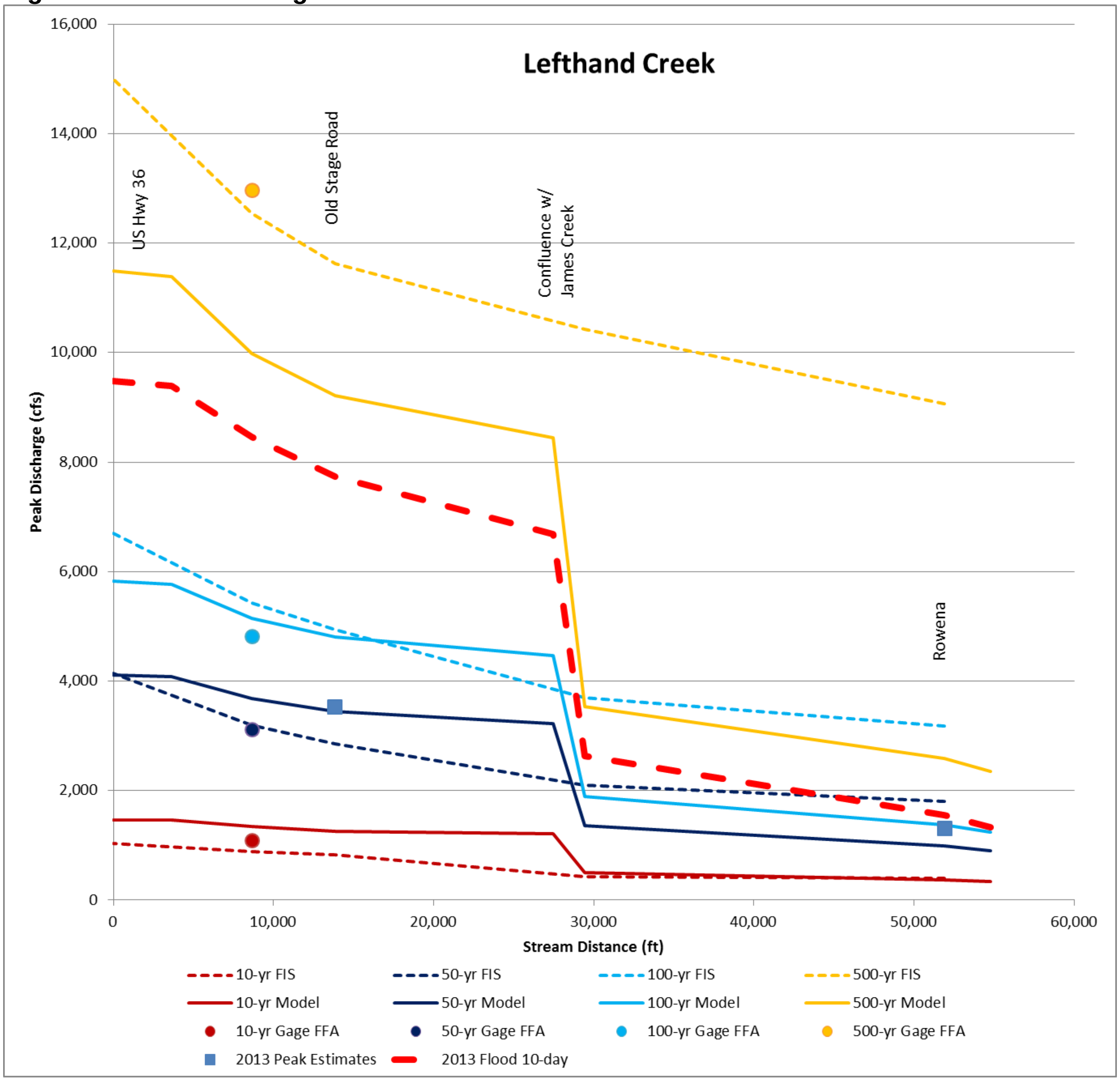
On James Creek the 10-year model results were consistently higher than the Effective FIS discharges. For the 50-year storm, from Jamestown down to the confluence with Lefthand Creek, the model results matched the Effective FIS discharges within 5% (2,520 cfs vs. 2,660 cfs). Upstream of Jamestown, the 50-year model results were higher than the Effective FIS. The 100-year model results on James Creek were approximately 10% to 30% lower than the current regulatory peak discharges with the exception of the reach upstream of the confluence with Little James Creek where the model results were approximately 10% higher. The model produced peak discharges approximately 30% to 50% lower than the Effective FIS for the 500-year storm; however, the effective peak discharges were based on extrapolated rainfall depths for the 500-year event. .

Figure 4. Peak Discharge Profile Plot for James Creek



On Lefthand Creek, the 10-day calibrated model was able to match the estimated peak discharge within 20% (1,550 cfs vs. 1,300 cfs) upstream of Rowena near Lickskillet Road. On Lefthand Creek near Old Stage Road (downstream of the confluence with James Creek), the estimated peak discharge of 3,520 cfs was well below the model result of 7,730 cfs. However, based on the peak discharge estimate on James Creek downstream of Jamestown (3,300 cfs), the estimate near Old Stage Road appears low. Several models were run in an attempt to account for possible channel attenuation and to evaluate the potential for offset peaks when the hydrographs for Lefthand Creek and James Creek meet at the confluence. None of these attempts were able to reduce to modeled peak discharges down to the level of the estimate near Old Stage Road. Due to the long duration of the storm and the multiple periods of intense rainfall there were two significant peaks that occurred during the storm. As a result, attempts to shift the timing of the hydrograph peaks at the confluence of Lefthand Creek and James Creek to reduce the combined peak discharge were unsuccessful. Therefore, the calibration focused on the six peak discharge estimates upstream of the confluence. With this systematic calibration approach in mind, and because the modeled peak discharges on James Creek downstream of Jamestown and on Lefthand Creek near Rowena closely matched the peak discharge estimates, further attempts to match the peak discharge estimate downstream of the confluence at Old Stage Road would only cause peak discharges at the other locations to be further away from the observed estimates.

Figure 5. Peak Discharge Profile Plot for Lefthand Creek



On Lefthand Creek, the results of the updated flood-frequency analysis were not considered reliable for calibration purposes because of the limited period of available gage data (17 years) and the wide confidence limits for larger storms. Figure 5 includes the flood-frequency results shown as circles. It can be seen that the FFA results were less than the modeled results for the 10-year, 50-year and 100-year storms, but greater than the modeled 500-year storm.

Comparisons against the effective peak discharge values published in the 1983 Floodplain Information Report reveal a distinct difference. The predictive model results show a clear jump in the profile at the confluence of Lefthand Creek and James Creek whereas the 1983 peak discharge profile does not appear to be impacted by James Creek. This difference is due to the methodology used in the original SWMM study which evaluated different rainfall durations at different locations in the watershed to determine which storm duration produced the largest peak discharge at a given location. The end result was that according to the 1983 study, the 1-hour storm produced the highest discharges on Lefthand Creek from near Ward to upstream of Spring Gulch. From Spring Gulch to upstream of James Creek, a 3-hour storm was used. Below the confluence with James Creek, the peak discharges were determined using a 6-hour storm. As a result of this critical storm analysis, the predictive model results on Lefthand Creek (upstream of the confluence of James Creek) are lower than the 1983 peak discharges by approximately 30% to 70%. The exception to this is the 10-year predictive model results which match almost exactly.

Downstream of the confluence with James Creek, the predictive model peak discharges were much closer to the peak discharges from the 1983 report. The 10-year results were consistently higher whereas the 50-year results were higher for most of the reach but matched almost exactly at Highway 36. The 100-year predictive model results are within 5% of the 1983 peak discharges for most the reach but were approximately 13% lower at Highway 36. The 500-year predictive model results were approximately 20% lower than the 1983 peak discharges, still within the range of hydrologic uncertainty.

The predictive model peak discharges were compared on a unit discharge basis (in cfs per square mile of watershed area) against flood frequency results, current regulatory discharges and predictive model peak discharges in the Big Thompson, North Fork Big Thompson, Buckhorn, and St. Vrain watersheds to get a sense for how the different sources of discharge estimates compare. This information is shown on a log-log scale in Figure 6. The following is a summary of the abbreviations used in the Figure 6 legend. Each watershed is indicated by a single color and each method is indicated by a specific marker shape:

Watershed (color):

BT = Big Thompson River (red)
NFBT = North Fork Big Thompson (green)
BH = Buckhorn Creek (light blue)
SV = St. Vrain Creek (dark blue)
LH = Lefthand Creek (orange)
JC = James Creek (gray)

Analysis Method/Data Source (marker shape):

FFA = Flood Frequency Analysis (triangle)
Reg = FIS Regulatory Peak Discharge (square)
HMS = HEC-HMS Calibrated Model (circle)

Figure 7 is a simplified version of Figure 6, showing only Lefthand Creek and James Creek.

Figure 6. Comparison of 100-year Discharges in Lefthand Creek and Adjacent Watersheds

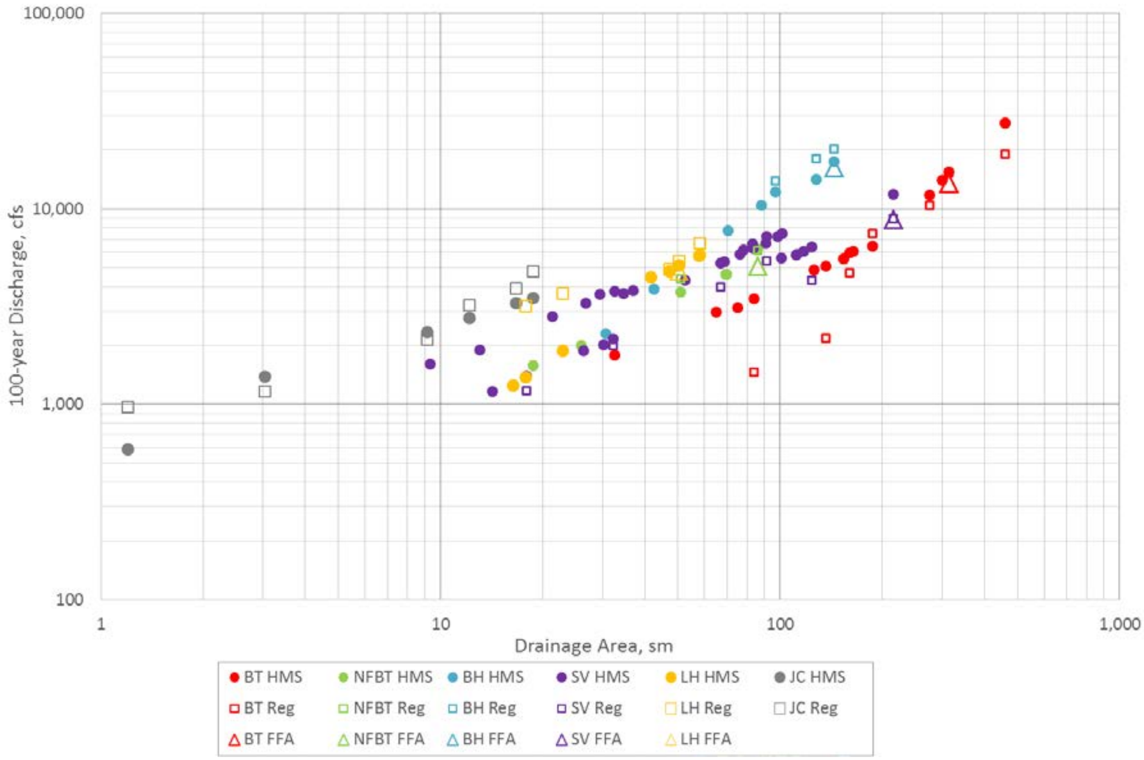
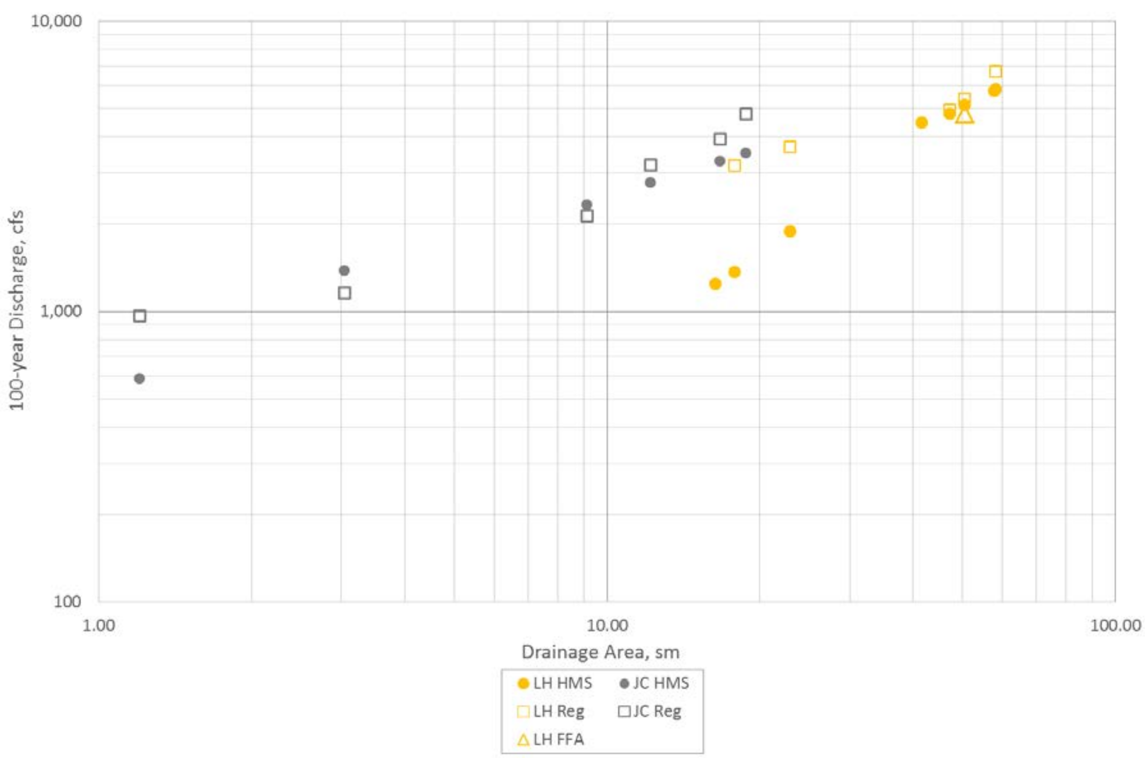


Figure 7. Comparison of 100-year Discharges in Lefthand Creek and James Creek Watersheds



Based on the information in Figures 6 and 7, several observations can be made:

1. The predictive model discharges for James Creek tend to be slightly higher than those in adjacent watersheds whereas the predictive model discharges for Lefthand Creek are in line with results from the adjacent watersheds (both above and below the James Creek confluence).
2. The predictive model peak discharges are slightly lower than the current regulatory peak discharges for James Creek, although typically within 20 percent.
3. The predictive model peak discharges closely match the current regulatory peak discharges and updated flood frequency analysis for the reach of Lefthand Creek downstream of the James Creek confluence.
4. The predictive model peak discharges are less than the current regulatory peak discharges for Lefthand Creek upstream of the James Creek confluence.

4.0 CONCLUSIONS AND RECOMMENDATIONS

This report documents a hydrologic investigation of Lefthand Creek associated with the extreme flood event of September, 2013. Peak discharges experienced during the flood were estimated and compared to current regulatory discharges as discussed in Appendix A. A summary of the peak discharge estimates is shown in Table 6 below. Comparisons of the 2012 Effective FIS and 1983 Floodplain Information Report discharges with the estimated flood estimates indicate that the September 2013 Flood ranged from a 50-year event to a 500-year event in some locations.

An updated flood frequency analysis was performed on the limited 17 years of gage record for Lefthand Creek. Table 6 below shows a summary of the updated flood frequency analysis for Lefthand Creek (analysis includes 2013 Flood event in the record). The flood frequency analysis results indicate slightly lower peak discharges than the current regulatory peak discharges and the predictive model results. Due to the limited period of data and the wide range of the 5% and 95% confidence limits, it was therefore not heavily relied on for calibration.

Table 6. Comparison of Peak Discharge Estimates

	2012 Effective FIS or 1983 FIR Peak Discharge				Ayres 2013 Updated				2013 Flood	2013 Flood
	Approximate Location for Comparison				Flood Frequency Analysis				Estimated	Estimated
Description	10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	Peak Discharge (cfs)	Recurrence Interval (yr)
Upper Little James Creek (Jarrett #10)	109	544	970	2,690					1,050	100
Lower Little James Creek (NRCS)	130	650	1,160	3,220					1,800	100 - 500
James Creek above confluence (Jarrett #11)	200	1,190	2,140	6,010					2,900	100 - 500
Confluence of James and Little James (NRCS)	300	1,785	3,205	8,850					4,800	100 - 500
James Creek below Jamestown (Jarrett #12)	355	2,180	3,930	10,880					3,300	50 - 100
Lefthand Creek at Lickskilllet Gulch (Jarrett #13)	400	1,800	3,180	9,060					1,300	10 – 50
Lefthand Creek at Old Stage Road (Jarrett #9)	830	2,850	4,940	11,630					3,520	50 - 100
LHC at confluence with Spring Gulch					1,091	3,110	4,810	12,965		

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 ten-day period representing the 2013 storm. The second 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 for the September 2013 event. This model was calibrated to the full 10-day period. The third 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 for the maximum 24-hour period of rainfall based on the ratio of runoff to rainfall from the full ten-day 2013 flood event. Once the curve numbers were adjusted to generate the same ratio of runoff to rainfall for the maximum 24-hour rainfall as the full ten day event, the design rainfall was applied. The results of this predictive model are summarized in Table 5 and in Appendix C.

Table 7 summarizes the predictive model results for the 100-year event compared to current regulatory discharges.

Table 7. 100-year Modeled Peak Flows Compared to Current Regulatory Discharges

Location	Current Regulatory Discharge (cfs)	Modeled Discharge (cfs)	Percent Difference
Little James Creek at upstream limit of detailed study	970	590	- 40%
Little James Creek upstream of confluence with James Creek	1,160	1,390	+ 19%
James Creek upstream of confluence with Little James Creek	2,140	2,340	+ 9%
James Creek at confluence with Little James Creek	3,205	2,780	- 13%
James Creek below Jamestown	3,930	3,300	- 16%
James Creek above confluence with Lefthand Creek	4,810	3,510	- 27%
Lefthand Creek at Lickskillet Gulch	3,180	1,370	- 57%
Lefthand Creek upstream of confluence with James Creek	3,690	1,890	- 49%
Lefthand Creek at Old Stage Road	4,940	4,800	- 3%
Lefthand Creek at Spruce Gulch	5,420	5,150	- 5%
Lefthand Creek at Highway 36	6,700	5,820	- 13%

On James Creek and the lower reach of Lefthand Creek (below the confluence with James Creek), the predictive model results are generally within 20 percent of the current regulatory discharges. However, on Lefthand Creek upstream of the confluence with James Creek, the results are 50 to 60 percent lower than the current regulatory discharges. The primary reason for the large difference in 100-year unit discharges on James Creek and Upper Lefthand Creek is that the model was calibrated to the 2013 Flood estimates which varied dramatically between these two locations. The unit discharges on Upper Lefthand Creek are reasonable when compared with modeled unit discharges in the St. Vrain and Big Thompson watersheds. In contrast, the unit discharges for James Creek are significantly higher than those modeled in adjacent watersheds. The high regulatory peak discharges in the headwater areas (Upper Lefthand Creek and James Creek) are due to the critical storm duration method used in the 1983 Study.

Based on the predictive model discharges for the return periods analyzed, the peak discharges observed in the Lefthand Creek watershed during the September 2013 Flood event had an estimated recurrence interval ranging from approximately a ~~1~~ 2 percent annual peak discharge to a 0.2 percent annual peak discharge, or from a ~~100~~ 50-year to a 500-year storm event as shown in Table 8.

Table 8. Estimate of September 2013 Peak Discharge Recurrence Interval based on Model Results

Location	Estimated Discharge (cfs)	Annual Chance Peak Discharge (cfs)					Estimated Recurrence Interval (yr)
		10%	4%	2%	1%	0.2%	
Upper Little James Creek	1,050	187	316	442	587	1,010	~ 500
Lower Little James Creek	1,800	423	730	1,036	1,386	2,425	100 to 500
James Creek above Little James Creek	2,900	776	1,278	1,772	2,339	4,003	100 to 500
James Creek in Jamestown	4,800	912	1,502	2,095	2,777	4,834	~ 500
James Creek below Jamestown	3,300	954	1,647	2,395	3,304	6,195	~ 100
Lefthand Creek at Lickskillet Gulch	1,300	~~200~~ 364	~~410~~ 666	~~650~~ 988	~~960~~ 1,371	~~2,000~~ 2,581	~~100 to 500~~ ~ 100
Lefthand Creek below Old Stage Road	3,520	~~840~~ 1,260	~~1,640~~ 2,336	~~2,490~~ 3,438	~~3,550~~ 4,804	~~6,990~~ 9,220	~~~ 100~~ 50 to 100

Based on these comparisons, 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 at Highway 36 and further downstream where these model results will be incorporated into a separate model for the entire St. Vrain watershed. The results are available for local floodplain administrators to consider using for regulatory discharges in the Lefthand Creek watershed; however, as mentioned, the modeling effort documented herein does not focus specifically on the determination of critical storms and associated discharges in smaller tributary areas. In this case, current regulatory discharges in upper Lefthand Creek are higher than the predictive peak flows documented herein.

