Flow Frequency Evaluation For Three Watersheds in the Denver Metro Area

Van Bibber Creek, Lena Gulch, and Little Dry Creek

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1.0 INTRODUCTION

WWE conducted flow frequency analyses for streamgages in three watersheds in the western part of the Denver metropolitan area on behalf of the Mile High Flood District (MHFD). Annual peak flow data were analyzed from streamgages including:

- Three gages on Van Bibber Creek – Van Bibber Creek at Highway 93 and at the Sports Complex, and Ralston Creek at Carr Street, which is actually just below the confluence of Van Bibber Creek with Ralston Creek,
- Four gages on Lena Gulch – Lena Gulch at Highway 6, at Lakewood, at Nolte Pond, and at Maple Grove Reservoir, and
- Two gages on Little Dry Creek – Little Dry Creek at Westminster and at 64th Avenue.

WWE relied upon the peak annual flow data and corresponding reports provided by Water and Earth Technologies (WET) which included annual peak flow data and flagged potential data issues for each gage (see Attachment D for Van Bibber Creek, Attachment E for Lena Gulch, and Attachment F for Little Dry Creek). This dataset served as a basis for a Bulletin 17C flow frequency analysis (England et al., 2018).

The final computed flow frequency curves for each gage are contained in the following sections. The results of each flow frequency analysis are also compared to flow frequency values found in the most current FEMA Flood Insurance Study (FIS), MHFD Flood Hazard Area Delineations (FHAD) or Major Drainageway Planning (MDP) reports. Where the FIS did not report effective flows at the gage location, the flows from the FIS's relevant hydrology reference are reported (as noted in each section). Technical memoranda prepared for each watershed are attached to this report and provide additional detail on homogeneity considerations, datasets used, and additional flow frequency analyses conducted. The purpose of this report is to provide a summary of the results for each gage and WWE's recommendations as they relate to maintaining the existing, published flow frequency values in these watersheds.

2.0 GENERAL APPROACH

WWE used the United States Army Corps of Engineers' (USACE) HEC-SSP software to perform a Bulletin 17C analysis for all gages. Unless otherwise noted, a weighted skew was used which was weighted based on the individually-calculated station skew and a regional skew value of 0.05 (with a regional skew mean square error [MSE] of 0.302), consistent with the Bulletin 17B Average Skew Coefficient By One Degree Quadrangles map. Lower and upper (5% and 95%) confidence limits were also calculated. High and low outlier tests were conducted through HEC-SSP, and flagged data were evaluated on a case by case basis. Note that the annual peak flow data reported were based on the calendar year (as opposed to the water year).

Each dataset was evaluated for homogeneity considerations including urbanization in the watershed, upstream detention such as reservoirs, and the fit of the data to a log-normal distribution. In general, the data were considered homogeneous and fit a log-normal distribution, with the noteworthy exceptions of Lena Gulch at Nolte Pond and at Maple Grove Reservoir. Additional discussion for these two gages is presented in Section 4.0 and the attached Lena Gulch memorandum.

WWE handled missing years of data or data with questionable accuracy using perception thresholds in HEC-SSP on a case-by-case basis. The specifics of these decisions and their bases can be found in the attached memoranda. The subsequent sections of this report provide general commentary on final computational methods used for each gage along with the results of the analyses.

Note that another United States Geological Survey (USGS) gage, Little Dry Creek Below Federal Boulevard at Westminster (number 06719845), also exists within the Little Dry Creek watershed. However, this site has very limited data and therefore was not analyzed as a part of this effort.

3.0 VAN BIBBER CREEK RESULTS

3.1 Van Bibber Creek at Highway 93

The Van Bibber at Highway 93 gage is MHFD ALERT Gage Number 330 and has a watershed area of 9.4 mi^2 (StreamStats). It has a period of record from 1991 to the present. Due to measurement accuracy issues, the final flow frequency relationship was based on data from 1991 through 2015. A paleoflood event that likely occurred in 1948 and was estimated to have a peak flow between 2,580 cfs and 4,500 cfs was also included in the analysis. The final computed flow frequency values and confidence limits are presented in Table 1 and Figure 1.