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TECHNICAL APPENDICES

Appendix A
2013 Discharge Estimates in Comparison to Current Regulatory Flows

Flood and Paleoflood Methodologies for the September 2013 Flood Area

For Colorado Department of Transportation

Robert D. Jarrett

December 17, 2013

Flood Methods

Although flood measurements were made by the U.S. Geological Survey at many of their streamflow-gaging stations located in the September 2013 flood area in the Northern Colorado Front Range and downstream locations, there are other sites where flood data are needed such as for determining design floods for bridges affected by the flood. Post-flood, or indirect, methods are used to estimate peak discharges at ungaged sites or gaged sites that were inaccessible due to hazardous conditions, washed out bridges from which direct (current meter) measurements often are made, or other factors. Various indirect methods such as the slope-area, step-backwater, contracted-opening, and flow over a highway embankment are commonly used to compute flood discharge using standard hydraulic computational methods. The critical-depth method increasingly is being used on streams with channel gradients exceeding 0.005 to 0.01 ft/ft (~25 to 50 ft/mi) and has been validated to be within about 15 percent of discharge measured with current meter (Jarrett and England, 2002); this field documentation study of 212 stream sites in the western US for floods ranging from about the 2 year to 10,000 year recurrence interval (average of about the 75-year flood) confirms the theoretical reliability of the critical-depth method in higher gradient channels (Grant, 1997). Most streams within the 2013 flood area have gradients exceeding 0.005 ft/ft making them conducive for application of the critical-depth method. In addition, because of the extensive channel erosion and deposition, finding sufficient reach length for application of the other indirect methods is extremely problematic. Thus, the critical-depth method is very applicable in short, relatively straight but un-eroded sections of channel. Several channel cross sections are made in these relatively stable (in many cases, bedrock reaches can be located in mountain channels). An important benefit of using the critical-depth method is that discharge is not a function of channel roughness (e.g., Manning's n value); rather discharge is solely a function of channel geometry. Peak discharge calculations are made directly from channel cross-section data and associated high-water marks (HWMs). Peak discharge is estimated to be the average of the values at each cross section (typically 2-4 per site). An approximation of the peak-discharge uncertainty at a site can be made from the individual estimates and the average peak-discharge value.

The primary benefits of the critical-depth method are their cost effectiveness (about a tenth the cost of a standard indirect method referenced above) and how rapidly data can be provided from beginning of fieldwork to completion of the summary table (about two weeks for the requested sites, weather permitting). For September 2013 flood site visits, appropriate data reduction, computations, and quality assurance will be included. Photographs at all sites (on CD-ROM by site) and the site description (including latitude and longitude) will be provided in table form (Excel spreadsheet) for use by the Colorado Department of Transportation.

Paleoflood Method

In the past two decades, there has been growing interest by dam-safety officials and floodplain managers to incorporate risk-based analyses for design-flood hydrology. Extreme or rare floods, with recurrence intervals exceeding the 50-year flood to about the 10,000-year flood (annual exceedence probabilities, AEPs, in the range of about 0.02 to 10^{-4} chance of occurrence per year), are of increasing interest to the hydrologic and engineering communities for the purposes of planning, design, and maintenance of structures such as dams and levees. Flood-frequency analysis is a major component of flood-risk assessment. Reliable flood-frequency relations are difficult to estimate when using short gage record lengths typical of streamflow-gaging stations in the United States, 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. Reliable flood-frequency estimates are needed as input to risk assessments for determining appropriate levels of public safety, prioritizing projects, and allocating limited resources in a wide range of water-resources investigations such as dam safety, flood-plain management, and design of infrastructure such as bridges located in floodplains.

Because of the important role of paleoflood hydrology, it has increasingly been used in a range of water-resources investigations over the past 20 years. The American Society of Civil Engineers (ASCE) is also assessing the use of paleoflood hydrology as it relates to dam safety and risk-based assessments as well as better use of historical data and paleoflood data in many water-resources investigations. One ASCE focus area emphasized the need to develop standard protocols for using paleoflood techniques for applications by practicing hydrologists, engineers, and scientists in related fields. Paleoflood hydrology can provide useful information to assist the Colorado Department of Transportation and floodplain managers in their assessments of the probability of large floods. Documenting maximum paleofloods combined with regional analyses of contemporary extreme rainfall and floods help provide reliable flood-frequency estimates. Current regional flood-frequency methods available for eastern Colorado – defined as streams below about 8,000 feet and eastward) have uncertainties exceeding 100 percent. A CDOT-USGS eastern Colorado paleoflood study is underway to help reduce these uncertainties; I am providing field training in paleoflood methods for USGS and CDOT engineers. I collected substantial amounts of paleoflood data in the September 2013 flood area with the assistance of graduate students before I retired. They need only to be compiled from published papers, theses/dissertations, and field notebooks.

Paleoflood hydrology is the science of reconstructing the magnitude and estimating the frequency of large floods using geological evidence and a variety of interdisciplinary techniques. Although most paleoflood studies involve prehistoric floods, the methodology is applicable to historic or modern floods at gaged and ungaged sites (Jarrett and Tomlinson, 2000). Paleoflood studies to obtain data for contemporary floods (about 150 years ago to the present) also are used to complement short gage records and can be used to estimate flood-frequency relations at sites with limited gage data (Jarrett and Tomlinson, 2000).

Floods leave distinctive sedimentary deposits, along with botanical, erosional features on channel margins, and modifications of geomorphic surfaces by floodwaters in channels and on floodplains. These features, termed paleostage indicators (PSIs - PSIs can be thought of like old flood high-water marks, but with less reliability), can be used to infer the stage of past floods. In paleoflood studies, the most commonly used PSIs are slack-water deposits (SWDs) of silt and sand rapidly deposited from suspension in sediment-laden waters where velocities are minimal during the time that inundation occurs. SWDs are most commonly found in streams in the deserts of the south-western US. Another type of PSI used in paleoflood studies, particularly in mountain streams, is flood bars (FBs) of sand, gravel, cobble, and boulder deposits. A difference in studies that use SWDs and FBs is that SWDs can provide evidence for multiple (20-30) distinct floods that can be dated with ^{14}C, whereas coarse grained sediments in FBs (gravel, cobble, and boulders) can make it difficult to excavate a deposit to ascertain more than a few floods. The important factor for paleoflood studies is that the largest flood in a defined time scale is the primary flood documented. Another difference is most paleoflood studies are very detailed at a specific site, whereas the methods I developed are for documenting the largest paleoflood and discharge bounds on non-inundation surfaces (NISs) at many sites (50 to 200) along streams and their tributaries in a hydrologically homogeneous study region using relative dating methods for PSIs and NISs (e.g., Jarrett and Tomlinson, 2000).

When discharges are large enough, streambed and bank materials are mobilized and transported (Jarrett and England, 2002). These can be observed throughout the September 2013 flood area. When stream velocity, depth, and slope decrease, flowing water often is no longer competent to transport sediments, which are then deposited as slack-water deposits on the floodplain and flood bars in the channel. The types of sites where flood deposits commonly are found and studied include: (1) locations of rapid energy dissipation, where transported sediments would be deposited, such as tributary junctions, reaches of decreased channel gradient, abrupt channel expansions, or reaches of increased flow depth; (2) locations along the sides of valleys in wide, expanding reaches where fine-grained sediments or slack-water deposits would likely be deposited; (3) ponded areas upstream from channel contractions; (4) the inside of bends or overbank areas on the outside of bends, and; (5) locations at and downstream from terminal moraines across valley floors where floods would likely deposit sediments eroded from the moraines.

Flood-transported sediments and woody debris can scar trees, yielding an approximate flood height. Most commonly, trees along the main flow channel are scarred, whereas, trees protected by upstream trees and those in the margin of a floodplain may not have flood scars. Scars from older floods may have healed since the flood. Systematic coring on the upstream and streamward sides of trees can identify old scars. A lack of scarring at multiple sites in a reach is an indicator that substantial flooding has not occurred since establishment of trees on the floodplain. Use of multiple types of flood evidence at numerous sites for a stream and regional increases confidence for determining paleoflood magnitude and ages as well as ascertaining approximate levels of uncertainty.

The geomorphic evidence of floods in steep mountain basins (Jarrett and Tomlinson, 2000; the 2013 flood) is unequivocal. Paleoflood evidence in higher gradient streams is relatively easy to recognize and long lasting (tens of thousands of years) because of the quantity, morphology, structure, and size of sediments deposited by floods. In paleoflood investigations, lack of physical evidence of the occurrence of flooding is as important as discovering tangible on-site evidence of such floods (Jarrett and Costa, 1988; Jarrett and Tomlinson, 2000). Jarrett and Costa (1988) used PSIs and the lack of evidence of flooding (e.g., relatively undisturbed terminal moraines in stream valleys) to help understand the spatial variability of the maximum flooding throughout the Big Thompson River basin in Colorado. A paleohydrologic bound is a time interval since a particular discharge has not been exceeded. These bounds or non-inundation surfaces (NIS) have no fluvial erosional or depositional evidence and are determined to be stable surfaces with the age estimated such as by ^{14}C dating and relative-dating methods such soil-profile development.

Estimating paleoflood discharge using SWDs and PSIs is similar to estimating peak discharge using recent HWMs with step-backwater analysis, the slope-area, critical-depth, and slope-conveyance methods. Paleoflood discharge is reconstructed from estimates of flood width and depth corresponding to the elevation of the top of flood-deposited sediments (or new PSIs) and channel slope obtained during on-site visits to streams. Flood depth is estimated by using the PSIs in the channel or on the floodplain above the channel-bed elevation. Using the estimated flood depth and channel geometry, the mean depth, width, and cross-sectional area below the PSI elevation is determined. For streams that have higher gradient channels where slope exceeds 0.005 to 0.01 ft/ft, which are common in mountainous basins, flood and paleoflood discharge can be estimated using the critical-depth method, particularly for large floods (Jarrett and England, 2002). The slope-conveyance method can be used for relatively uniform channels (Jarrett and England, 2002) in the 2013 flooded area. Flow-resistance coefficients for these channels can be estimated from analysis of data for Colorado streams (Jarrett, 1985).

STATE OF COLORADO

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TO: Johnny Olson
CDOT Incident Command

FROM: Kevin Houck, P.E.
Chief, CWCB Watershed & Flood Protection Section

DATE: January 21, 2014
REVISED AND UPDATED JULY16, 2014

SUBJECT: **CDOT/CWCB Hydrology Investigation**
Phase One – 2013 Flood Peak Flow Determinations

As you are aware, northern Colorado experienced one of its worst flood disasters in state history in September 2013. This flood damaged or destroyed numerous state highways and bridges, primarily in the South Platte River basin. In addition, this flood destroyed numerous streamgauges and other measuring devices and created significant erosion and stream movement, which made measurement of flood flows extremely difficult.

The Colorado Department of Transportation (CDOT), in partnership with the Colorado Water Conservation Board (CWCB), has undertaken a significant effort to measure peak flows from the 2013 flood and to investigate an update of hydrologic models for watersheds that experienced significant damage. This memorandum summarizes the initial findings for peak flows during the flood. The effort is currently underway to reevaluate basin hydrology for the affected watersheds. Results from that effort will be summarized in a future memorandum.

Currently, best available information is being used for comparison to peak flood discharges. This comparison involves matching the peak flow rates from the 2013 flood to the regulatory discharges published in the Flood Insurance Study (FIS) for each county, as prepared by the Federal Emergency Management Agency (FEMA). When the new hydrologic models for each watershed are completed and approved, an updated comparison to peak flowrates from the 2013 flood will be made. This may result in a different peak flow frequency for some of the watersheds. While it is my belief that the updated information will yield a better overall estimate, this information is not yet available at this time. As such, the estimated flood frequencies presented in this memorandum is based on the best available information as of this date, but should be treated as provisional and subject to change.

The watersheds studied during this analysis include the South Platte River, Coal Creek, Boulder Creek, Lefthand Creek, the St. Vrain River, the Little Thompson River, and the Big Thompson River.

A summary of peak flood discharges from the 2013 flood, a comparison to regulatory flows, and an estimate of the observed flood frequency is presented in the table below. A discussion of the process will follow this table. In addition, Figures 1-4 present location maps of the various watersheds.

TABLE 1 – SUMMARY OF OBSERVED DISCHARGES AND FREQUENCY ESTIMATES

Location	Drainage Area (sq. mi.)	Regulatory Discharges (cfs)				2013 Peak Discharge Estimate (cfs)	2013 Estimated Frequency
		10-Year	50-Year	100-year	500-Year		
South Platte River							
South Platte River at Fort Lupton	5,043	10,000	22,000	29,000	52,000	10,100	10-Year
South Platte River at Kersey	9,659	11,000	24,500	32,500	57,500	55,000[1]	500 Year[1]
Coal Creek[2]							
Coal Creek at SH72 Near Wondervu	10.3	77	1,580	2,930	5,240	1,110	25-50 Year
Coal Creek Near Plainview Road	15.1	67	1,690	3,340	6,260	3,900	>100 Yr
Boulder Creek							
Boulder Creek near Orodell[3]	102	1,520	5,270	6,920	12,360	2,020	> 10 Year
Boulder Creek at 28th Street	136	2,200	7,800	8,000	20,600	5,300	25 Year
St. Vrain River Watershed							
Middle St. Vrain River above S. St. Vrain	32.4	590	1,430	2,000	4,070	1,750	50-100 Yr
South St. Vrain River at Middle St. Vrain	66.7	1,220	2,790	3,990	8,560	2,700	50-Year
South St. Vrain above confluence N. St. Vrain	92	1,400	3,750	5,430	11,900	9,000	<500 Year
North St. Vrain above confluence S. St. Vrain	125	1,000	2,850	4,310	10,630	12,300	>500 Year
St. Vrain below confluence N and S branches	211	2,040	6,670	8,890	20,260	23,300[4]	<500 Year
St. Vrain River at Interstate 25[5]	854	5,950	12,850	16,700	41,960	18,000[5]	>100 Year
Lefthand Creek upstream of US36[6]	47.2[6]	1,035	4,145	6,700	14,990	3,520[6]	50-Year
Little James Creek at Confl. James Creek	1.8	109	544	970	2,690	1,050	100 Year
James Creek above Little James Creek	8.9	200	1,190	2,140	6,010	2,900	>100 Year
James Creek at X/S A (d/s of Main Street)	14.5	355	2,180	3,930	10,880	3,300	50-100 Yr
Little Thompson River[7]							
Little Thompson River above West Fork	13.8	170	280	340	490	2,680	>500-Year
Little Thompson River below West Fork	43.2	775	2,166	2,585	N/A	12,300	>500-Year
Little Thompson River at Interstate 25[5]	170	5,535	12,723	14,728	19,923	14,500[5]	100 Year
Big Thompson River Watershed							
Big Thompson at Loveland Heights	156	2,250	3,800	4,700	7,200	9,300	>500-Year
Big Thompson at Drake Above North Fork	191	2,750	5,700	7,500	13,600	12,500	500 Year
Big Thompson below Drake	274	3,700	7,850	10,400	19,200	14,800[4]	>100 Year
Big Thompson at CR 29	314	3,800	10,500	15,300	37,000	15,500	100 Year
Big Thompson River at Interstate 25[5]	515	4,300	8,800	11,500	21,000	19,000	<500 Year
North Fork Big Thompson River at Drake	83	1,500	4,100	6,100	14,100	5,900[4,8]	100-Yr
Buckhorn Creek at Masonville above Redstone	92	4,674	10,321	13,862	24,000	7,700[4]	25-Year
Buckhorn Cr. at Confluence w/ Big Thompson	142.9	6,844	15,090	20,244	36,000	11,200	25-Year

[1]Discharge estimates from direct measurements below Fort Lupton not yet available. Hydrology team used values from other flood sites and professional judgment to estimate flow at Kersey, but this is not a direct measurement.
[2]Coal Creek regulatory values have been submitted to and approved by FEMA, but not yet published in Flood Insurance Study.
[3]Per Upper Boulder Creek & Fourmile Creek Floodplain Information Report (Gingery and Associates, 1981)
[4]Revision to a previous estimate.
[5]Information at Interstate 25 provided by Steve Griffin, Region 4 Hydraulics. See Peak Flow Hydrology Investigation for the September 2013 Flood at Interstate 25, dated January 7, 2014.
[6]Regulatory discharge values for Lefthand Creek are not available upstream of Longmont in the 2012 FIS. Values reported in the table above represent the discharges at Highway 36 provided by Boulder County from the 1983 Simons Li report for Upper Lefthand Creek, which are presented for comparison, but they do not directly correspond to the location of the observed flood peak, which is further upstream of US36. Professional judgment was used to estimate the observed frequency based on available information.
[7]No regulatory discharge values are available for the Little Thompson River. "Regulatory discharges" presented in the table above are from a hydrologic model developed by CDOT (courtesy Steve Griffin, Region 4 Hydraulics) or from regression equations (Capesius and Stephen, 2009). This represents the best available information, but it is not regulatory.
[8]Measurement at Drake listed. NRCS established an estimate of 18,400 cfs at a location 4.5 miles upstream of Drake. The larger value is judged to be a result of a natural dambreak whose flows were quickly attenuated downstream.

STUDY DESCRIPTION AND BACKGROUND

Following the September 2013 flood event, it became immediately apparent by State leaders in various departments that updated floodplain information would be needed for the purposes of infrastructure repair and land use decisions. Put simply, current regulatory information no longer applied in many areas, although it still represented the only information available following the flood. As such, CDOT and CWCB began a massive effort to update the hydrology and hydraulics of many of the watersheds affecting CDOT infrastructure damaged by the 2013 floods. This effort is being phased to develop information for various steps of the analysis.

The first phase, described here, involves an initial analysis of the 2013 flood to determine which frequencies may have occurred for six key watersheds. This enables CDOT and other land use agencies to determine how infrastructure performed during a flood of a particular magnitude. This memorandum summarizes the preliminary information obtained during this phase.

The second phase will involve update and redevelopment of the hydrologic models for the same six watersheds. In some cases, this will be the first major update to the regulatory watershed in over thirty years (see below).

Ultimately, the CWCB resolves to utilize updated topographic information to develop new hydraulic information. CDOT would be able to use this information for infrastructure design decisions, and CWCB plans to use this information to update regulatory floodplains.

HYDROLOGY

Hydrology involves the computation of design flow rates expected to occur at various locations for various design frequencies (i.e. 10-year or 100-year). It is a complex modeling effort involving rainfall, infiltration, soil types, land uses, and other watershed characteristics such as slope and imperviousness. Detention and reservoir storage can be incorporated into the modeling, but it is a state and federal requirement that attenuation from storage components can only be considered in areas where dedicated flood storage is set aside that cannot be used for other purposes, such as water supply. For cases such as Barker Reservoir on Boulder Creek, flood flows may be incidentally detained in less-than-full reservoirs (as happened during this event since it was well past the spring season, when reservoirs are typically filled), but if this storage cannot be relied upon during a flood event, it is typically ignored.

In practical uses, it is not desirable to update hydrology on a frequent basis. Floodplain studies and maps, and the modeling behind them, are expensive to update, and there are important and sometimes controversial land use impacts associated with changing floodplain maps often. Any time hydrology is updated for a watershed, all floodplain maps must generally be changed to reflect this new hydrology. Practically speaking, large scale hydrology has not been updated for many of the watersheds in over twenty or thirty years.

However, the circumstances that exist now render the creation of new hydrology to be a uniquely appropriate effort at this time. There are many reasons for this:

- Because of stream erosion and movement, the hydraulic characteristics of many large rivers are vastly different than what they were just ten months ago. For this reason, it is assumed that floodplains associated with many reaches of these large watersheds will need to be updated in any case to reflect new hydraulic conditions.
- The National Oceanic and Atmospheric Administration (NOAA) updated design rainfall information for the first time in forty years in 2013. Prior to the new information becoming available, design rainfall was still based on documents released in 1973. The new information incorporates an additional forty years of data and underwent heavy peer review prior to being published, and it is widely regarded as far superior to previous information.
- This flood represents a unique opportunity for hydrologic reevaluation because it occurred in an area with a large volume of data available (including detailed gridded rainfall, sufficient soils and land use information,

reservoir releases during the flood, newly obtained LIDAR topography, and ample direct and indirect flow measurements). This provides a one-time opportunity for a recorded event to calibrate the models to.

- Perhaps most important, there is increased political and public support for updating information used for recovery activities for the express purpose of mitigating future flood threats.

For these reasons, the hydrology team agreed that this is an appropriate time to restudy basin hydrology at the watershed level. This process has already begun, but it is a rigorous and detailed process, and no preliminary results are available at this time.

STUDY PROCESS AND KEY ASSUMPTIONS/CAVEATS

As mentioned above, many measurement devices failed during the September 2013 flood, rendering the need for indirect analysis to determine flow rates that occurred during the flood. For this study, field measurements were taken at key locations in an effort to estimate these flow rates forensically (indirect post-flood determinations). Locations were chosen based on need, accessibility, and site conditions, with surveys taking place in November and December 2013. Fieldwork involved determination of high water marks and development of new rating curves based on updated topography, which in many cases was vastly different than what existed prior to the flood.

It is important to note that there is a degree of subjectivity and professional judgment necessary for these indirect peak flow calculations. In many cases, it is a challenge to determine what the stream looked like at the moment of peak flow, especially as streams continued to migrate or erode following the peak of the flow. As such, a certain amount of statistical uncertainty is inherent in developing measurements of this type. The team estimates that the uncertainty in some cases can be as high as +/- 20%. While this envelope of uncertainty will not, in most cases, affect the stated frequency, this range should nonetheless, we factored into consideration when viewing measured discharges in Table 1. Finally, the results presented herein will undergo subsequent review and may be revised. However, I am quite confident that the computed flow rates using indirect methods presented in this memorandum are as good as can be obtained anywhere.

It is known by the hydrology team that others have undertaken similar efforts, but to the team's knowledge, no results have yet been released. One such effort has been undertaken by the United States Geological Survey (USGS). The USGS took field measurements within the first two months following the flood. However, as of the date of this memorandum, nothing has been made publicly available. While I am confident that the measurements presented in this memorandum will stand up under comparison, it should be emphasized that due to the inherent uncertainties referenced above, it is likely that small deviations would be present when comparing these results to eventual results from others.

These computed flow rates were then compared to currently published regulatory flow values for the purpose of assigning flood frequencies. In most cases, this regulatory information can be obtained from FEMA's Flood Insurance Studies. This is the source of this regulatory information in all cases from Table 1 unless otherwise noted. Regulatory information from the FIS generally includes 10-year, 50-year, 100-year, and 500-year values.

It is also important to note that the locations for field measurements were not always exactly in the same locations as design hydrological points from the FIS. However, unless specifically noted otherwise, the observed flows can generally be compared to the regulatory flows as they are proximate in location and generally do not represent a hydrologic departure (for example, without intervening tributaries).

Perhaps most importantly, it is critical to understand that these computed flow rates are being compared to established regulatory floodplain information that was developed prior to the flood. This simply represents the best available information that can currently be used. As noted above, there are plans to conduct an updated comparison based on results from the hydrologic analysis developed during the second phase of this study. It is extremely likely that somewhat different results will be obtained during this reanalysis. As such, <u>comparisons and flood frequencies presented in this memorandum should be treated as provisional based on the best information available at this time and subject to revision</u>.