Table 1. Van Bibber Creek at Highway 93 Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	50	31	80
5	152	92	273
10	284	162	568
20	486	255	1,121
50	906	419	2,676
100	1,391	582	5,132
200	2,079	784	9,799
500	3,422	1,118	22,994

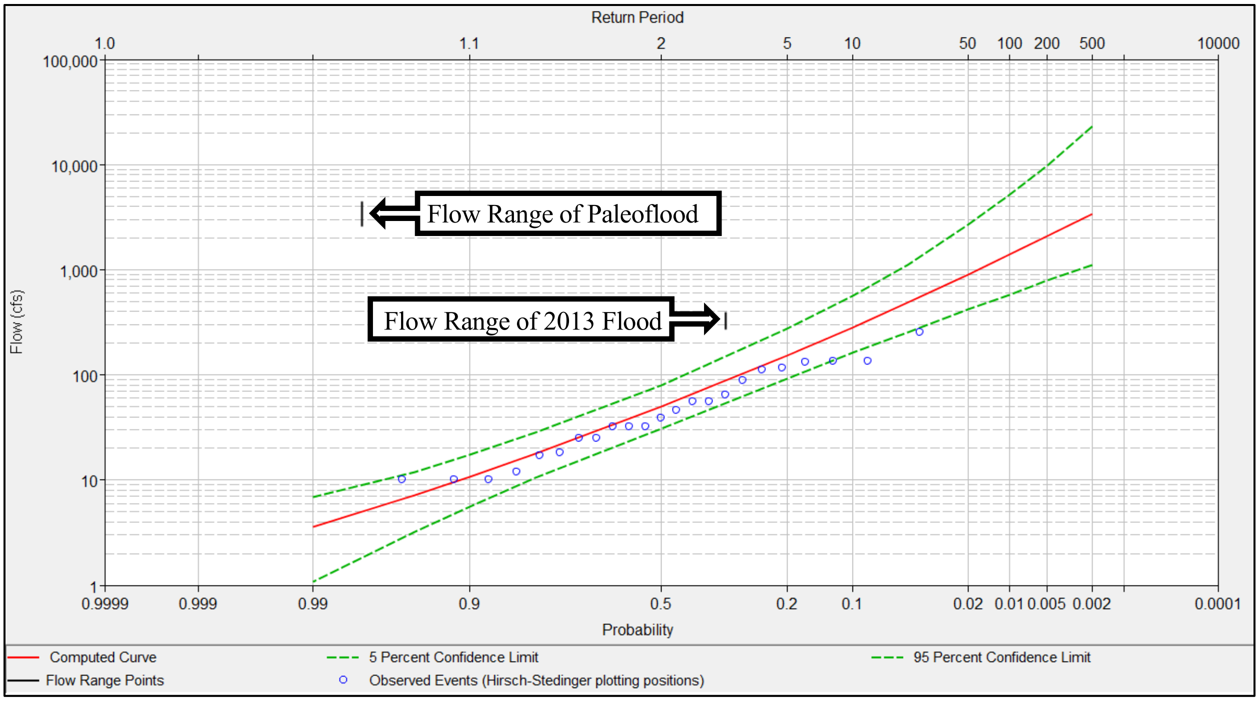


Figure 1. Van Bibber Creek at Highway 93 Flow Frequency Curve

The computed flow frequency curve was also compared with the existing published flow values as well as Olsson Associates (Olsson's) recommended peak discharges (from their February 2021 draft report), presented in Figure 2. The published flows shown are from the 1986 MDP for Van Bibber Creek at the gage location (the 1986 MDP was used as the hydrology reference for the FIS)[1]. Olsson's peak discharge values are for existing conditions at Highway 93. In general, the published values were greater than the computed curve values but were within the confidence limits for the 100-year event. Olsson's 25-, 50, 100-, and 500-year events are also within the confidence intervals, although higher than both the computed value and the published value (for the 100-year event). Olsson's 2-, 5- and 10-year peak discharges are significantly lower than the computed curve. This discontinuity between the smaller, more frequent events and the larger, less frequent events may be due to how Olsson's hydrology was calibrated for small events. While it is certainly possible that there is relatively low runoff from this more undeveloped watershed during small events, the large jump between the 10-year and 25-year events is likely a result of modeling and not reflective of real-world conditions. Based on a comparison of the published values and the computed curve and confidence limits, WWE recommends continuing to use the published flows, specifically at the 100-year level, for floodplain mapping purposes.

[1] On pg. 21 of the 2021 Jefferson County FIS, it states: "The peak discharge-frequency relationships utilized within this study for Ralston, Van Bibber, and Leyden Creeks within the City of Arvada, were obtained from the previously mentioned Major Drainageway Planning Report (Reference 37)." The 1986 MDP flows at the gage locations were used for the Van Bibber at Highway 93, Van Bibber at Sports Complex, and Ralston Creek at Carr Street.

Note that different horizontal scales are used in Figure 1 and Figure 2 (and later figures parallel to these). HEC-SSP output (which is used to generate Figure 1 and uses the red and green color scheme) uses a specialized probability scale that is unavailable in Excel.

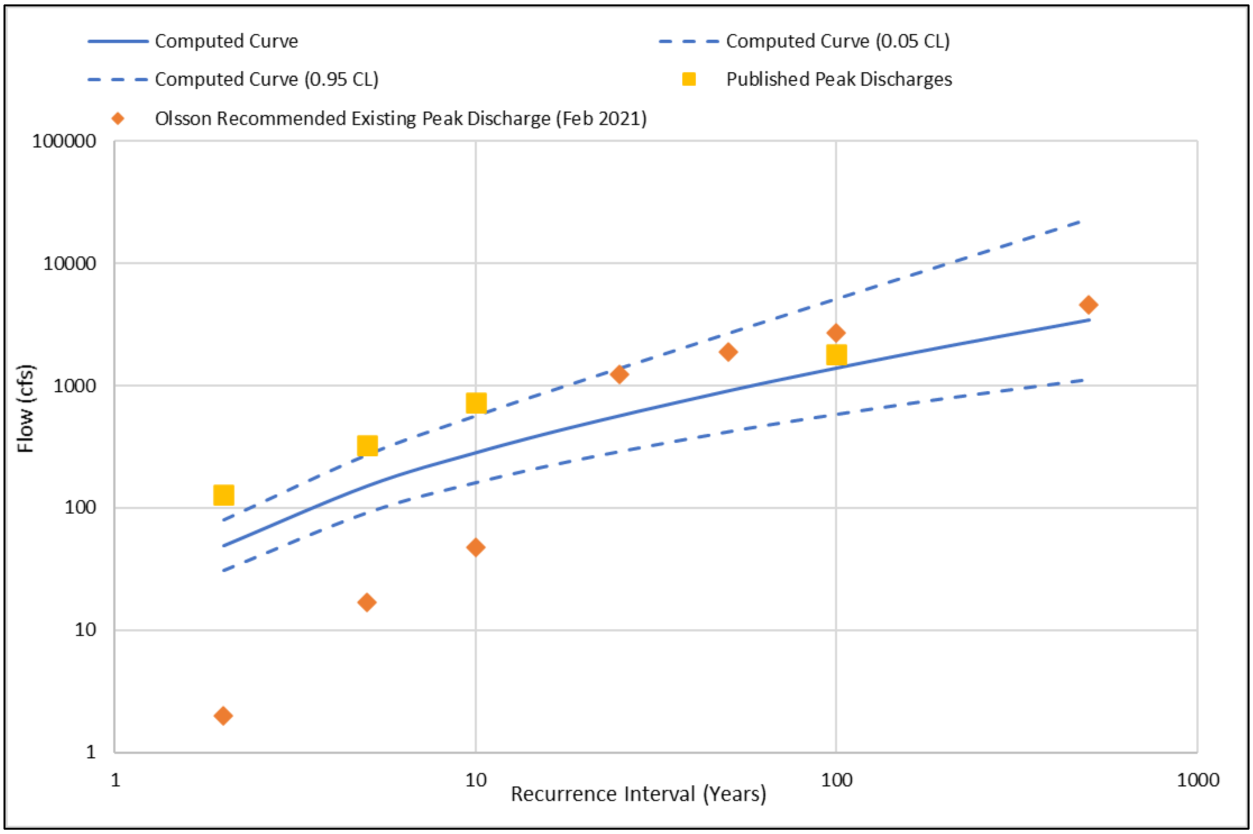


Figure 2. Van Bibber Creek at Highway 93 Comparison of Computed Frequency Curve with Published Frequency Curve and Olsson Recommended Existing Conditions Peak Discharges

3.2 Van Bibber Creek at Sports Complex

The Van Bibber at Sports Complex gage is MHFD ALERT Gage Number 320 and has a period of record from 1990 through the present. The watershed area at this gage is 17.5 mi^2 (StreamStats). No data were available for the year 2006, when the gage was moved and channel realignment construction occurred. Also, there were insufficient data to determine an annual peak for 2012. The same paleoflood peak flow value estimated at Highway 93 for 1948 was also applied at the Sports Complex gage. The exact relationship between peak flow at Highway 93 and the Sports Complex gage is unknown, and while there is increased watershed size at the latter gage, there are also potential canal interactions or backwater attenuation that could impact peak flows. For this reason, no adjustment to the peak flow estimated at Highway 93 for the paleoflood event was made. The final computed flow frequency curve and confidence limits are presented in Table 2 and Figure 3.