Appendix B
Flood Frequency Analysis at Stream Flow Gage

Bulletin 17B Frequency Analysis

Lefthand Creek near Boulder

USGS Gage 06724500 (1929 – 1980 broken)

CDWR Gage LEFCRECO (2012 – 2012, no peak included)

Bulletin 17B Frequency Analysis
22 Aug 2014 01:43 AM

--- Input Data ---

Analysis Name: 06724500 LT HD CK BLDR 2013STA
Description: Copy of Copy of 06724500 LEFT HAND CK BLDR REG

Data Set Name: LEFT HAND CREEK-BOULDER, 2013
DSS File Name: H:\32-176904 Big Thompson Hydrology\Six_Rivers_HEC-SSP_FFA_Results\Six_Rivers\Six_Rivers.dss
DSS Pathname: /LEFT HAND CREEK/BOULDER, CO./FLOW-ANNUAL PEAK/01jan1900/IR-CENTURY/Save Data As: LEFT HAND CREEK-BOULDER, 2013/

Report File Name: H:\32-176904 Big Thompson Hydrology\Six_Rivers_HEC-SSP_FFA_Results\Six_Rivers\Bulletin17bResults\06724500_LT_HD_CK_BLDR_2013STA\06724500_LT_HD_CK_BLDR_2013STA.rpt
XML File Name: H:\32-176904 Big Thompson Hydrology\Six_Rivers_HEC-SSP_FFA_Results\Six_Rivers\Bulletin17bResults\06724500_LT_HD_CK_BLDR_2013STA\06724500_LT_HD_CK_BLDR_2013STA.xml

Start Date:
End Date:

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

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---

--- Preliminary Results ---

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

Events Analyzed		Ordered Events			
Day Mon Year	FLOW CFS	Rank	Water Year	FLOW CFS	Median Plot Pos
17 Jul 1929	216.0	1	2013	3,140.0*	4.02
26 Aug 1930	340.0	2	1949	1,140.0	9.77
02 Aug 1931	396.0	3	1951	785.0	15.52
22 Jun 1947	254.0	4	1957	707.0	21.26
06 Jun 1948	510.0	5	1980	571.0	27.01
04 Jun 1949	1,140.0	6	1948	510.0	32.76
15 Jun 1950	234.0	7	1931	396.0	38.51
03 Aug 1951	785.0	8	1930	340.0	44.25
15 Jun 1952	252.0	9	1978	331.0	50.00
12 Jun 1953	245.0	10	1947	254.0	55.75
22 May 1956	230.0	11	1952	252.0	61.49
09 May 1957	707.0	12	1953	245.0	67.24
06 Jun 1977	230.0	13	1950	234.0	72.99
15 Jun 1978	331.0	14	1977	230.0	78.74
09 Jun 1979	187.0	15	1956	230.0	84.48
30 Apr 1980	571.0	16	1929	216.0	90.23
13 Sep 2013	3,140.0	17	1979	187.0	95.98

* Outlier

<< Skew Weighting >>

Based on 17 events, mean-square error of station skew = 0.63
Mean-square error of regional skew = -?

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

Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
12,965.4	43,389.1	0.2	56,119.8	5,685.1
7,394.3	16,689.7	0.5	25,625.4	3,650.6
4,809.9	8,619.8	1.0	14,090.2	2,594.7
3,110.4	4,612.6	2.0	7,708.7	1,829.9
1,726.6	2,132.2	5.0	3,444.1	1,132.0
1,091.1	1,228.6	10.0	1,862.4	768.0
676.4	712.7	20.0	1,007.9	499.2
338.1	338.1	50.0	456.1	242.5
218.2	214.8	80.0	297.6	142.8
188.8	184.7	90.0	260.8	118.7
173.4	168.8	95.0	241.8	106.3
157.6	153.9	99.0	222.4	93.9

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

Log Transform: FLOW, CFS		Number of Events	
Mean	2.606	Historic Events	0
Standard Dev	0.324	High Outliers	0
Station Skew	1.490	Low Outliers	0
Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	1.490	Systematic Events	17

--- End of Preliminary Results ---

<< High Outlier Test >>

Based on 17 events, 10 percent outlier test deviate K(N) = 2.309
Computed high outlier test value = 2,256.95

1 high outlier(s) identified above test value of 2,256.95

* *
* 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)

<< Low Outlier Test >>

Based on 17 events, 10 percent outlier test deviate K(N) = 2.309
Computed low outlier test value = 72.26

0 low outlier(s) identified below test value of 72.26

--- Final Results ---

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

Events Analyzed				Ordered Events			
Day	Mon	Year	FLOW CFS	Rank	Water Year	FLOW CFS	Median Plot Pos
17	Jul	1929	216.0	1	2013	3,140.0*	0.82
26	Aug	1930	340.0	2	1949	1,140.0	4.48
02	Aug	1931	396.0	3	1951	785.0	10.63
22	Jun	1947	254.0	4	1957	707.0	16.77
06	Jun	1948	510.0	5	1980	571.0	22.92
04	Jun	1949	1,140.0	6	1948	510.0	29.07
15	Jun	1950	234.0	7	1931	396.0	35.22
03	Aug	1951	785.0	8	1930	340.0	41.36
15	Jun	1952	252.0	9	1978	331.0	47.51
12	Jun	1953	245.0	10	1947	254.0	53.66
22	May	1956	230.0	11	1952	252.0	59.81
09	May	1957	707.0	12	1953	245.0	65.95
06	Jun	1977	230.0	13	1950	234.0	72.10
15	Jun	1978	331.0	14	1977	230.0	78.25
09	Jun	1979	187.0	15	1956	230.0	84.40
30	Apr	1980	571.0	16	1929	216.0	90.54
13	Sep	2013	3,140.0	17	1979	187.0	96.69

Note: Plotting positions based on historic period (H) = 85
Number of historic events plus high outliers (Z) = 1
Weighting factor for systematic events (W) = 5.25

* Outlier

<< Skew Weighting >>

Based on 17 events, mean-square error of station skew = 0.63
Mean-square error of regional skew = -?

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

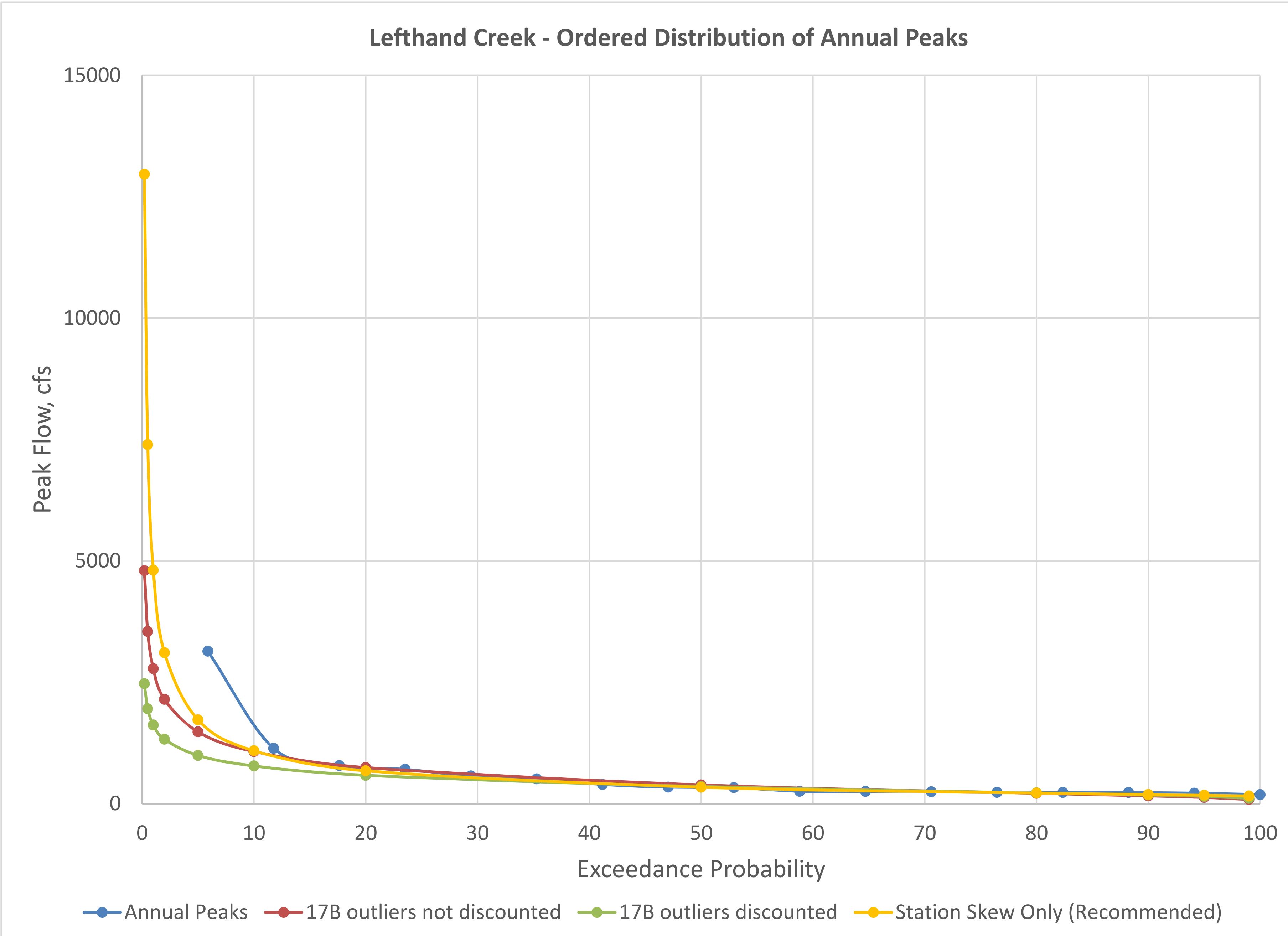
Computed Curve FLOW, CFS	Expected Probability FLOW, CFS	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, CFS	Confidence Limits 0.95 FLOW, CFS
12,965.4	43,389.1	0.2	56,119.8	5,685.1
7,394.3	16,689.7	0.5	25,625.4	3,650.6
4,809.9	8,619.8	1.0	14,090.2	2,594.7
3,110.4	4,612.6	2.0	7,708.7	1,829.9
1,726.6	2,132.2	5.0	3,444.1	1,132.0
1,091.1	1,228.6	10.0	1,862.4	768.0
676.4	712.7	20.0	1,007.9	499.2
338.1	338.1	50.0	456.1	242.5
218.2	214.8	80.0	297.6	142.8
188.8	184.7	90.0	260.8	118.7
173.4	168.8	95.0	241.8	106.3
157.6	153.9	99.0	222.4	93.9

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

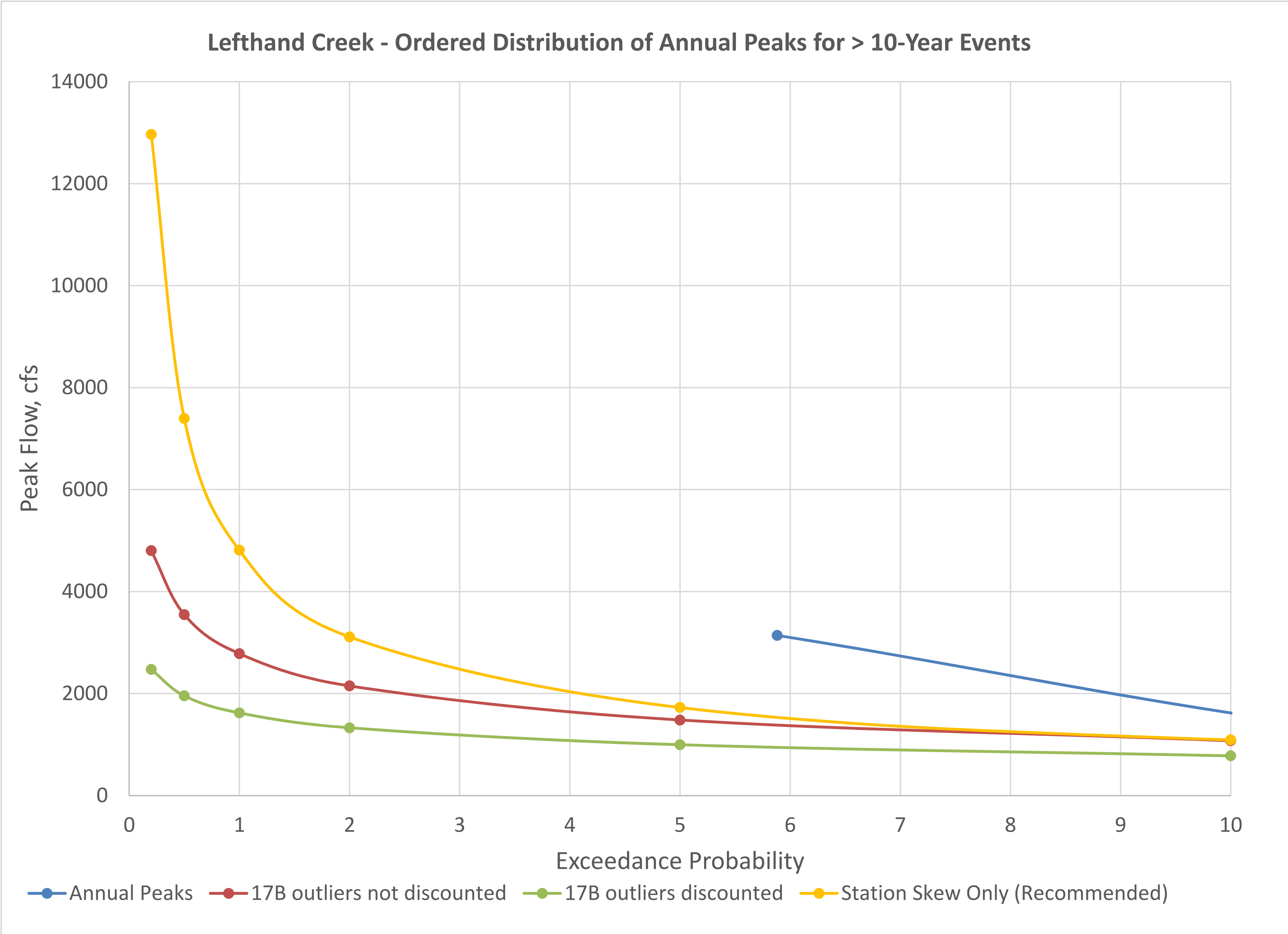
Log Transform: FLOW, CFS		Number of Events	
Mean	2.606	Historic Events	0
Standard Dev	0.324	High Outliers	1
Station Skew	1.490	Low Outliers	0

Regional Skew	---	Zero Events	0
Weighted Skew	---	Missing Events	0
Adopted Skew	1.490	Systematic Events	17
		Historic Period	85

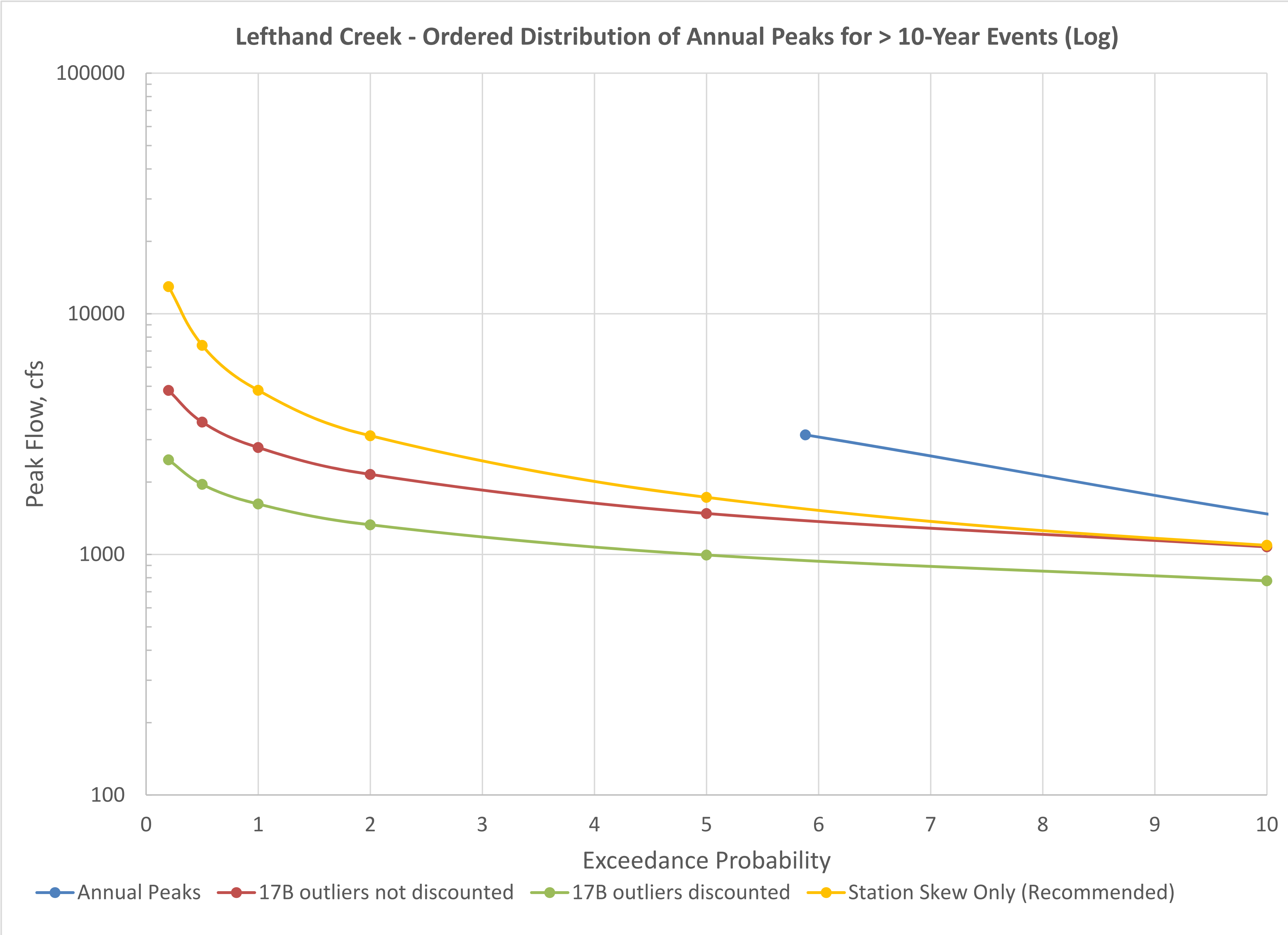
--- End of Analytical Frequency Curve ---



Lefthand Creek - Ordered Distribution of Annual Peaks
15000
10000
5000
0
Peak Flow, cfs
0
10
20
30
40
50
60
70
80
90
100
Exceedance Probability
Annual Peaks
17B outliers not discounted
17B outliers discounted
Station Skew Only (Recommended)

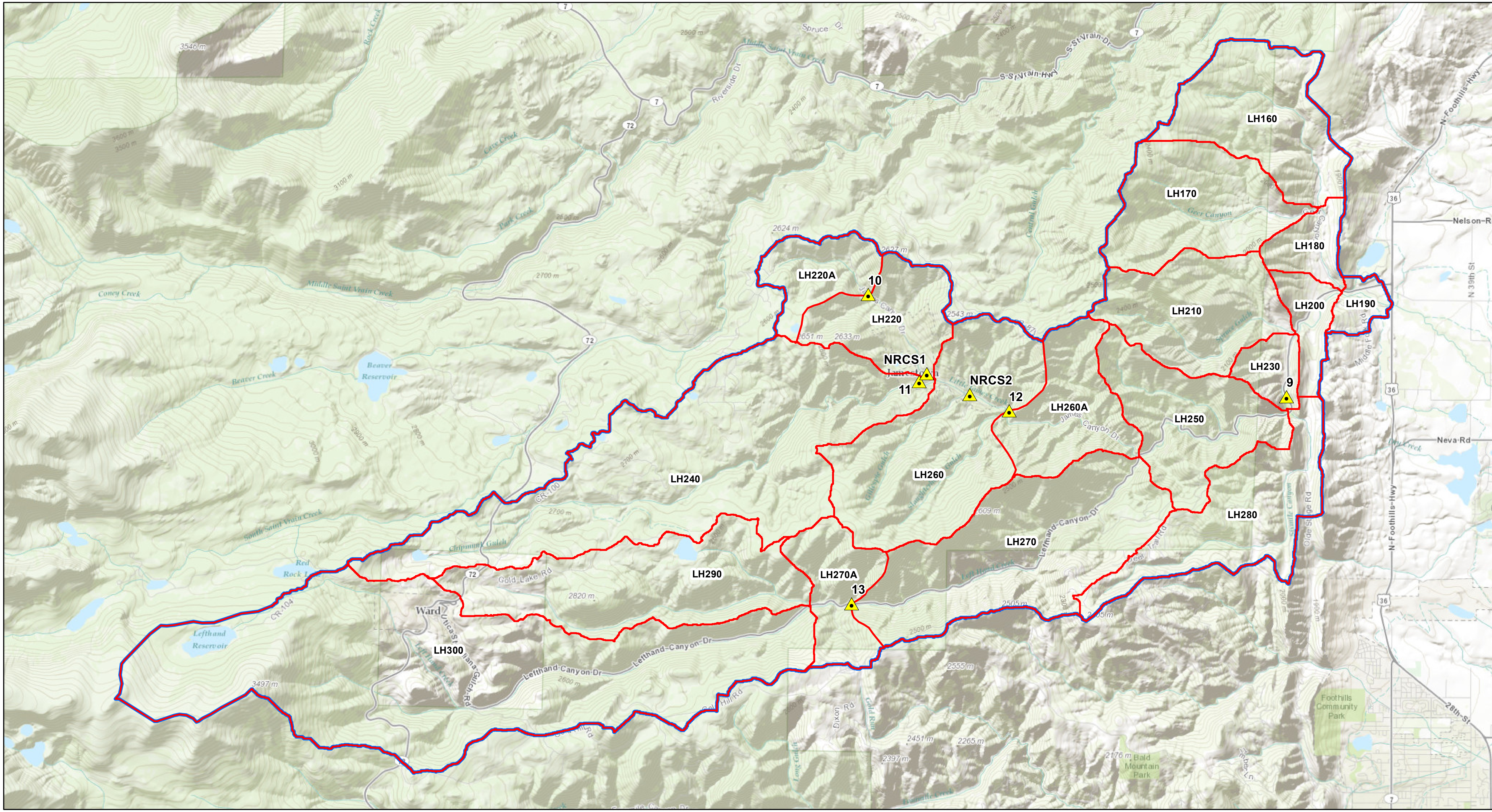


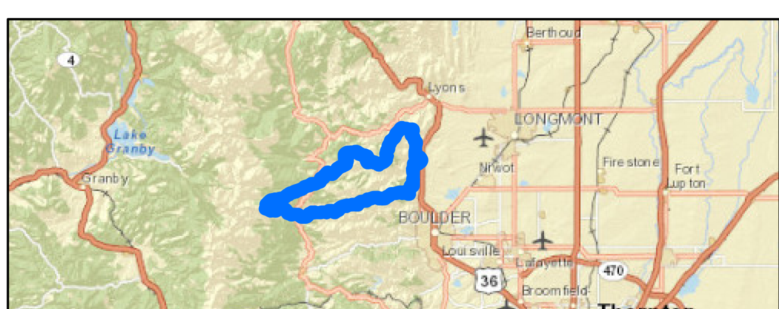
Lefthand Creek - Ordered Distribution of Annual Peaks for > 10-Year Events
14000
12000
10000
8000
6000
4000
2000
0
Peak Flow, cfs
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
17B outliers not discounted
17B outliers discounted
Station Skew Only (Recommended)