Table 2. Van Bibber Creek at Sports Complex Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	94	71	126
5	200	147	292
10	304	214	492
20	436	291	806
50	664	407	1,521
100	887	507	2,443
200	1,163	618	3,905
500	1,629	784	7,214

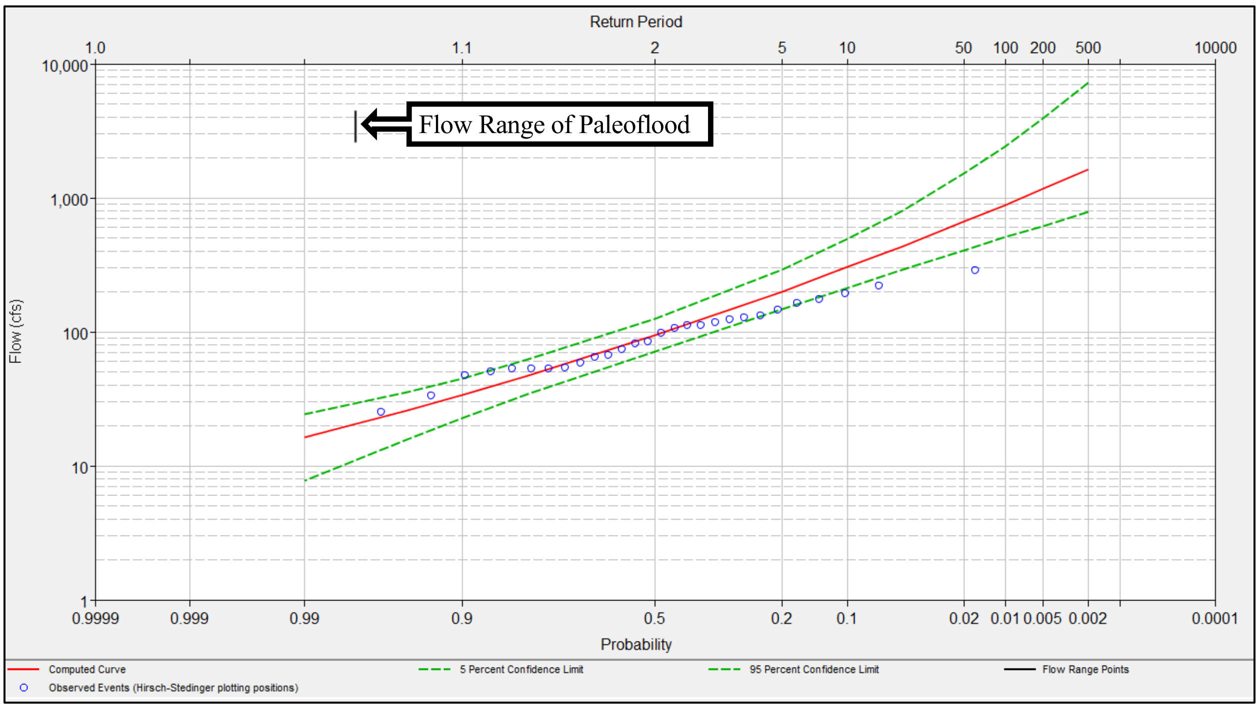


Figure 3. Van Bibber Creek at Sports Complex Flow Frequency Curve

A comparison between the computed curve and the published frequency values plus the Olsson recommended peak discharges (from their February 2021 draft report) is presented in Figure 4. The published flows presented are from the 1986 MDP for Van Bibber Creek at the gage location (see footnote 1). The peak flow values from Olsson are for current conditions at design point 105T, which is the gage location. Again, there is a discontinuity at this gage between the 10-year and the 25-year events, when the discharge jumps significantly not following the trend for the events below the 10-year. The computed flow frequency curve is significantly lower than both the published flow frequency curve and Olsson's recommended peak-discharges for events larger than the 10-year flow. Olsson's recommendation for the 2-, 5-, and 10-year events are similar to the computed curve and within the confidence intervals). One contributing factor may be the possible diversions by canals upstream of the Sports Complex, which