Lefthand Creek - Ordered Distribution of Annual Peaks for > 10-Year Events (Log)
100000
10000
1000
100
Peak Flow, cfs
0
1
2
3
4
5
6
7
8
9
10
Exceedance Probability
Annual Peaks
17B outliers not discounted
17B outliers discounted
Station Skew Only (Recommended)

Appendix C
Rainfall / Runoff Modeling





Left Hand Creek Watershed

CDOT Flood Recovery Hydrologic Evaluation

0 0.5 1 1.5
Miles

Investigation Sites
Left Hand Creek SubBasins
Left Hand Creek Watershed Boundary

Site	Stream and Location	Peak Discharge	Tributary Area
9	Lefthand Creek 1/4 mile d/s Stagecoach Road nr Boulder	3520	47.3
10	Little James Creek u/s Jamestown	1050	1.1
11	James Creek u/s Jamestown	2900	9.0
12	James Creek d/s Jamestown	3300	15.0
13	Lefthand Creek u/s Rowena	1300	17.5
NRCS1			
NRCS2			

JACOBS

Appendix C.1

Lefthand Creek Watershed

Design Point	Description	Drainage Area (sq. mi.)	2013 Flood Estimated Peak Discharge (cfs)	2013 Flood 10-day Period Calibrated (cfs)	2013 Flood Max 24hr Period Calibrated (cfs)	2013 Flood Max 24hr Period Runoff/Rainfall Ratio Adjusted	NOAA 24-hr Type II Predictive Storms (Depth-Area Adjusted) 10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	Effective FIS Peak Discharge Approximate Location for Comparison 10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)
LH220A	Upper Little James Creek	1.20		508	507	282	187	316	442	587	1010	109	544	970	2690
J64A	**Upper Little James Creek (Jarrett #10)**	**1.20**	**1050**	**508**	**507**	**282**	**187**	**316**	**442**	**587**	**1010**				
R70	Little James Creek	1.20		508	506	282	187	315	441	586	1009				
LH220	Lower Little James Creek	1.84		766	763	420	257	440	623	834	1454				
J64C	**Lower Little James Creek (NRCS #1)**	**3.04**	**1800**	**1256**	**1249**	**682**	**423**	**730**	**1036**	**1386**	**2425**	**130**	**650**	**1160**	**3220**
LH240	Upper James Creek	9.13		2197	2083	1150	776	1278	1772	2339	4003				
J64D	**James Creek above confluence (Jarrett #11)**	**9.13**	**2900**	**2197**	**2083**	**1150**	**776**	**1278**	**1772**	**2339**	**4003**	**200**	**1190**	**2140**	**6010**
J64	**Confluence of James and Little James (NRCS #2)**	**12.17**	**4800**	**3250**	**2914**	**1743**	**912**	**1502**	**2095**	**2777**	**4834**	**300**	**1785**	**3205**	**8850**
R170	James Creek through Jamestown	12.17		3248	2914	1742	912	1502	2095	2777	4833				
LH260	James Creek below Jamestown	4.52		734	1153	784	48	174	346	588	1459				
J64B	**James Creek below Jamestown (Jarrett #12)**	**16.69**	**3300**	**3824**	**3871**	**2513**	**954**	**1647**	**2395**	**3304**	**6195**	**355**	**2180**	**3930**	**10880**
R90	James Creek channel above LHC	16.69		3821	3869	2511	953	1647	2395	3303	6191				
LH260A	James Creek area above LHC	2.03		343	513	377	25	89	177	299	736				
J57b	James Creek above confluence with LHC	18.72		4095	4139	2824	973	1710	2517	3512	6728	390	2660	4810	13500
LH300	Upper Lefthand Creek	12.20		928	933	449	247	442	648	893	1665				
LH290	Spring Gulch	4.15		445	482	276	96	186	283	403	785				
J49	Confluence of LHC and Spring Gulch	16.35		1325	1390	684	333	606	898	1246	2349				
R160	LHC above Lickskillet Road	16.35		1325	1390	684	333	606	898	1246	2348				
LH270A	Lefthand Creek at Lickskillet Road	1.44		532	587	299	68	137	212	305	601				
J49A	**Lefthand Creek at Lickskillet Road**	**17.79**	**1300**	**1548**	**1603**	**856**	**364**	**666**	**988**	**1371**	**2581**	**400**	**1800**	**3180**	**9060**
R130	LHC above confluence with James Creek	17.79		1548	1603	856	364	666	988	1371	2580				
LH270	Lefthand Creek upstream of James Creek	5.13		1119	1161	794	168	311	458	637	1184				
J57a	Lefthand Creek upstream of James Creek	22.92		2626	2575	1619	502	920	1359	1886	3536	430	2100	3690	10430
J57	Confluence of LHC and JC	41.64		6687	6224	4368	1212	2214	3227	4472	8439				
R100	LHC below confluence with JC	41.64		6684	6222	4365	1211	2213	3226	4471	8437				
LH250	Lefthand Creek above Old Stage Road	2.68		572	754	387	32	80	139	217	483				
LH280	Six Mile Canyon	2.98		562	709	430	31	75	127	196	432				
J54	**Lefthand Creek below Old Stage Road**	**47.29**	**3520**	**7734**	**7389**	**5148**	**1260**	**2336**	**3438**	**4804**	**9220**	**830**	**2850**	**4940**	**11630**
R80	LHC below Old Stage Road	47.29		7732	7385	5145	1259	2335	3437	4803	9218				
LH230	LH230	0.59		203	226	142	34	70	108	154	299				
LH210	LH210	2.66		742	842	464	104	196	293	411	773				
J67	LHC confluence with Spruce Gulch	50.54		8461	8083	5539	1340	2493	3678	5151	9975	890	3190	5420	12540
R60	LHC above confluence with Geer Canyon	50.54		8458	8078	5536	1340	2492	3677	5150	9973				
LH160	LH160	2.91		511	733	286	44	104	175	268	582				
LH170	LH170	2.85		591	812	392	67	144	231	341	697				
J81	Confluence of Plumley and Geer Canyons	5.75		1102	1546	665	110	247	405	608	1277				
R30	Geer Canyon Creek	5.75		1101	1546	665	110	247	405	608	1276				
LH180	LH180	0.72		258	295	187	57	104	151	208	377				
LH200	LH200	0.71		212	237	145	31	62	95	136	262				
J74	Confluence of LHC and Geer Canyon	57.72		9394	9269	6332	1457	2743	4085	5771	11388				
R40	LHC above Hwy 36	57.72		9390	9267	6330	1456	2743	4084	5771	11385				
LH190	LH190	0.41		176	191	142	65	107	146	191	318				
Outlet1	**LHC at Hwy 36**	**58.12**		**9474**	**9362**	**6358**	**1469**	**2765**	**4117**	**5822**	**11493**	**1035**	**4145**	**6700**	**14990**

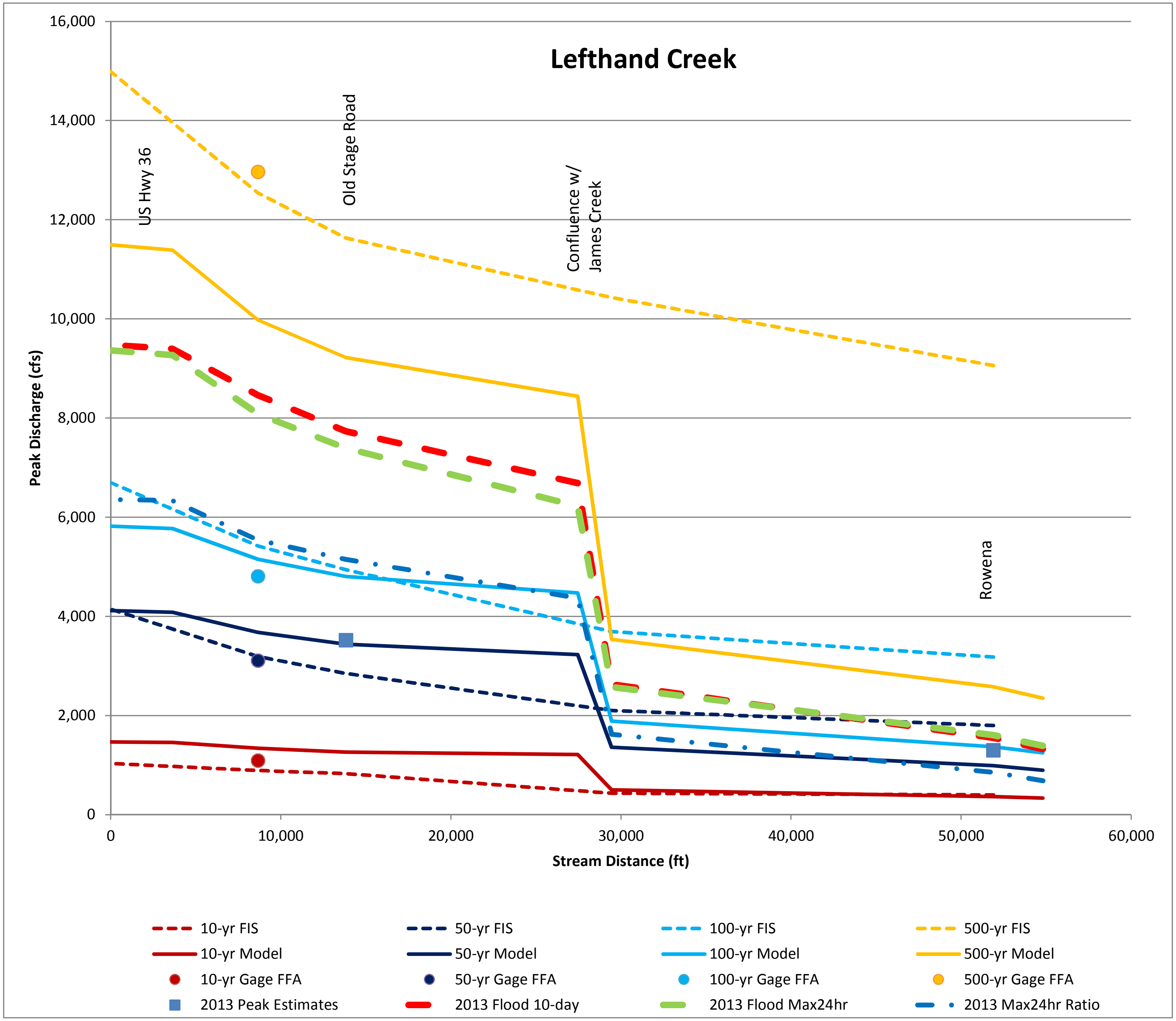
Appendix C.1 (continued)

Condensed for Profiles

LEFTHAND WATERSHED

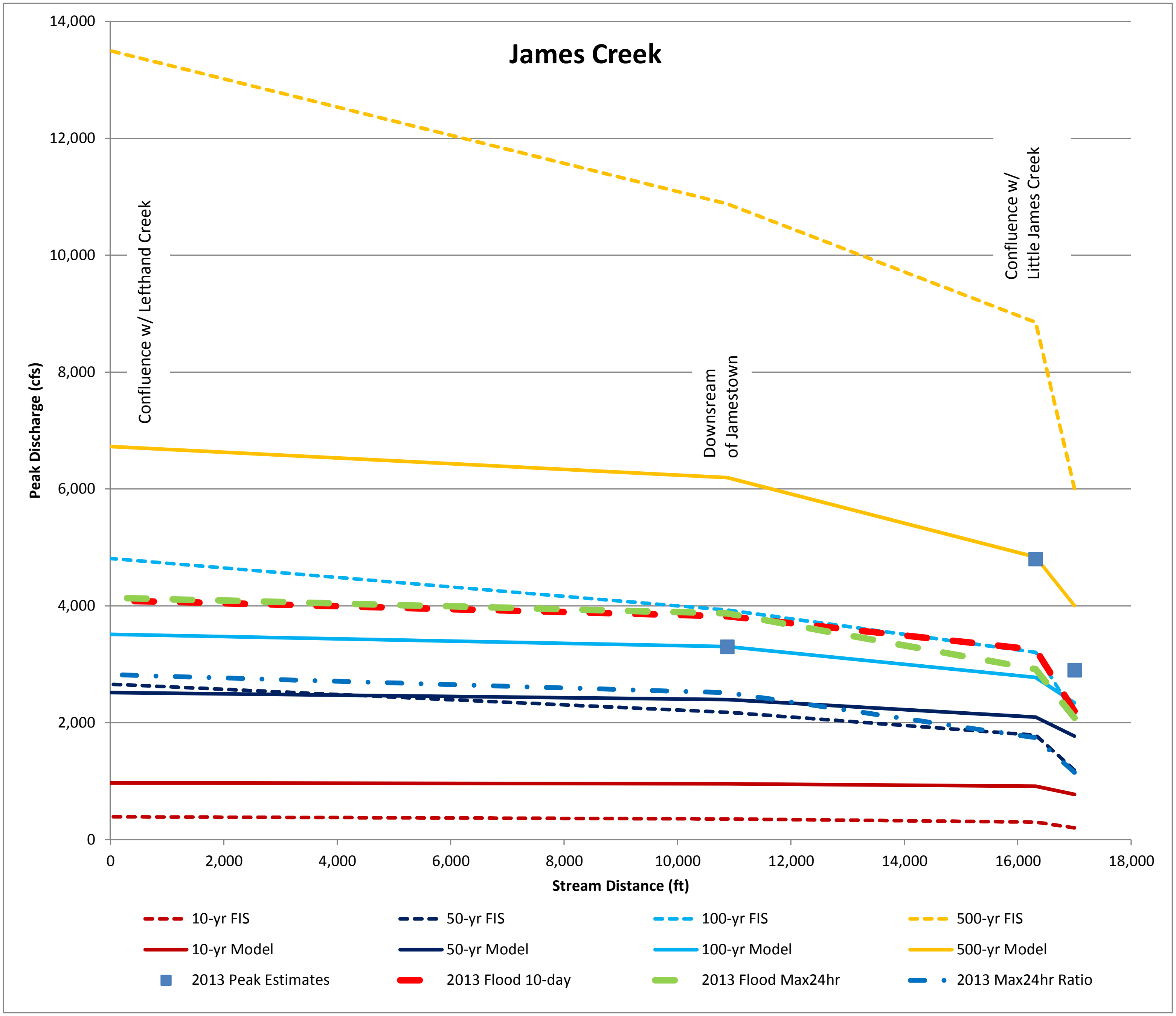
1	2	3	4	5	6		7	8	9	10	11	12	13	14	15	16	17	18	19	20
HEC-HMS		**Approx. Station**	**Drianage Area**	**2013 Flood Estimated Peak Discharge**	**2013 Flood 10-day Calibrated**	**2013 Flood Max 24hr Period Calibrated**	**2013 Flood Max 24hr Period Rainfall/Runoff Ratio Adjusted**	**NOAA Design Storms (Depth-Area Adjusted)**					**Effective FIS Peak Discharge**				**Ayres 2013 Flood Frequency Analysis**			
								10-yr	**25-yr**	**50-yr**	**100-yr**	**500-yr**	**10-yr**	**50-yr**	**100-yr**	**500-yr**	**10-yr**	**50-yr**	**100-yr**	**500-yr**
Design Point	**Description**	**(ft)**	**(sq. mi.)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**	**(cfs)**
J64A	**Upper Little James Creek (Jarrett #10)**	7,128	1.20	**1,050**	**508**	**507**	**282**	**187**	**316**	**442**	**587**	**1,010**	**109**	**544**	**970**	**2,690**				
J64C	**Lower Little James Creek (NRCS 1)**	0	3.04	**1,800**	**1,256**	**1,249**	**682**	**423**	**730**	**1,036**	**1,386**	**2,425**	**130**	**650**	**1,160**	**3,220**				
J64D	**James Creek above confluence with Little James Creek (Jarrett #11)**	17,000	9.13	**2,900**	**2,197**	**2,083**	**1,150**	**776**	**1,278**	**1,772**	**2,339**	**4,003**	**200**	**1,190**	**2,140**	**6,010**				
J64	Confluence of James and Little James (NRCS 2)	16,315	12.17	**4,800**	**3,250**	**2,914**	**1,743**	**912**	**1,502**	**2,095**	**2,777**	**4,834**	**300**	**1,785**	**3,205**	**8,850**				
J64B	**James Creek below Jamestown (Jarrett #12)**	10,877	16.69	**3,300**	**3,824**	**3,871**	**2,513**	**954**	**1,647**	**2,395**	**3,304**	**6,195**	**355**	**2,180**	**3,930**	**10,880**				
J57b	James Creek at Confluence with Lefthand Creek	0	18.72		4,095	4,139	2,824	973	1,710	2,517	3,512	6,728	390	2,660	4,810	13,500				
J49	Confluence of LHC and Spring Gulch	54,805	16.35		1,325	1,390	684	333	606	898	1,246	2,349								
J49A	**Lefthand Creek at Rowena (Jarrett #13)**	51,901	17.79	**1,300**	**1,548**	**1,603**	**856**	**364**	**666**	**988**	**1,371**	**2,581**	400	1,800	3,180	9,060				
J57a	Lefthand Creek above confluence with James Creek	29,455	22.92		2,626	2,575	1,619	502	920	1,359	1,886	3,536	430	2,100	3,690	10,430				
J57	Confluence of Lefthand Creek and James Creek	27,455	41.64		6,687	6,224	4,368	1,212	2,214	3,227	4,472	8,439								
J54	**Lefthand Creek below Old Stage Road (Jarrett #9)**	13,833	47.29	**3,520**	**7,734**	**7,389**	**5,148**	**1,260**	**2,336**	**3,438**	**4,804**	**9,220**	830	2,850	4,940	11,630				
J67	LHC confluence with Spruce Gulch	8,659	50.54		8,461	8,083	5,539	1,340	2,493	3,678	5,151	9,975	890	3,190	5,420	12,540	1,091	3,110	4,810	12,965
J74	Confluence of LHC and Geer Canyon	3,643	57.72		9,394	9,269	6,332	1,457	2,743	4,085	5,771	11,388								
Outlet1	**Lefthand Creek at Hwy 36**	0	58.12		9,474	9,362	6,358	1,469	2,765	4,117	5,822	11,493	1,035	4,145	6,700	14,990				

Appendix C.2



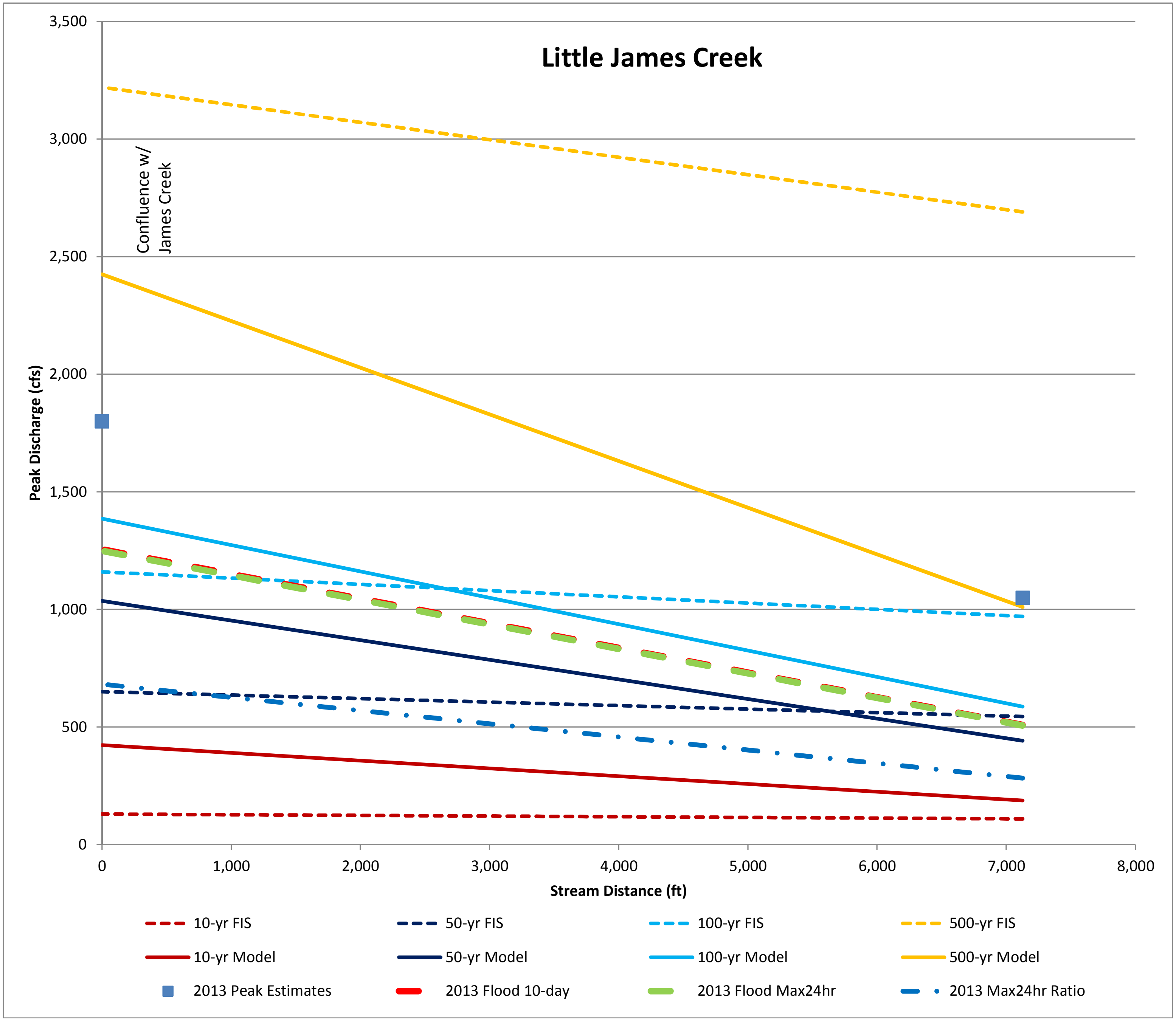
Lefthand Creek
Peak Discharge (cfs)
Stream Distance (ft)
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
10,000
20,000
30,000
40,000
50,000
60,000
US Hwy 36
Old Stage Road
Confluence w/ James Creek
Rowena
10-yr FIS
50-yr FIS
100-yr FIS
500-yr FIS
10-yr Model
50-yr Model
100-yr Model
500-yr Model
10-yr Gage FFA
50-yr Gage FFA
100-yr Gage FFA
500-yr Gage FFA
2013 Peak Estimates
2013 Flood 10-day
2013 Flood Max24hr
2013 Max24hr Ratio

Appendix C.3



James Creek
Peak Discharge (cfs)
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Confluence w/ Lefthand Creek
Downsream of Jamestown
Confluence w/ Little James Creek
0
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000
18,000
Stream Distance (ft)
10-yr FIS
50-yr FIS
100-yr FIS
500-yr FIS
10-yr Model
50-yr Model
100-yr Model
500-yr Model
2013 Peak Estimates
2013 Flood 10-day
2013 Flood Max24hr
2013 Max24hr Ratio

Appendix C.4



Little James Creek
Peak Discharge (cfs)
3,500
3,000
2,500
2,000
1,500
1,000
500
0
Confluence w/ James Creek
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
Stream Distance (ft)
10-yr FIS
50-yr FIS
100-yr FIS
500-yr FIS
10-yr Model
50-yr Model
100-yr Model
500-yr Model
2013 Peak Estimates
2013 Flood 10-day
2013 Flood Max24hr
2013 Max24hr Ratio

Big Thompson - Curve Number Optimization

Appendix C.5

Project: Big Thompson Calibrated   Optimization Trial: BT CN Optimization

Start of Trial:	09Sep2013, 00:00	Basin Model:	Big Thompson Calibrated
End of Trial:	15Sep2013, 23:45	Meteorologic Model:	BigThompson 2013 Flood
Compute Time:	12Feb2014, 20:21:51	Control Specifications:	Big Thompson - Lake Estes

Objective Function at Basin Element "J4055"

Start of Function :	09Sep2013, 00:00	Type :	Peak-Weighted RMS Error
End of Function :	15Sep2013, 23:45	Value :	742.0

Volume Units: IN

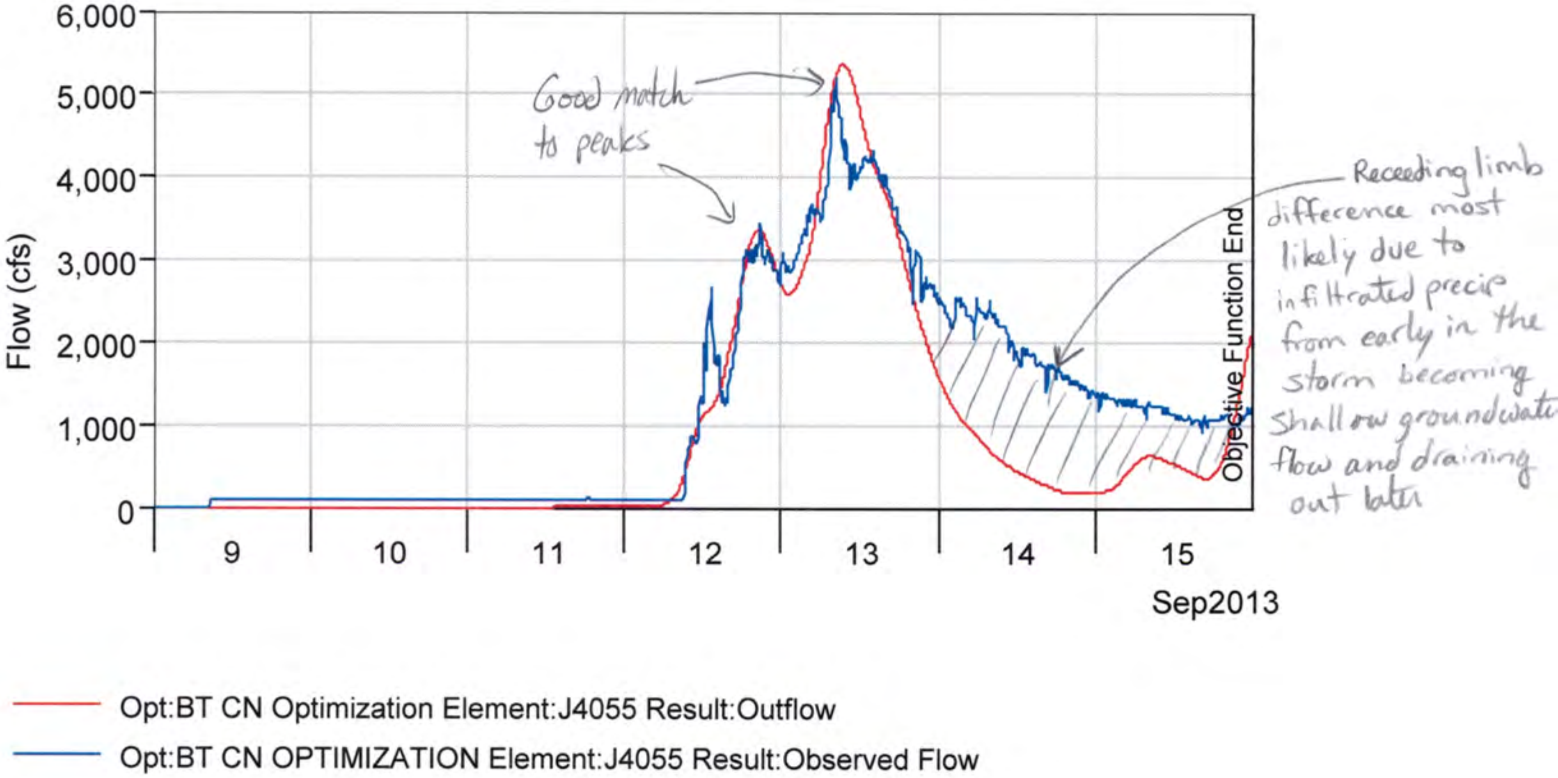
Measure	Simulated	Observed	Difference	Percent Difference
Volume (IN)	1.45	1.98	-0.53	-26.80
Peak Flow (CFS)	5347.3	5162.4	184.8	3.6
Time of Peak	13Sep2013, 09:15	13Sep2013, 08:15		
Time of Center of Mass	13Sep2013, 14:26	13Sep2013, 19:10		

Model optimization involved changing the initial abstraction (in) and the Curve Number to acheive a best fit to the Inflow hydrograph for Lake Estes

Curve Numbers increased by 12% over previously calibrated model

Initial Abstraction doubled ($I_a \approx 0.45$)

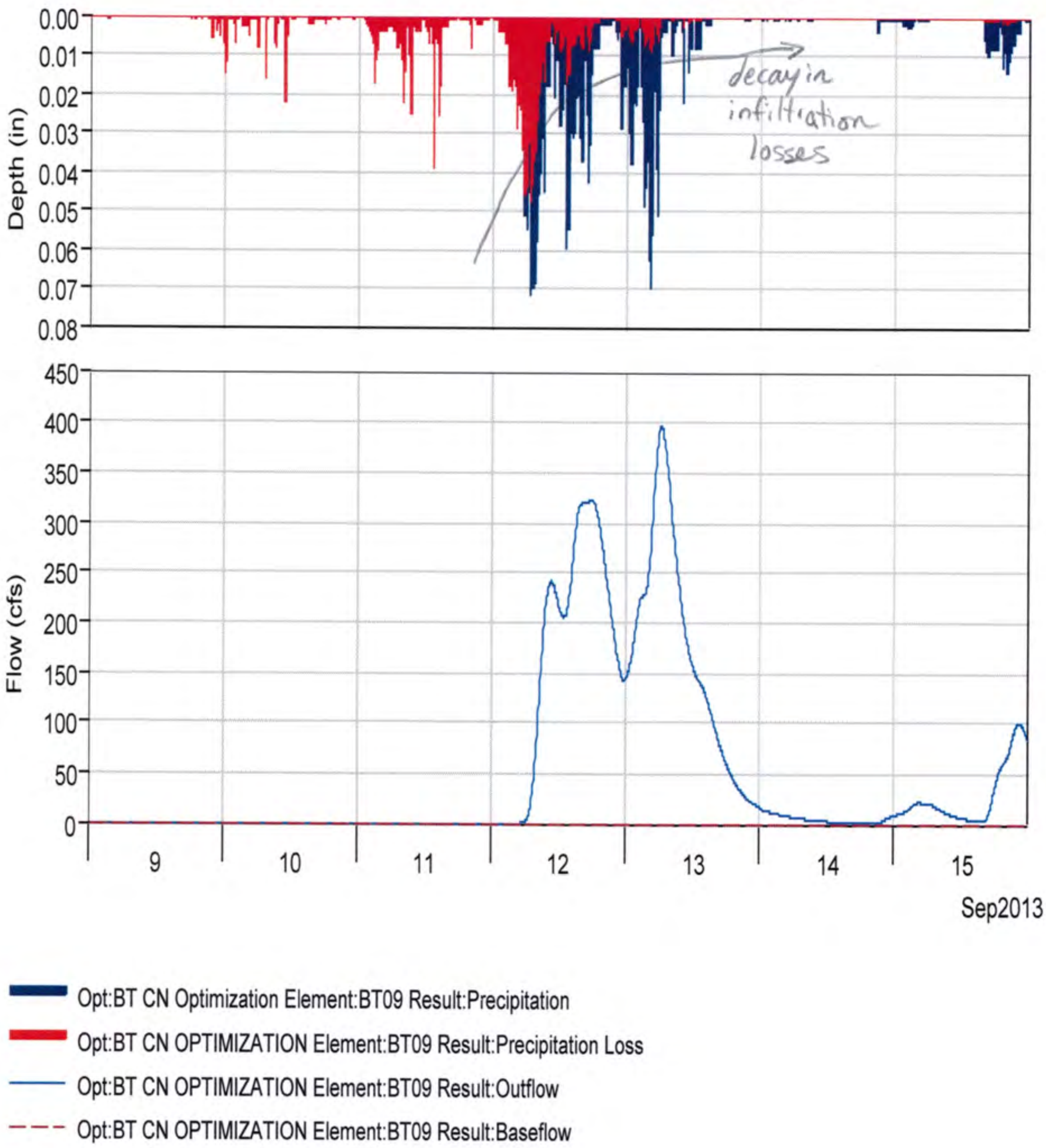
Lake Estes Inflow Hydrograph Comparison



2013 Flood
(9.48 inches precip)

Upstream of Lake Estes

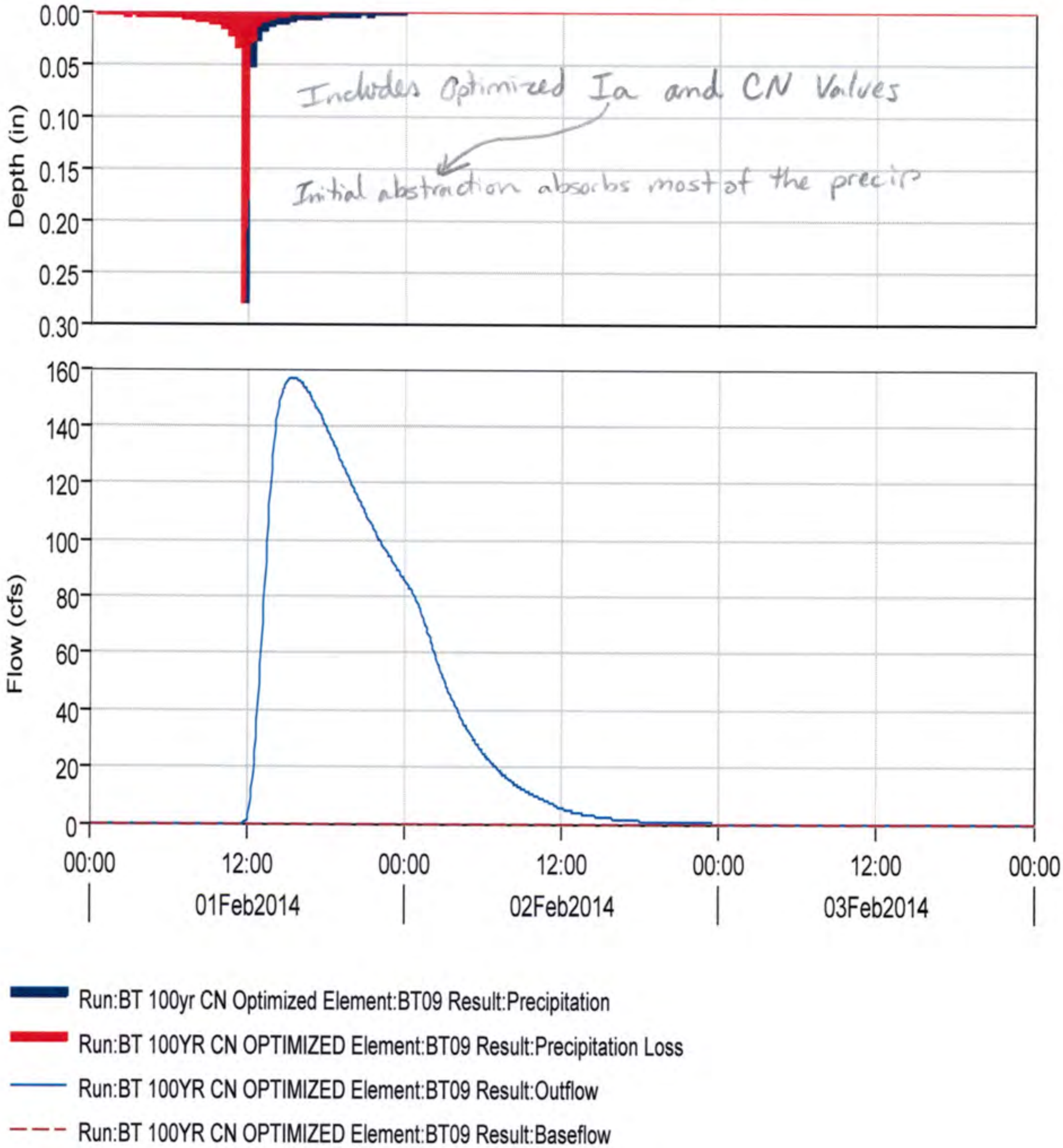
Subbasin "BT09" Results for Trial "BT CN Optimization"



NOAA 100-yr 24hr Storm
(4.42 inches Precip)

Upstream of Lake Estes

Subbasin "BT09" Results for Run "BT 100yr CN Optimized"

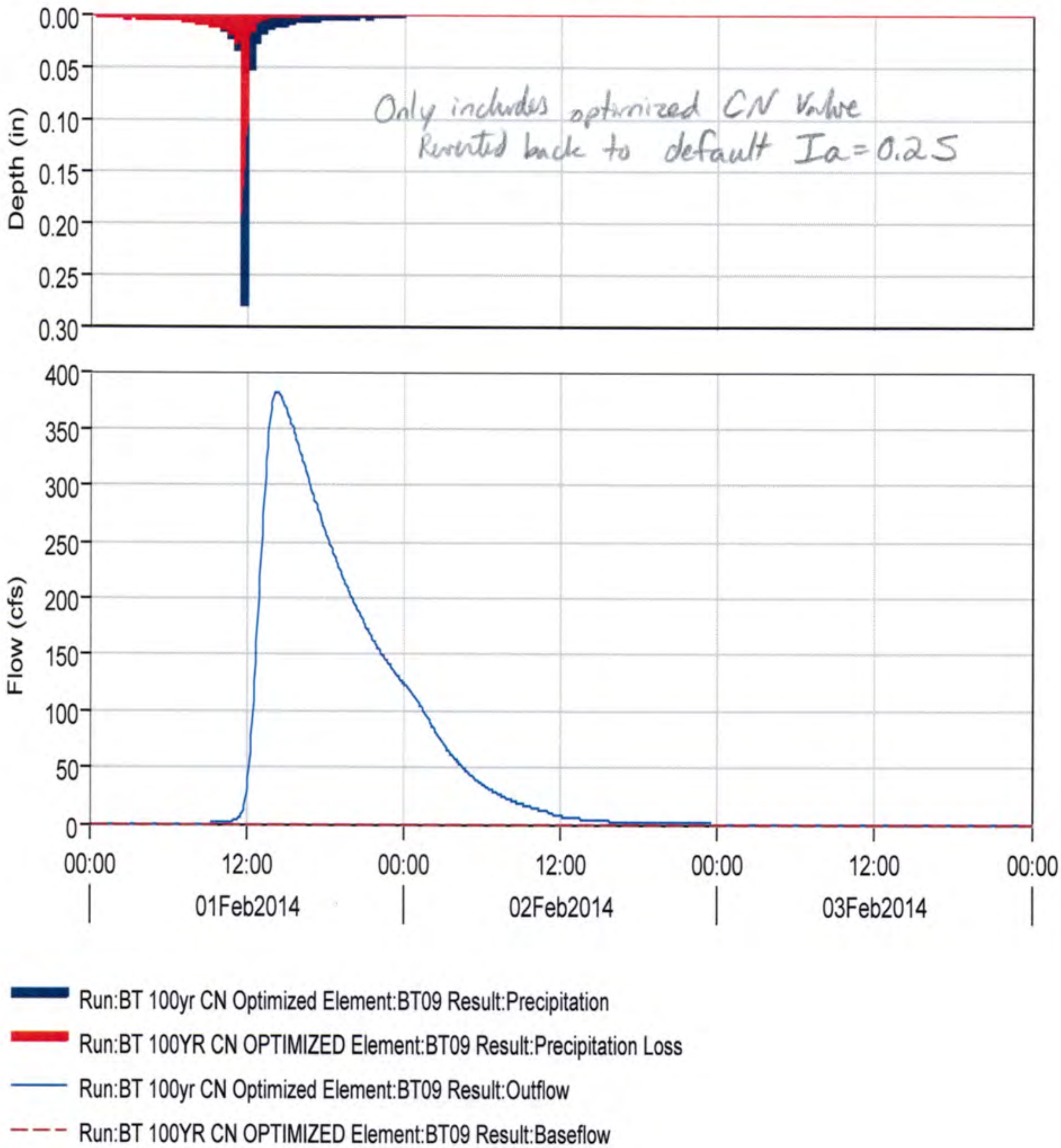


NOAA 100-yr 24 hr Storm

(4.42 inches precip)

← upstream of lake Estes

Subbasin "BT09" Results for Run "BT 100yr CN Optimized"



Big Thompson - Initial + Constant Loss Method

Project: Big Thompson Calibrated   Optimization Trial: BT Constant Optimization

Start of Trial: 09Sep2013, 00:00   Basin Model: BT Constant Loss
End of Trial: 15Sep2013, 23:45   Meteorologic Model: BigThompson 2013 Flood
Compute Time: 12Feb2014, 18:19:07   Control Specifications: Big Thompson - Lake Estes

Objective Function at Basin Element "J4055"

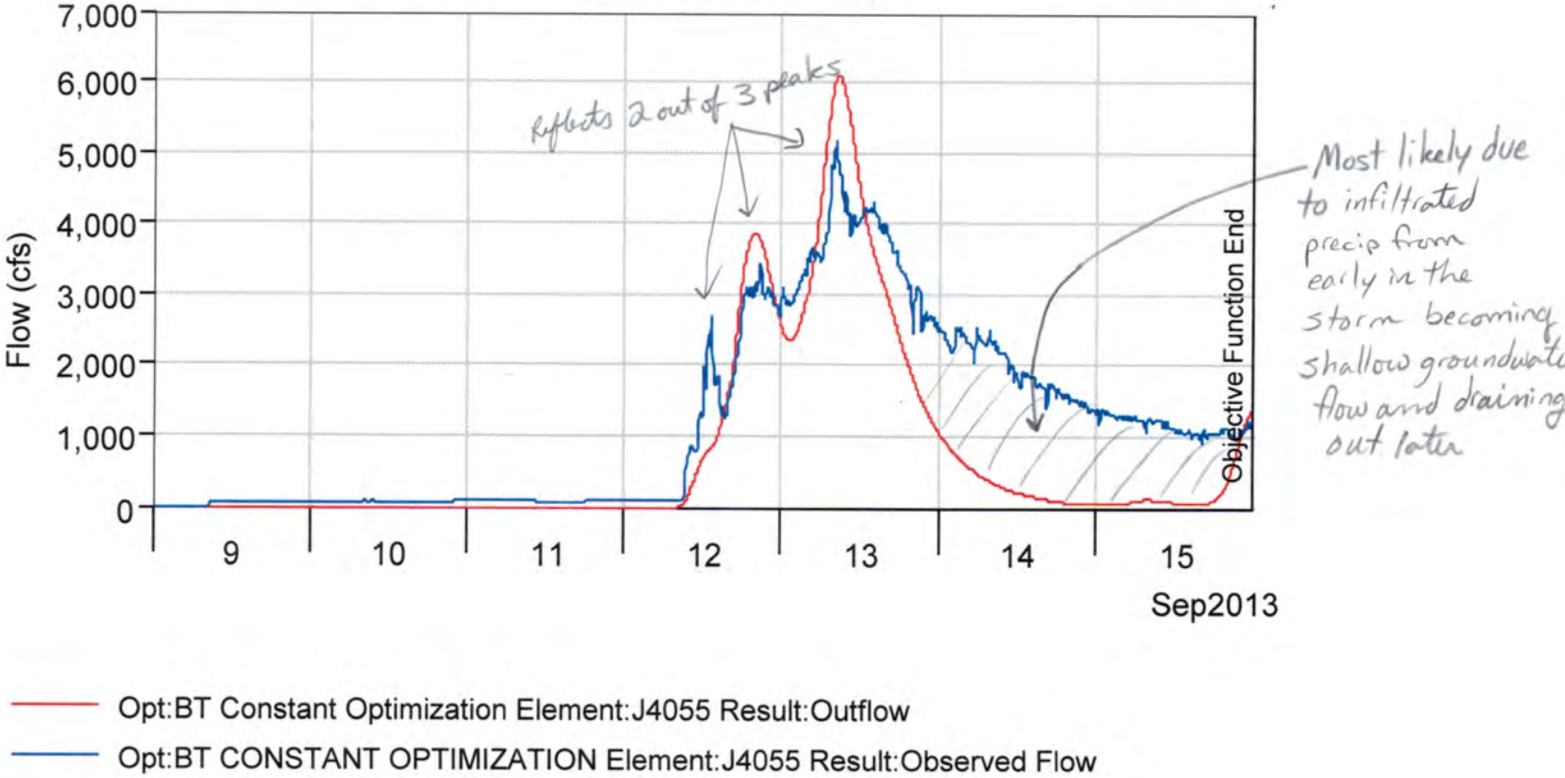
Start of Function : 09Sep2013, 00:00   Type : Peak-Weighted RMS Error
End of Function : 15Sep2013, 23:45   Value : 994.3

Volume Units: IN

Measure	Simulated	Observed	Difference	Percent Difference
Volume (IN)	1.23	1.98	-0.75	-37.74
Peak Flow (CFS)	6088.4	5162.4	926.0	17.9
Time of Peak	13Sep2013, 08:40	13Sep2013, 08:15		
Time of Center of Mass	13Sep2013, 10:16	13Sep2013, 19:10		

Model Optimization involved changing Initial Loss (in) and Constant Infiltration rate (fc) to acheive a best fit to the Inflow Hydrograph for Lake Estes.