would likely not have been included in any published modeling, but which affect the measured flow values used for the computed curve. However, these canal diversions and any other inadvertent storage are not guaranteed to remain into the future. Therefore, for the purposes of design- and floodplain mapping, the higher published values, which are likely less influenced by the current and unintentional diversions at high flows, should continue to be used. The Olsson 100-year flow is higher that the published 100-year peak discharge and is above the upper confidence limit of the flow frequency curve. The modeling by Olsson, in accordance with MHFD policies, would not have considered effects of diversions by ditches, inadvertent storage, and on site detention, so it is not surprising that the model results for the larger events are significantly higher than the flow frequency results. As noted above, ditch diversions and inadvertent storage areas cannot be guaranteed to function as they currently do into the future, and this is a primary reason that WWE does not recommend decreasing the published 100-year peak discharge.

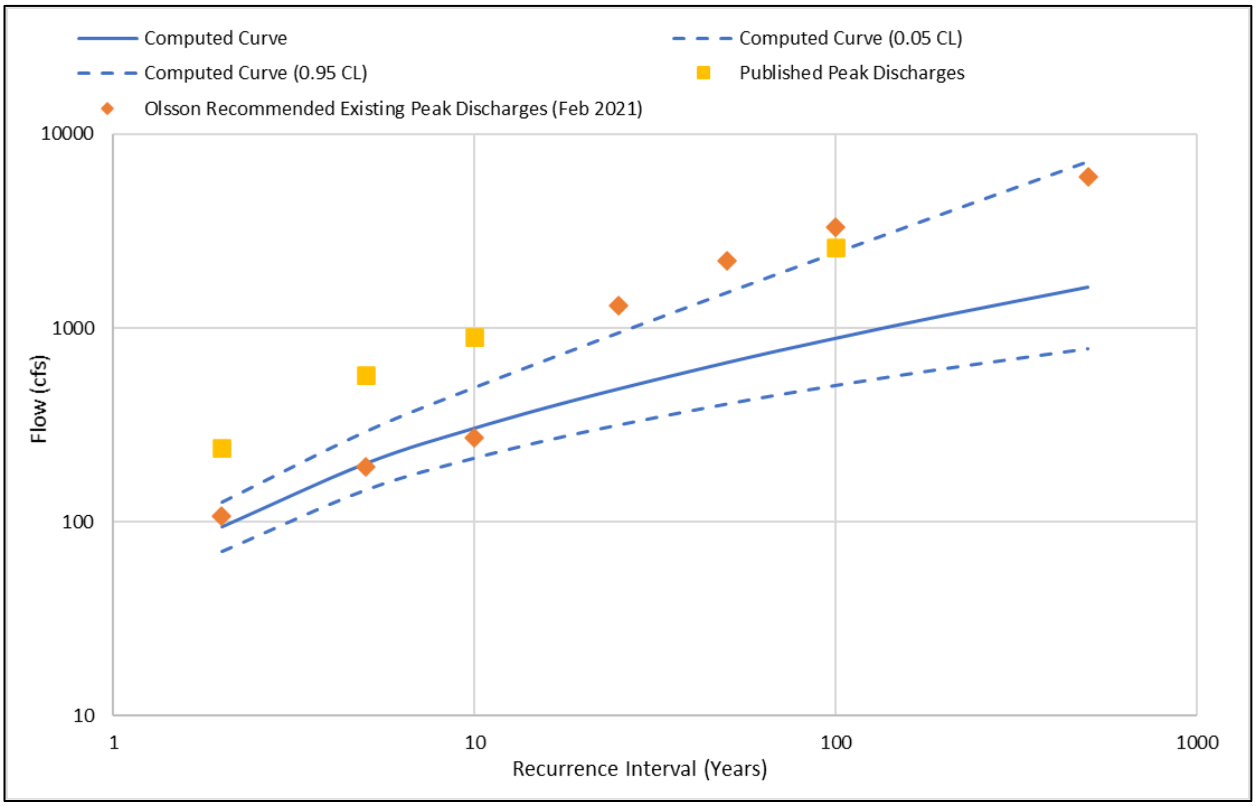


Figure 4. Van Bibber Creek at Sports Complex Comparison of Computed Frequency Curve with Published Frequency Curve and Olsson Recommended Existing Conditions Peak Discharges

3.3 Ralston Creek at Carr Street

The Ralston Creek at Carr Street gage is MHFD ALERT Gage Number 100 and is located below the confluence of Van Bibber Creek with Ralston Creek (watershed area of 89.1 mi^2, StreamStats). The period of record is from 1988 to the present, with no data for 2012. Data from 2014 through the present were based off a different gage rating than the previous years, which resulted in lower than average peak flow values. Many of these years were also identified as low outliers by HEC-SSP. For this reason, the computed curve was based on data from 1988 through 2013. The final results are presented in Table 3 and Figure 5.

Table 3. Ralston Creek at Carr Street Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	949	794	1,134
5	1,450	1,209	1,812
10	1,812	1,487	2,403
20	2,179	1,750	3,115
50	2,683	2,078	4,292
100	3,083	2,314	5,400
200	3,502	2,541	6,738
500	4,089	2,829	8,939

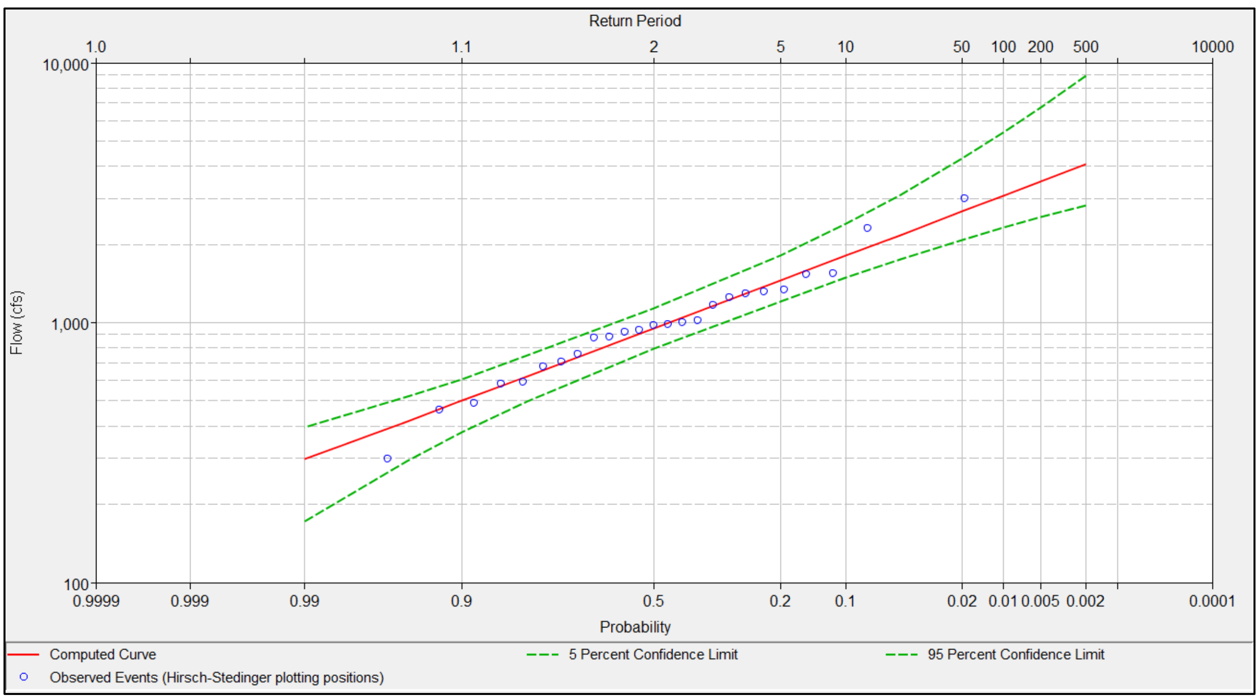


Figure 5. Ralston Creek at Carr Street Flow Frequency Curve

A comparison of the computed curve and the published flow values for Ralston Creek below the confluence with Van Bibber Creek is presented in Figure 6 (no 100-year flow value from Olsson exists at this location). The published flows are from the 1986 MDP for Ralston Creek at the gage location (see footnote 1). Again, the computed curve and confidence intervals are below the published flow values. However, from a planning and design perspective, WWE does not recommend decreasing the published values, especially for the 100-year event, due to the effects of inadvertent storage and upstream diversions that are reflected in the gage record but that may not be present in the future.