Lake Estes Inflow Hydrograph Comparison



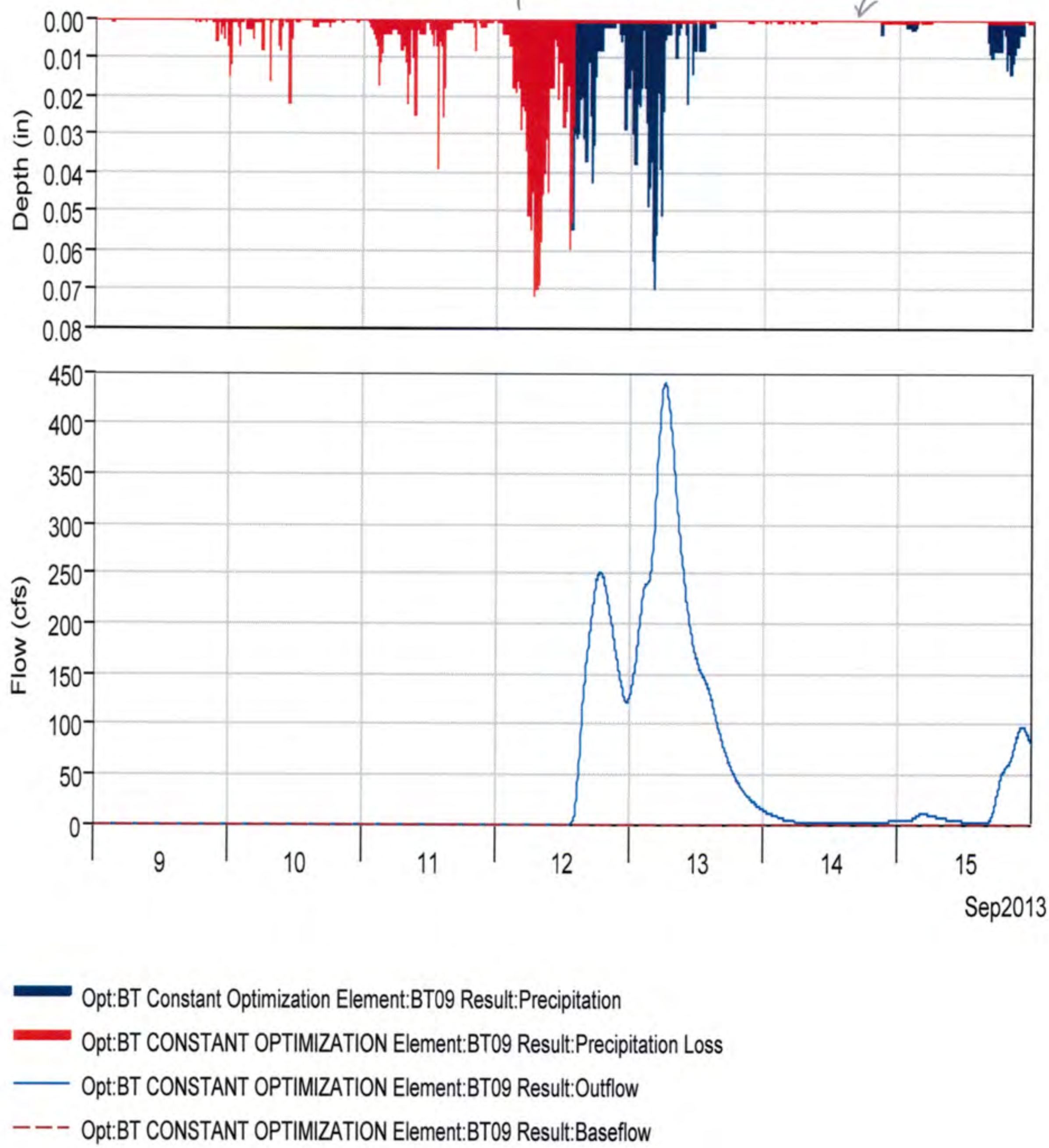
2013 Flood (9.48 inches precip)

upstream of Lake Estes

Optimization dominated by Initial loss

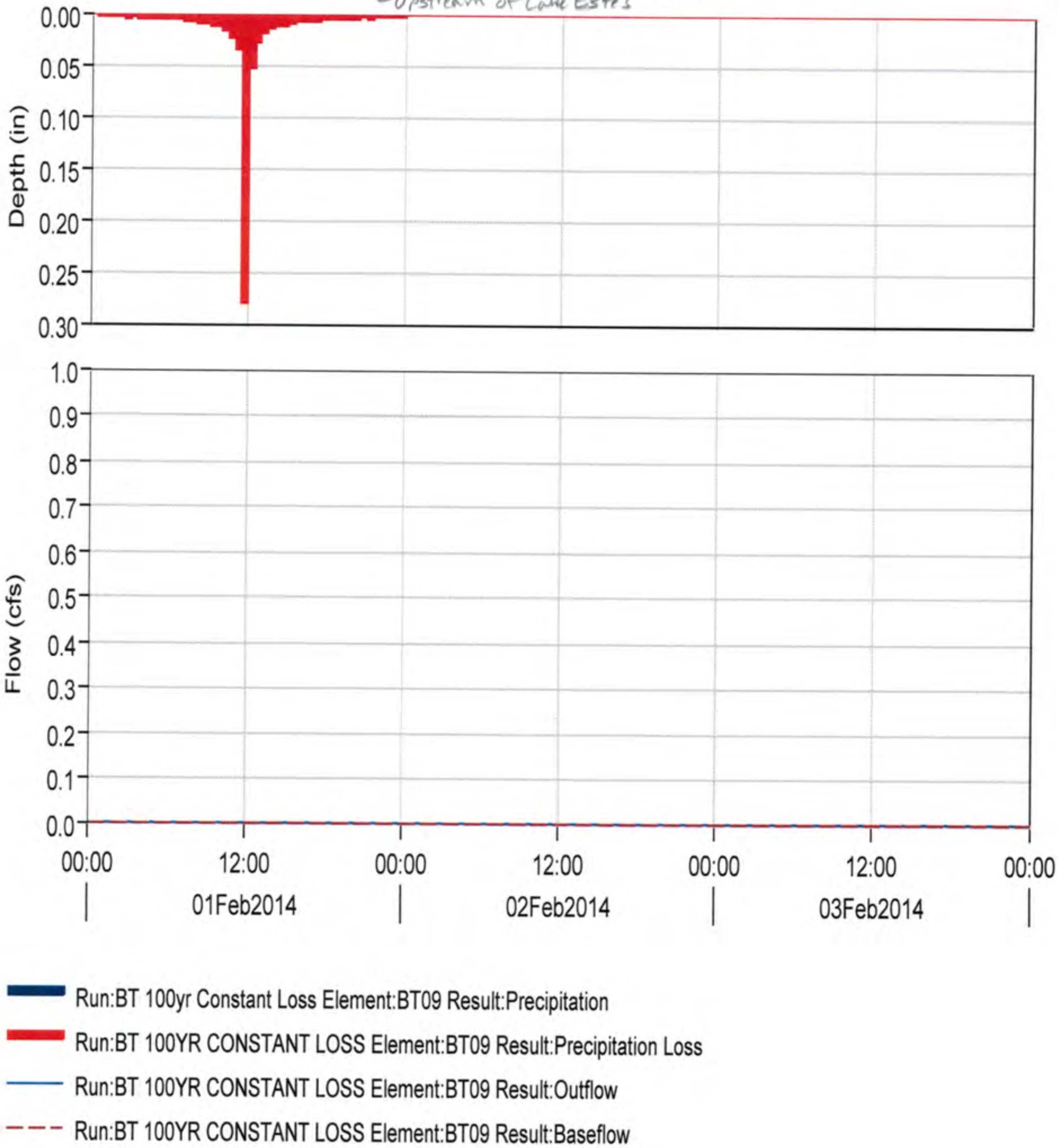
Constant infiltration rate is very low

Subbasin "BT09" Results for Trial "BT Constant Optimization"



100yr
NOAA 24-hr Storm
(4.42 inches)

Subbasin "BT09" Results for Run "BT 100yr Constant Loss"
Upstream of Lake Estes



Big Thompson - Green Ampt Optimization

Project: Big Thompson Calibrated   Optimization Trial: Green Ampt Optimization

Start of Trial:	09Sep2013, 00:00	Basin Model:	BT GreenAmpt Loss
End of Trial:	15Sep2013, 23:45	Meteorologic Model:	BigThompson 2013 Flood
Compute Time:	12Feb2014, 17:31:26	Control Specifications:	Big Thompson - Lake Estes

Objective Function at Basin Element "J4055"

Start of Function :	09Sep2013, 00:00	Type :	Peak-Weighted RMS Error
End of Function :	15Sep2013, 23:45	Value :	1411.3

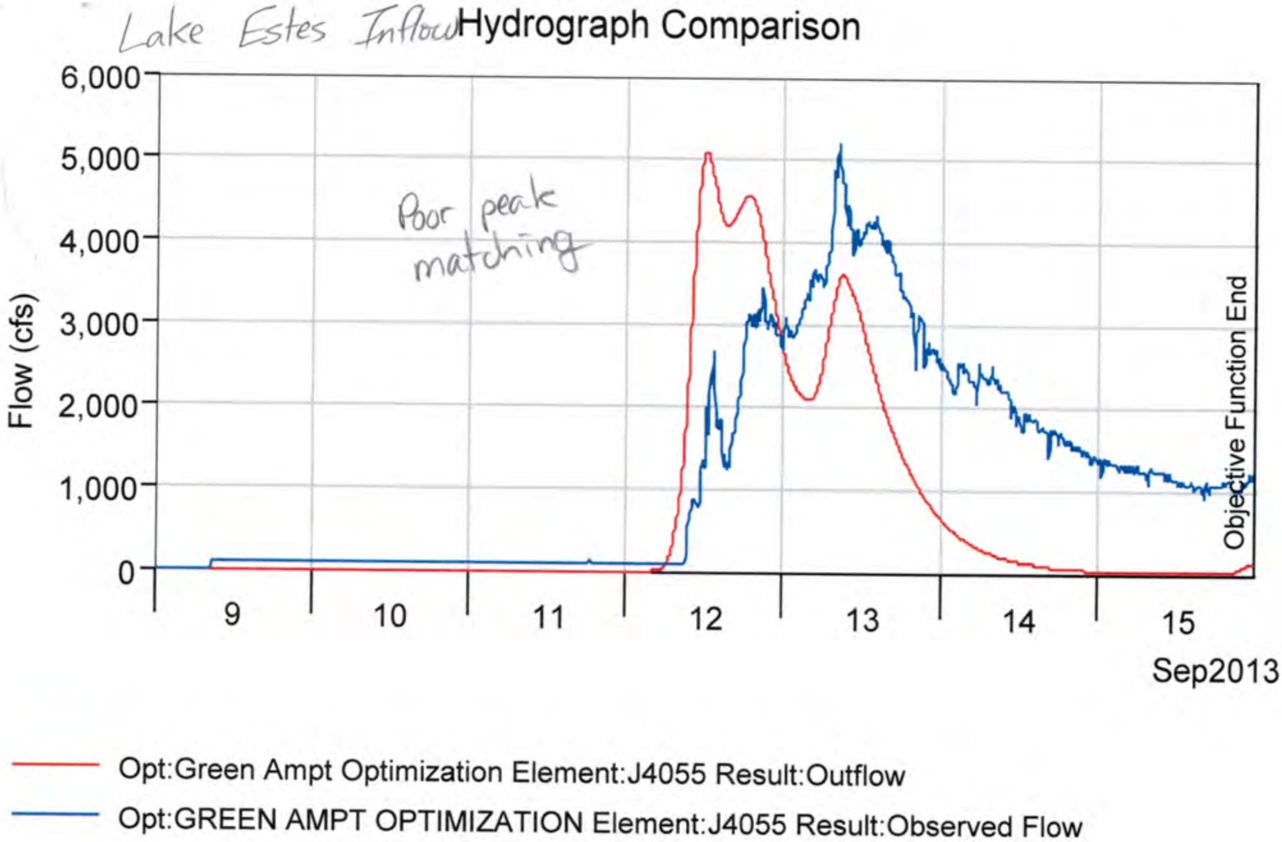
Volume Units: IN

Measure	Simulated	Observed	Difference	Percent Difference
Volume (IN)	1.20	1.98	-0.78	-39.42
Peak Flow (CFS)	5061.0	5162.4	-101.4	-2.0
Time of Peak	12Sep2013, 12:05	13Sep2013, 08:15		
Time of Center of Mass	13Sep2013, 01:37	13Sep2013, 19:10		

Model optimization involved changing:

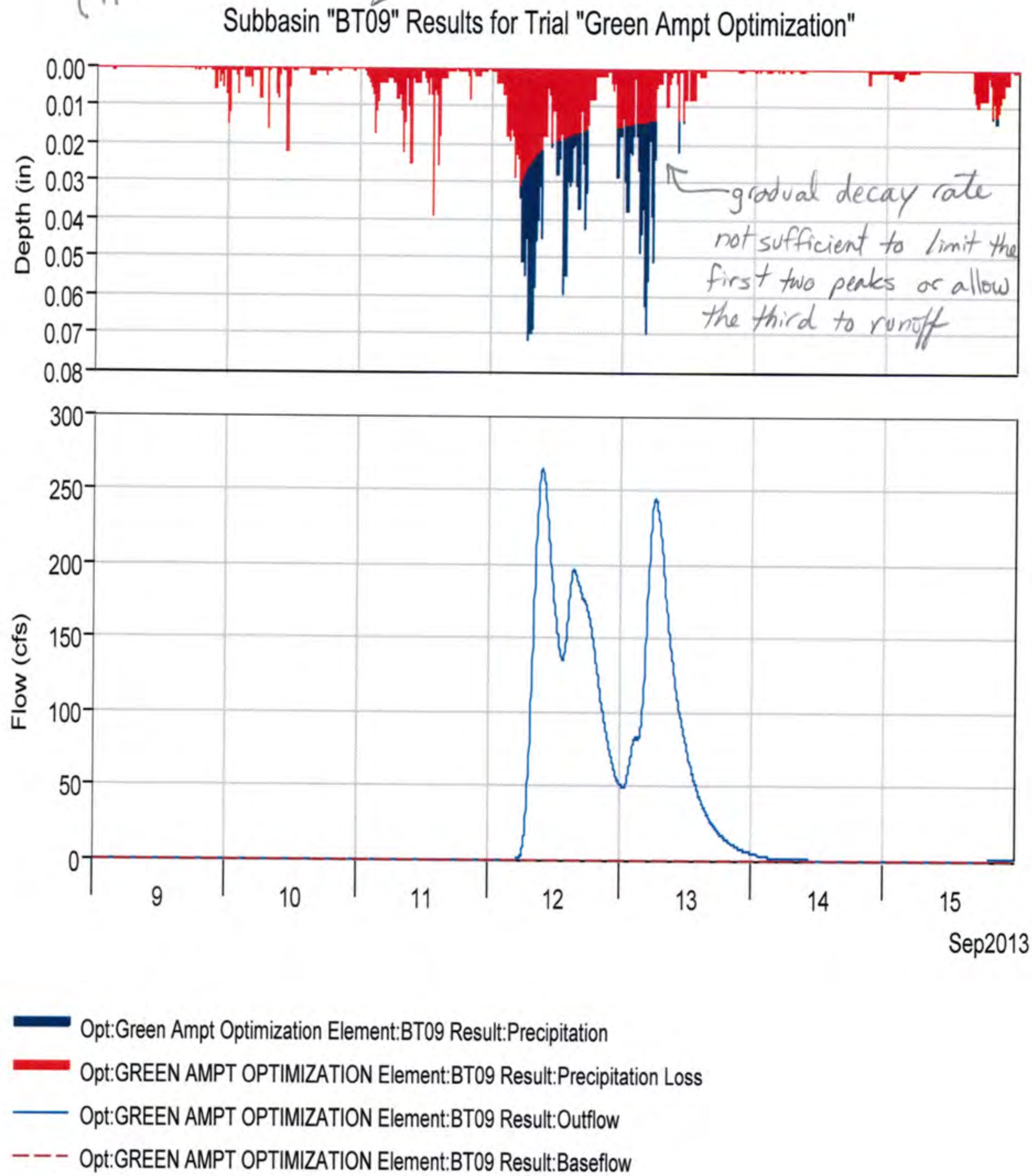
- Initial Water Content
- Saturated Water Content
- Wetting Front Suction
- Hydraulic Conductivity

to acheive a best fit to the inflow hydrograph for Lake Estes

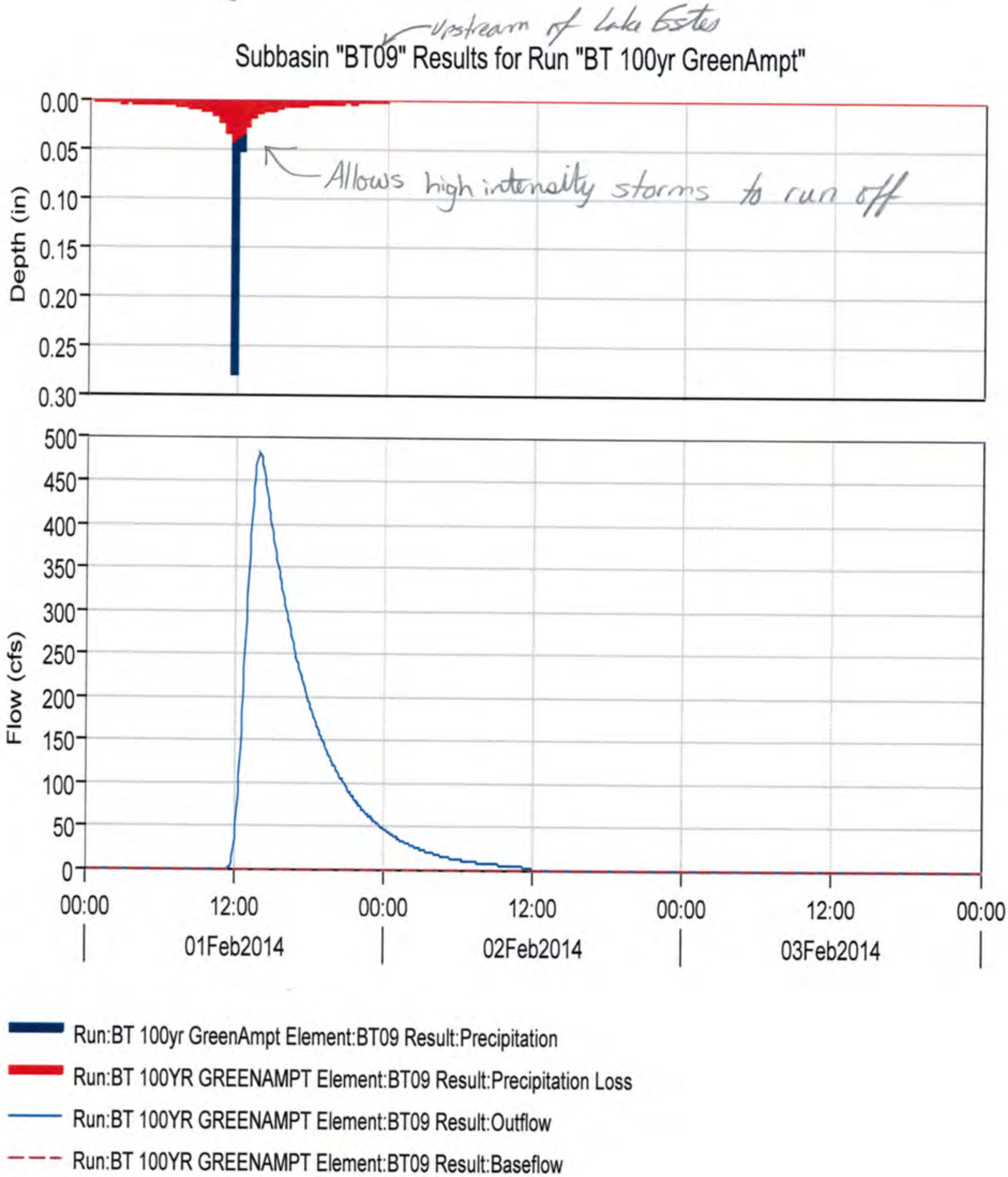

Lake Estes Inflow Hydrograph Comparison
Poor peak matching
6,000
5,000
4,000
3,000
2,000
1,000
0
Flow (cfs)
Objective Function End
9
10
11
12
13
14
15
Sep2013
Opt:Green Ampt Optimization Element:J4055 Result:Outflow
Opt:GREEN AMPT OPTIMIZATION Element:J4055 Result:Observed Flow

2013 Flood
(9.48 inches precip)

Upstream of Lake Estes



NOAA 100-yr 24 hr Storm
(4.42 inches precip)



Big Thompson - Exponential Loss Optimization

Project: Big Thompson Calibrated   Optimization Trial: BT Exponential Optimization

Start of Trial:	09Sep2013, 00:00	Basin Model:	BT Exponential Loss
End of Trial:	15Sep2013, 23:45	Meteorologic Model:	BigThompson 2013 Flood
Compute Time:	12Feb2014, 18:45:36	Control Specifications:	Big Thompson - Lake Estes

Objective Function at Basin Element "J4055"

Start of Function :	09Sep2013, 00:00	Type :	Peak-Weighted RMS Error
End of Function :	15Sep2013, 23:45	Value :	971.4

Volume Units: IN

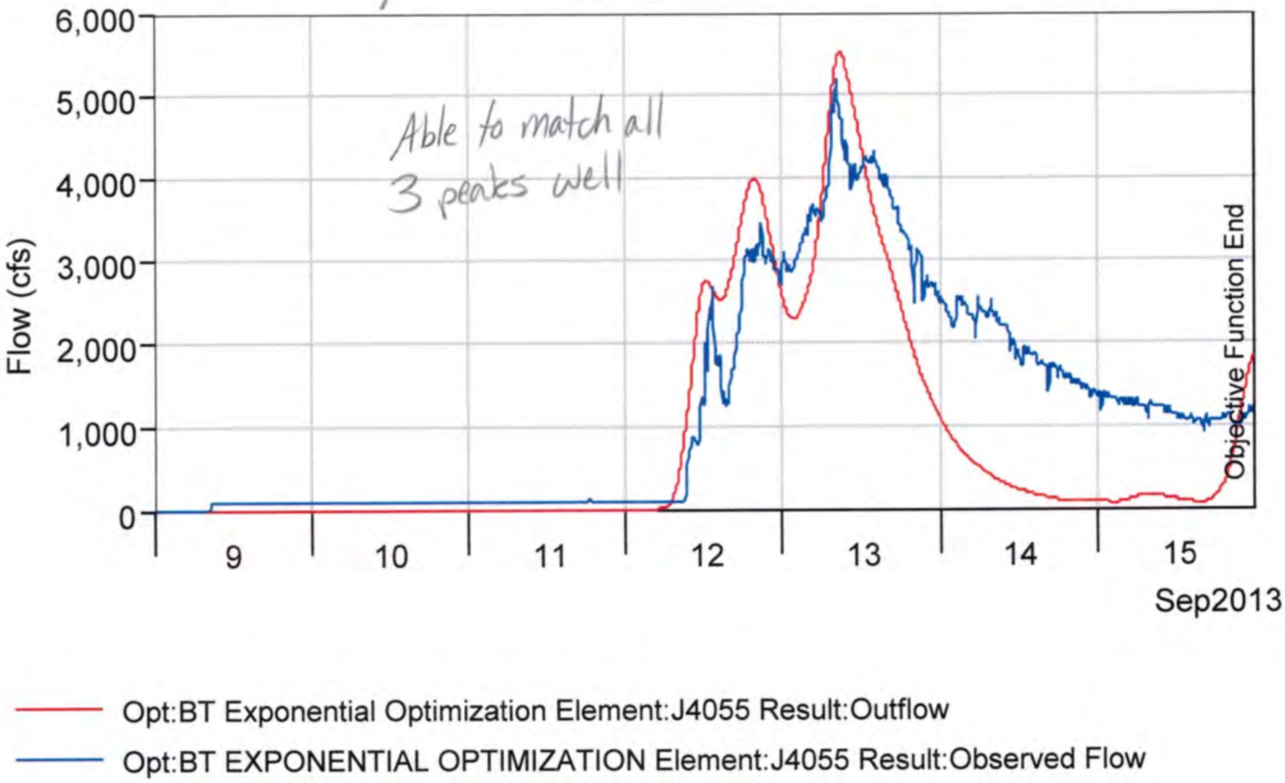
Measure	Simulated	Observed	Difference	Percent Difference
Volume (IN)	1.35	1.98	-0.63	-31.94
Peak Flow (CFS)	5498.0	5162.4	335.6	6.5
Time of Peak	13Sep2013, 08:55	13Sep2013, 08:15		
Time of Center of Mass	13Sep2013, 08:54	13Sep2013, 19:10		

Model optimization involved changing:

- Initial Range
- Initial Loss Rate Coefficient
- Coefficient Ratio

to acheive a best fit to the inflow hydrograph for Lake Estes

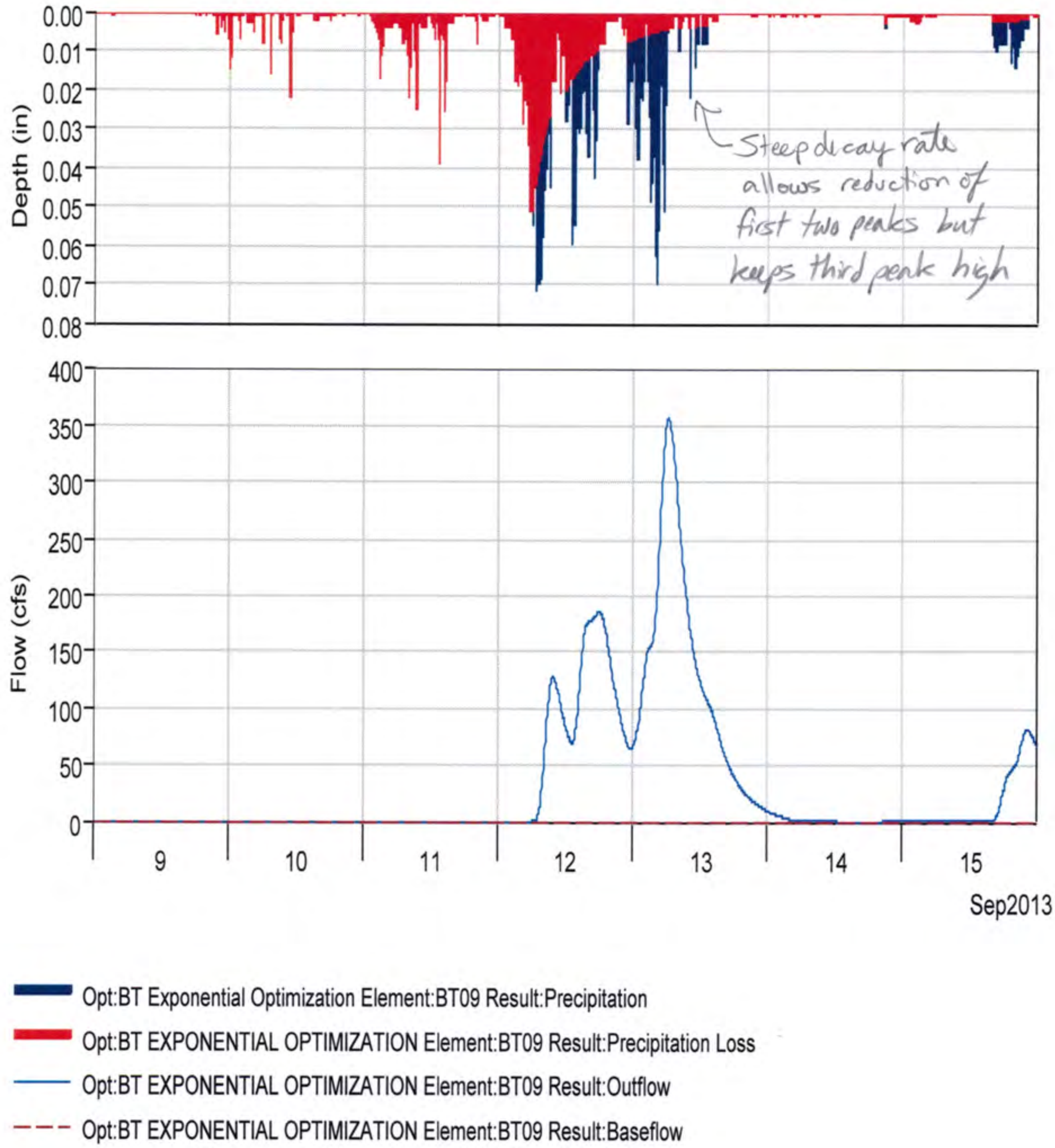
Lake Estes Inflow Hydrograph Comparison



2013 Flood
(9.48 inches Precip)

Upstream of Lake Estes

Subbasin "BT09" Results for Trial "BT Exponential Optimization"



Project: Big Thompson Calibrated
Simulation Run: BT Exponential Loss Junction: J4055

Start of Run:	09Sep2013, 00:00	Basin Model:	BT Exponential Loss
End of Run:	15Sep2013, 23:45	Meteorologic Model:	BigThompson 2013 Flood
Compute Time:	12Feb2014, 19:26:35	Control Specifications:	Big Thompson - Lake Estes

Volume Units: IN

vs. 5498 cfs for Optimization Routine

Computed Results

Peak Outflow :	4424.0 (CFS)	Date/Time of Peak Outflow :	13Sep2013, 08:50
Total Outflow :	1.02 (IN)		

vs. 1.35 inches for Optimization Routine

Observed Hydrograph at Gage Lake Estes Inflow Shifted

Peak Discharge :	5162.44 (CFS)	Date/Time of Peak Discharge :	13Sep2013, 08:15
Avg Abs Residual :	635.67 (CFS)		
Total Residual :	-0.96 (IN)	Total Obs Q :	1.98 (IN)

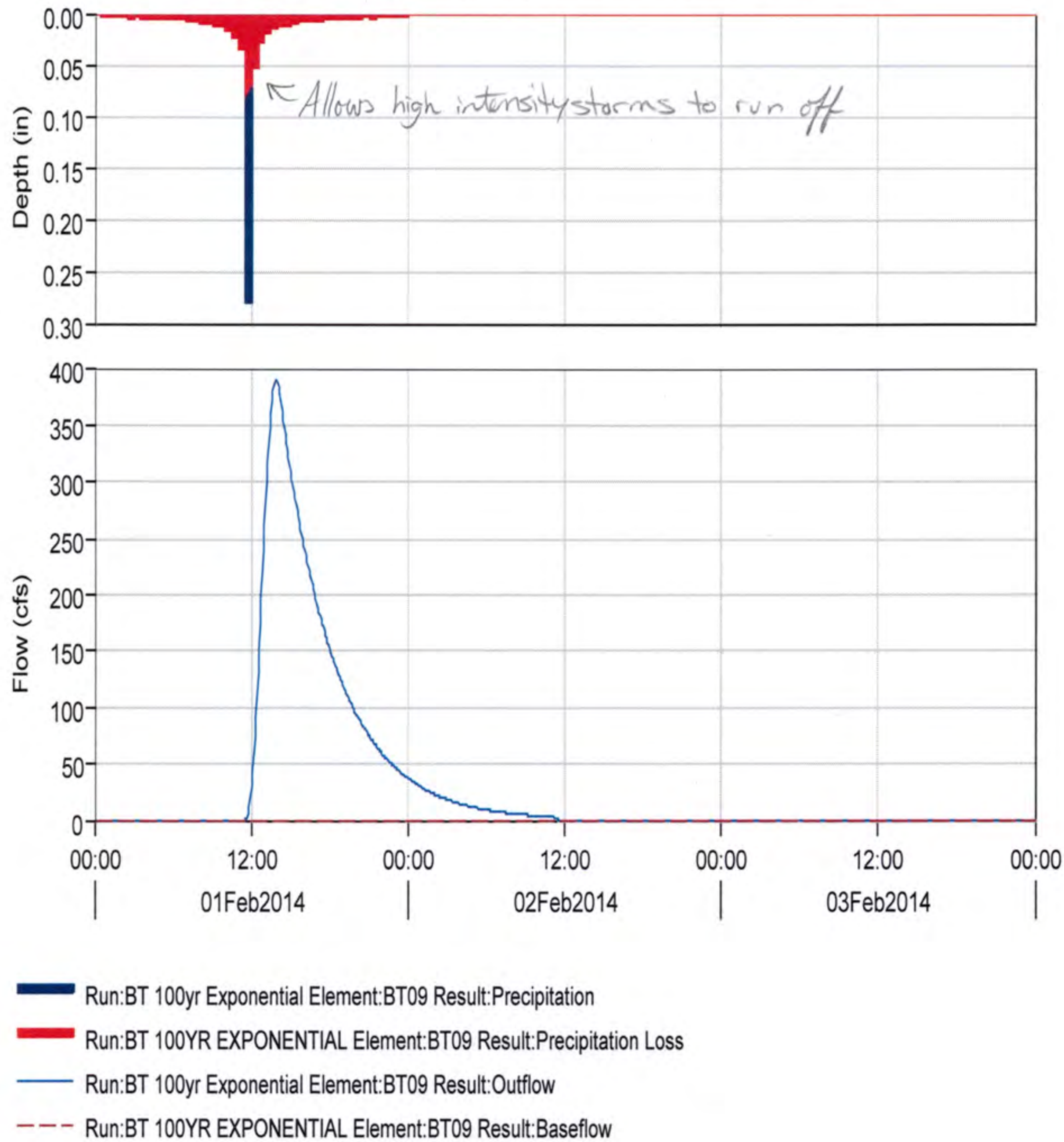
When the optimized values for:
- Initial Range
- Initial Loss Rate Coefficient
- Coefficient Ratio

are plugged back into the model as hardwired inputs, they do not produce the same results as the optimization routine. This was checked several times and may be an issue with the program.

NOAA 100yr 24hr Storm
(4.42 inches precip)

upstream of Lake Estes

Subbasin "BT09" Results for Run "BT 100yr Exponential"



Big Thompson - Curve Number Optimization for Max 24-hr Rainfall Period

Project: Big Thompson Calibrated Optimization Trial: BT CN Optimization Max24hr

Start of Trial:	12Sep2013, 04:15	Basin Model:	Big Thompson Calibrated
End of Trial:	15Sep2013, 04:15	Meteorologic Model:	Big Thompson Max 24hr
Compute Time:	12Feb2014, 21:31:54	Control Specifications:	Big Thompson Max 24hr

Objective Function at Basin Element "J4055"

Start of Function :	12Sep2013, 04:15	Type :	Peak-Weighted RMS Error
End of Function :	15Sep2013, 04:15	Value :	906.8

Volume Units: IN

Measure	Simulated	Observed	Difference	Percent Difference
Volume (IN)	1.30	1.71	-0.41	-23.81
Peak Flow (CFS)	5270.9	5162.4	108.5	2.1
Time of Peak	13Sep2013, 09:20	13Sep2013, 08:15		
Time of Center of Mass	13Sep2013, 09:39	13Sep2013, 15:56		

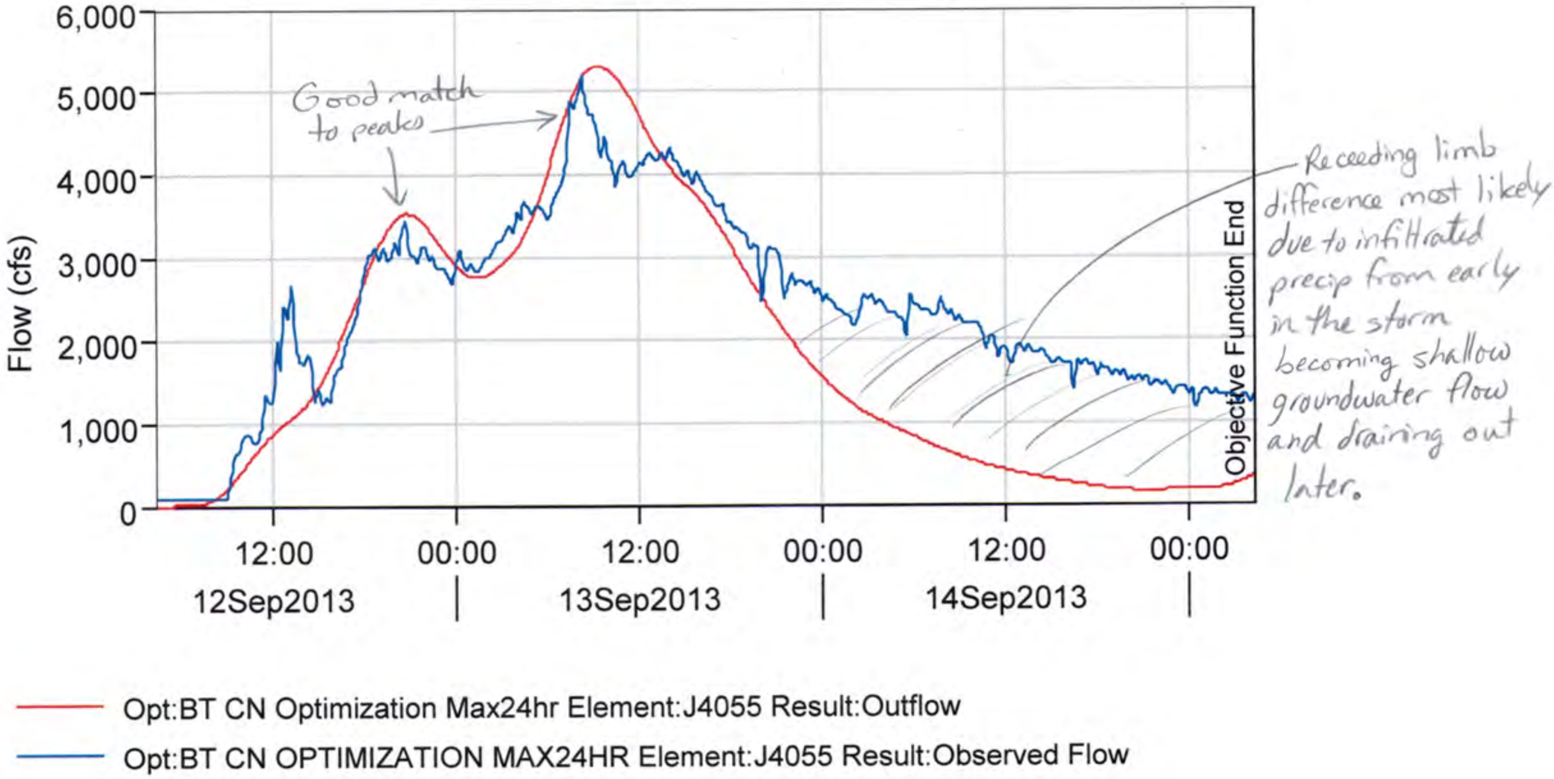
even better fit

Model optimization involved changing the initial abstraction (in) and the Curve Number to acheive a best fit to the inflow hydrograph for Lake Estes during the 24-hr period of maximum rainfall.

Curve numbers only decreased by 1.4%

Initial abstraction was reduced by 30% ($Ia = 0.14S$)

Lake Estes Inflow Hydrograph Comparison (24-hr Period)

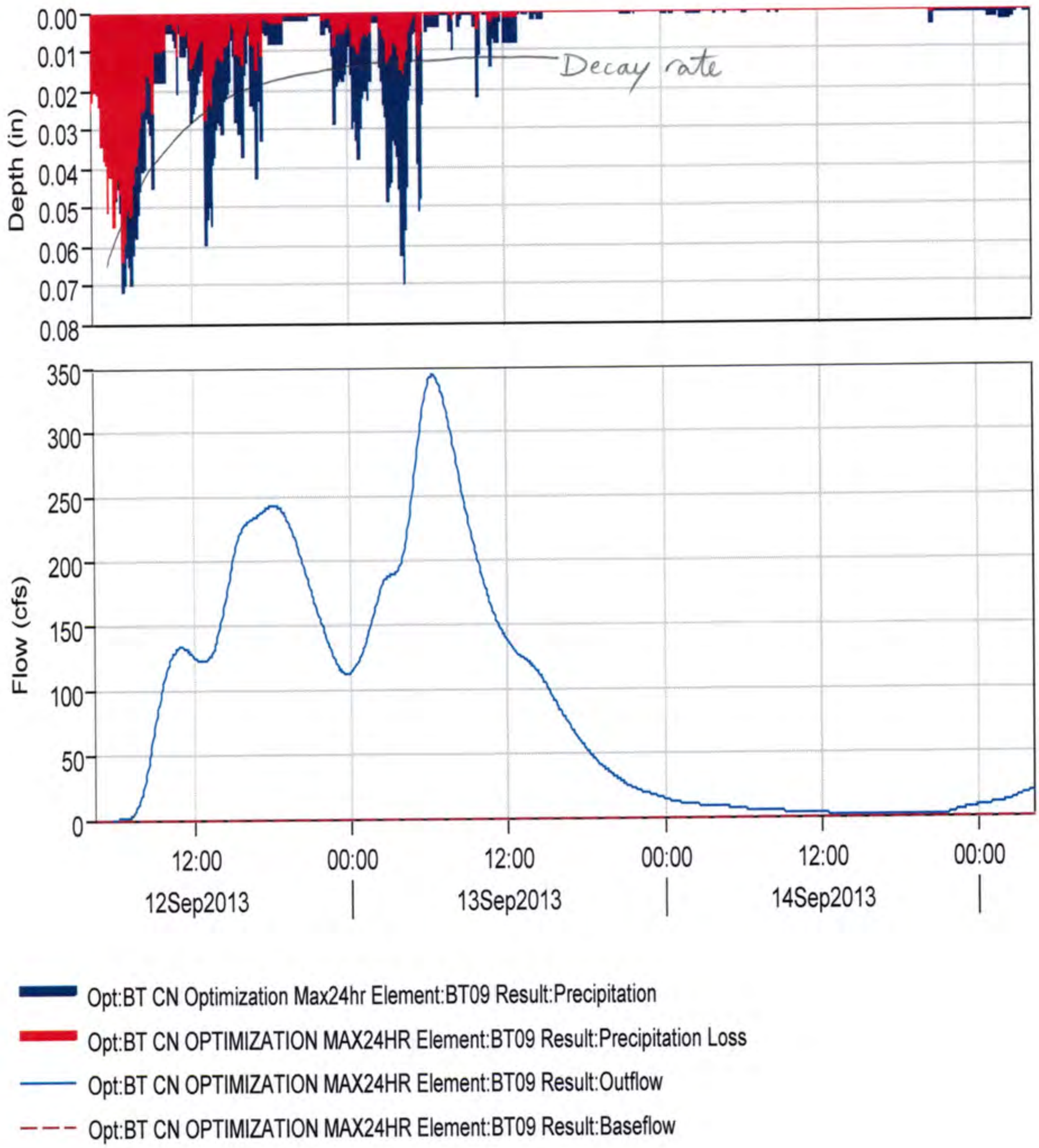


2013 Flood

(7.18 inches precip in 24-hr period)

upstream of Lake Estes

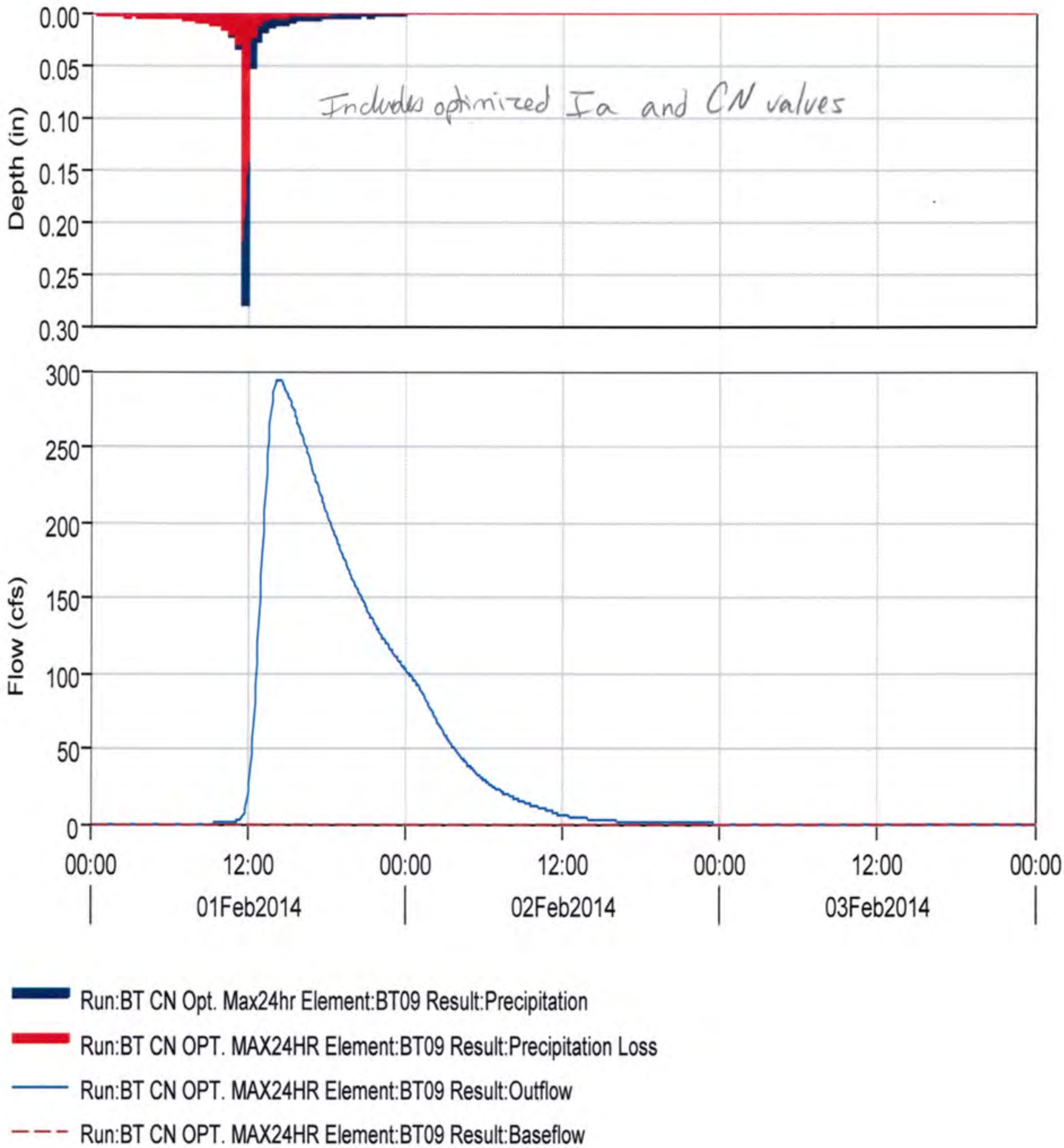
Subbasin "BT09" Results for Trial "BT CN Optimization Max24hr"



NOAA 100-yr 24-hr Storm
(4.42 inches Precip)

upstream of Lake Estes

Subbasin "BT09" Results for Run "BT CN Opt. Max24hr"



Appendix C.6

Lefthand Watershed - HEC-HMS Model Inputs

Model ID	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
LH220A	LH220A (Upper Little James Creek)	1.2	79.2	75	0.8	0.15	1.65	0.90	606	1.31					
J64A	J64A (Upper Little James Creek)														
R70	R70 (Little James Creek)										7128	0.065	0.15	0.15	0.15
LH220	LH220 (Lower Little James Creek)	1.8	78.9	74	0.8	0.15	2.27	0.80	661	1.38					
J64C	J64C (Lower Little James Creek)														
LH240	LH240 (James Creek above confluence)	9.1	79.9	76	0.8	0.15	6.40	2.80	400	3.20					
J64D	James Creek above confluence w/ Little James Ck.														
J64	J64 (Confluence of James and Little James)														
R170	R170 (James Creek through Jamestown)										5438	0.037	0.15	0.15	0.15
LH260	LH260 (James Creek below Jamestown)	4.5	46.7	53	0.4	0.15	2.94	1.25	680	1.74					
J64B	J64B (James Creek below Jamestown)														
R90	R90 (James Creek above confluence with LHC)										10877	0.037	0.15	0.15	0.15
LH260A	LH260A (James Creek at Confluence with LHC)	2.0	47.0	54	0.4	0.15	2.29	1.00	568	1.53					
J57b	James Creek above confluence w/ Lefthand Creek														
LH300	LH300 (Upper Lefthand Creek)	12.2	67.0	70	0.4	0.15	8.83	4.75	340	4.35					
LH290	LH290 (Spring Gulch)	4.1	65.0	67	0.4	0.15	4.72	1.90	339	2.61					
J49	J49 (Confluence of LHC and Spring Gulch)														
R160	R160 (LHC above Lickskillet Road)										2904	0.029	0.15	0.15	0.15
LH270A	LH270A (Lefthand Creek at Lickskillet Road)	1.4	65.0	67	0.4	0.15	1.42	0.50	915	0.96					
J49A	J49A (Lefthand Creek at Lickskillet Road)														
R130	R130 (LHC above confluence with James Creek)										24446	0.045	0.15	0.15	0.15
LH270	LH270 (Lefthand Creek upstream of James Creek)	5.1	65.0	68	0.4	0.15	4.45	2.15	404	2.59					
J57a	Lefthand Creek above confluence w/ James Creek														
J57	J57 (Confluence of LHC and JC)														
R100	R100 (LHC below confluence with JC)										13622	0.033	0.15	0.15	0.15
LH250	LH250 (Lefthand Creek above Old Stage Road)	2.7	50.1	57	0.4	0.15	3.28	1.50	579	1.96					
LH280	LH280 (Six Mile Canyon)	3.0	50.1	57	0.4	0.15	4.84	2.00	393	2.62					
J54	J54 (Lefthand Creek below Old Stage Road)														
R80	R80 (LHC below Old Stage Road)										5174	0.024	0.15	0.15	0.15
LH230	LH230	0.6	61.8	66	0.4	0.15	0.94	0.40	638	0.83					
LH210	LH210	2.7	63.5	68	0.4	0.15	3.21	1.50	717	1.88					
J67	J67 (LHC confluence with Spruce Gulch)														
R60	R60 (LHC above confluence with Geer Canyon)										5016	0.027	0.15	0.15	0.15
LH160	LH160	2.9	50.8	59	0.4	0.15	2.90	1.40	517	1.88					
LH170	LH170	2.8	50.4	62	0.4	0.15	2.83	1.50	777	1.78					
J81	J81 (Confluence of Plumley and Geer Canyons)														
R30	R30 (Geer Canyon Creek)										6442	0.043	0.15	0.15	0.15
LH180	LH180	0.7	67.3	71	0.4	0.15	1.31	0.55	802	0.99					
LH200	LH200	0.7	61.4	66	0.4	0.15	1.47	0.65	289	1.28					
J74	J74 (Confluence of LHC and Geer Canyon)														
R40	R40 (LHC above Hwy 36)										3643	0.016	0.15	0.15	0.15
LH190	LH190	0.4	77.2	77	0.4	0.15	0.82	0.30	549	0.74					
Outlet1	Outlet1														

Appendix C.6 (cont.)

Time Hours	t/T	Type 1 Storm	Type 1A Storm	Type II Storm	Type III Storm
		(Cumulative Precipitation)/(Total Storm Precipitation)			
0	0.000	0.000	0.000	0.000	0.000
0.5	0.021	0.008	0.010	0.005	0.005
1	0.042	0.017	0.020	0.011	0.010
1.5	0.063	0.026	0.035	0.016	0.015
2	0.083	0.035	0.050	0.022	0.020
2.5	0.104	0.045	0.067	0.028	0.025
3	0.125	0.055	0.082	0.035	0.031
3.5	0.146	0.065	0.098	0.041	0.037
4	0.167	0.076	0.116	0.048	0.043
4.5	0.188	0.087	0.135	0.056	0.050
5	0.208	0.099	0.156	0.063	0.057
5.5	0.229	0.112	0.180	0.071	0.064
6	0.250	0.126	0.206	0.080	0.072
6.5	0.271	0.140	0.237	0.089	0.081
7	0.292	0.156	0.268	0.098	0.091
7.5	0.313	0.174	0.310	0.109	0.102
8	0.333	0.194	0.425	0.120	0.114
8.5	0.354	0.219	0.480	0.133	0.128
9	0.375	0.254	0.520	0.147	0.146
9.5	0.396	0.303	0.550	0.163	0.166
10	0.417	0.515	0.577	0.181	0.189
10.5	0.438	0.583	0.601	0.204	0.217
11	0.458	0.624	0.624	0.235	0.250
11.5	0.479	0.655	0.645	0.283	0.298
12	0.500	0.682	0.664	0.663	0.500
12.5	0.521	0.706	0.683	0.735	0.702
13	0.542	0.728	0.701	0.772	0.750
13.5	0.563	0.748	0.719	0.799	0.784
14	0.583	0.766	0.736	0.820	0.811
14.5	0.604	0.783	0.753	0.838	0.834
15	0.625	0.799	0.769	0.854	0.854
15.5	0.646	0.815	0.785	0.868	0.872
16	0.667	0.830	0.800	0.880	0.886
16.5	0.688	0.844	0.815	0.891	0.898
17	0.708	0.857	0.830	0.902	0.910
17.5	0.729	0.870	0.844	0.912	0.919
18	0.750	0.882	0.858	0.921	0.928
18.5	0.771	0.893	0.871	0.929	0.936
19	0.792	0.905	0.884	0.937	0.943
19.5	0.813	0.916	0.896	0.945	0.950
20	0.833	0.926	0.908	0.952	0.957
20.5	0.854	0.936	0.920	0.959	0.963
21	0.875	0.946	0.932	0.965	0.969
21.5	0.896	0.956	0.944	0.972	0.975
22	0.917	0.965	0.956	0.978	0.981
22.5	0.938	0.974	0.967	0.984	0.986
23	0.958	0.983	0.978	0.989	0.991
23.5	0.979	0.991	0.989	0.995	0.996
24	1.000	1.000	1.000	1.000	1.000

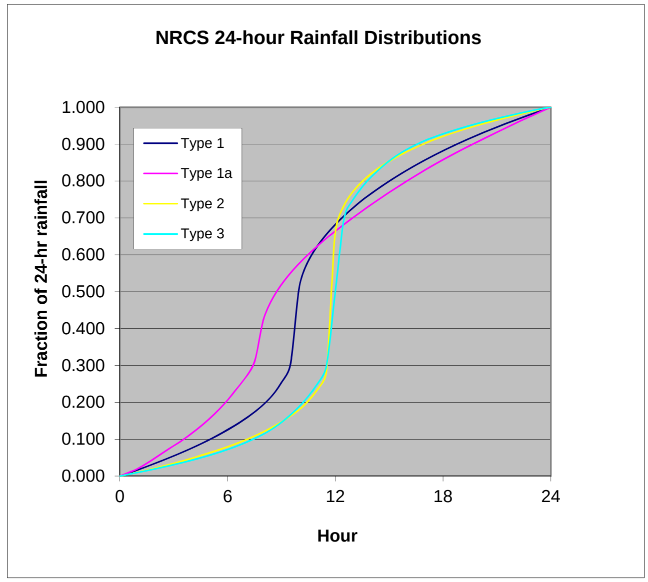


Figure 1: Geographic boundaries for the NRCS rainfall distributions

Lefthand Creek

	Raingage Zones	
	Zone 1 (West)	**Zone 2 (East)**
Lat	40.08393003	40.11537159
Long	-105.4683831	-105.34358
max 100-yr 1-hr Precip	1.92	2.08
10-yr, 24-hr	2.74	2.94
25-yr, 24-hr	3.45	3.72
50-yr, 24-hr	4.08	4.4
100-yr, 24-hr	4.76	5.14
500-yr, 24-hr	6.62	7.13
	LH220A	LH260
	LH220	LH260A
	LH240	LH270
	LH300	LH250
	LH290	LH280
	LH270A	LH230
		LH210
		LH200
		LH160
		LH170
		LH180
		LH190

Unadjusted NOAA Rainfall

Basin	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**500-yr**
LH160	2.94	3.72	4.4	5.14	7.13
LH170	2.94	3.72	4.4	5.14	7.13
LH180	2.94	3.72	4.4	5.14	7.13
LH190	2.94	3.72	4.4	5.14	7.13
LH200	2.94	3.72	4.4	5.14	7.13
LH210	2.94	3.72	4.4	5.14	7.13
LH220	2.74	3.45	4.08	4.76	6.62
LH220A	2.74	3.45	4.08	4.76	6.62
LH230	2.94	3.72	4.4	5.14	7.13
LH240	2.74	3.45	4.08	4.76	6.62
LH250	2.94	3.72	4.4	5.14	7.13
LH260	2.94	3.72	4.4	5.14	7.13
LH260A	2.94	3.72	4.4	5.14	7.13
LH270	2.94	3.72	4.4	5.14	7.13
LH270A	2.74	3.45	4.08	4.76	6.62
LH280	2.94	3.72	4.4	5.14	7.13
LH290	2.74	3.45	4.08	4.76	6.62
LH300	2.74	3.45	4.08	4.76	6.62

Lefthand Creek Appendix C.6 (continued)

NOAA Aerial Reduction (98% - 10 to 30 sq.mi.)

Basin	10-yr	25-yr	50-yr	100-yr	500-yr
LH160	2.88	3.65	4.31	5.04	6.99
LH170	2.88	3.65	4.31	5.04	6.99
LH180	2.88	3.65	4.31	5.04	6.99
LH190	2.88	3.65	4.31	5.04	6.99
LH200	2.88	3.65	4.31	5.04	6.99
LH210	2.88	3.65	4.31	5.04	6.99
LH220	2.69	3.38	4.00	4.66	6.49
LH220A	2.69	3.38	4.00	4.66	6.49
LH230	2.88	3.65	4.31	5.04	6.99
LH240	2.69	3.38	4.00	4.66	6.49
LH250	2.88	3.65	4.31	5.04	6.99
LH260	2.88	3.65	4.31	5.04	6.99
LH260A	2.88	3.65	4.31	5.04	6.99
LH270	2.88	3.65	4.31	5.04	6.99
LH270A	2.69	3.38	4.00	4.66	6.49
LH280	2.88	3.65	4.31	5.04	6.99
LH290	2.69	3.38	4.00	4.66	6.49
LH300	2.69	3.38	4.00	4.66	6.49

NOAA Aerial Reduction (96% - 30 to 50 sq.mi.)

Basin	10-yr	25-yr	50-yr	100-yr	500-yr
LH160	2.77	3.50	4.14	4.84	6.71
LH170	2.77	3.50	4.14	4.84	6.71
LH180	2.77	3.50	4.14	4.84	6.71
LH190	2.77	3.50	4.14	4.84	6.71
LH200	2.77	3.50	4.14	4.84	6.71
LH210	2.77	3.50	4.14	4.84	6.71
LH220	2.58	3.25	3.84	4.48	6.23
LH220A	2.58	3.25	3.84	4.48	6.23
LH230	2.77	3.50	4.14	4.84	6.71
LH240	2.58	3.25	3.84	4.48	6.23
LH250	2.77	3.50	4.14	4.84	6.71
LH260	2.77	3.50	4.14	4.84	6.71
LH260A	2.77	3.50	4.14	4.84	6.71
LH270	2.77	3.50	4.14	4.84	6.71
LH270A	2.58	3.25	3.84	4.48	6.23
LH280	2.77	3.50	4.14	4.84	6.71
LH290	2.58	3.25	3.84	4.48	6.23
LH300	2.58	3.25	3.84	4.48	6.23

Appendix C.6 (cont)

Lefthand Creek

HEC-HMS Design Point	Location Description	Area (sq. mi.)	Rainfall Depth-Area Reduction %
LH220A	Upper Little James Creek	1.20	100%
J64A	Upper Little James Creek (Jarrett #10)	1.20	100%
R70	Little James Creek	1.20	100%
LH220	Lower Little James Creek	1.84	100%
J64C	Lower Little James Creek (NRCS #1)	3.04	100%
LH240	Upper James Creek	9.13	100%
J64D	James Creek above confluence (Jarrett #11)	9.13	100%
J64	Confluence of James and Little James (NRCS #2)	12.17	98%
R170	James Creek through Jamestown	12.17	98%
LH260	James Creek below Jamestown	4.52	100%
J64B	James Creek below Jamestown (Jarrett #12)	16.69	98%
R90	James Creek channel above LHC	16.69	98%
LH260A	James Creek area above LHC	2.03	100%
J57b	James Creek above confluence with LHC	18.72	98%
LH300	Upper Lefthand Creek	12.20	98%
LH290	Spring Gulch	4.15	100%
J49	Confluence of LHC and Spring Gulch	16.35	98%
R160	LHC above Lickskillet Road	16.35	98%
LH270A	Lefthand Creek at Lickskillet Road	1.44	100%
J49A	Lefthand Creek at Lickskillet Road	17.79	98%
R130	LHC above confluence with James Creek	17.79	98%
LH270	Lefthand Creek upstream of James Creek	5.13	100%
J57a	Lefthand Creek upstream of James Creek	22.92	98%
J57	Confluence of LHC and JC	41.64	96%
R100	LHC below confluence with JC	41.64	96%
LH250	Lefthand Creek above Old Stage Road	2.68	100%
LH280	Six Mile Canyon	2.98	100%
J54	Lefthand Creek below Old Stage Road	47.29	96%
R80	LHC below Old Stage Road	47.29	96%
LH230	LH230	0.59	100%
LH210	LH210	2.66	100%
J67	LHC confluence with Spruce Gulch	50.54	96%
R60	LHC above confluence with Geer Canyon	50.54	96%
LH160	LH160	2.91	100%
LH170	LH170	2.85	100%
J81	Confluence of Plumley and Geer Canyons	5.75	100%
R30	Geer Canyon Creek	5.75	100%
LH180	LH180	0.72	100%
LH200	LH200	0.71	100%
J74	Confluence of LHC and Geer Canyon	57.72	96%
R40	LHC above Hwy 36	57.72	96%
LH190	LH190	0.41	100%
Outlet1	LHC at Hwy 36	58.12	96%

HEC-HMS Rainfall Depth-Area Reduction Zones	NOAA Atlas 2 Curves	
	Area (sq. mi.)	Rainfall Depth-Area Reduction %
0-10 mi^2 100%	0	100.0%
	5	99.0%
	10	98.5%
10-30 mi^2 98%	15	98.0%
	20	97.2%
30-50 mi^2 96%	30	96.5%
	40	95.8%
	50	95.2%
50-100 mi^2 94%	75	94.0%
	100	93.5%
100-400 mi^2 92%	125	93.0%
	150	92.5%
	200	92.0%
	250	91.7%
	300	91.4%
	350	91.1%
	400	90.8%
> 400 mi^2	> 400	N/A

Application of Rainfall Depth-Area Reduction for HEC-HMS Model

In order to evaluate the impacts of the NOAA Atlas 2 rainfall depth-area reduction factors on the Lefthand Creek watershed, several model scenarios were run using different rainfall depths. The three different scenarios included the unadjusted NOAA rainfall depth and two levels of reduced NOAA rainfall depths (98% and 96%). The results from each rainfall depth scenario were saved to a spreadsheet and the appropriate value at any given design point was determined based on the tributary area to that design point as shown in the table to the left. The steps to do this in HEC-HMS are described below.

1. Open the Basin Model "LH Max24hr Calibrated".
2. Open the Meteorological Model for the design storm of interest (e.g. NOAA 100-yr DARF) and select the "specified hyetograph".
3. Copy and Paste the desired rainfall depths (based on both design storm and depth-area reduction level) from the *LH Raingage Zones.xls* spreadsheet into the column for "Total Depth (in)" in the HEC-HMS user interface.
4. Run the HEC-HMS model and save the global summary results table to a summary spreadsheet.
5. Repeat Steps 3 and 4 with a different set of rainfall depths from the *LH Raingage Zones.xls* spreadsheet. This process must be repeated up to fifteen times to develop peak discharges for all five design storms and all three levels of rainfall depth-area reduction.
6. Once all of the model results have been produced, the summary spreadsheet can be used to determine the appropriate peak discharge at each design point using the table to the left.