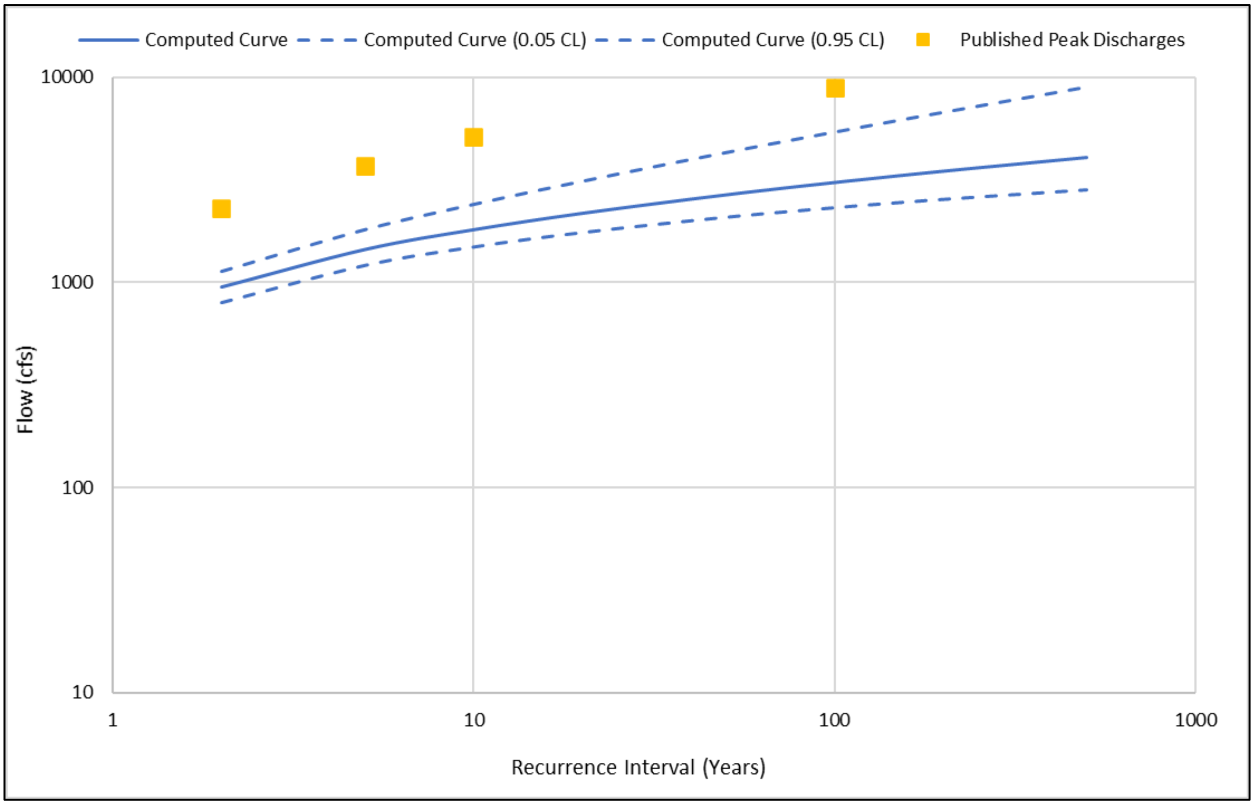


Figure 6. Ralston Creek at Carr Street Comparison of Computed Frequency Curve with Published Frequency Curve

4.0 LENA GULCH RESULTS

4.1 Lena Gulch at Highway 6

The Lena Gulch at Highway 6 gage is MHFD ALERT Gage Number 1043 and has a watershed area of 3.54 mi^2 (StreamStats). It has a period of record from 1985 through the present. However, for years prior to 1998, as well as 2007 through 2013, and 2018, the data are insufficient to determine an annual peak. This results in only 13 years of available annual peaks for analysis. The computed flow frequency curve and confidence limits are presented in Table 4 and Figure 7.

Table 4. Lena Gulch at Highway 6 Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	**0.95**
2	135	77	234
5	338	198	691
10	545	308	1,359
20	806	435	2,538
50	1,251	622	5,433
100	1,674	775	9,345
200	2,184	935	15,811
500	3,010	1,154	30,911

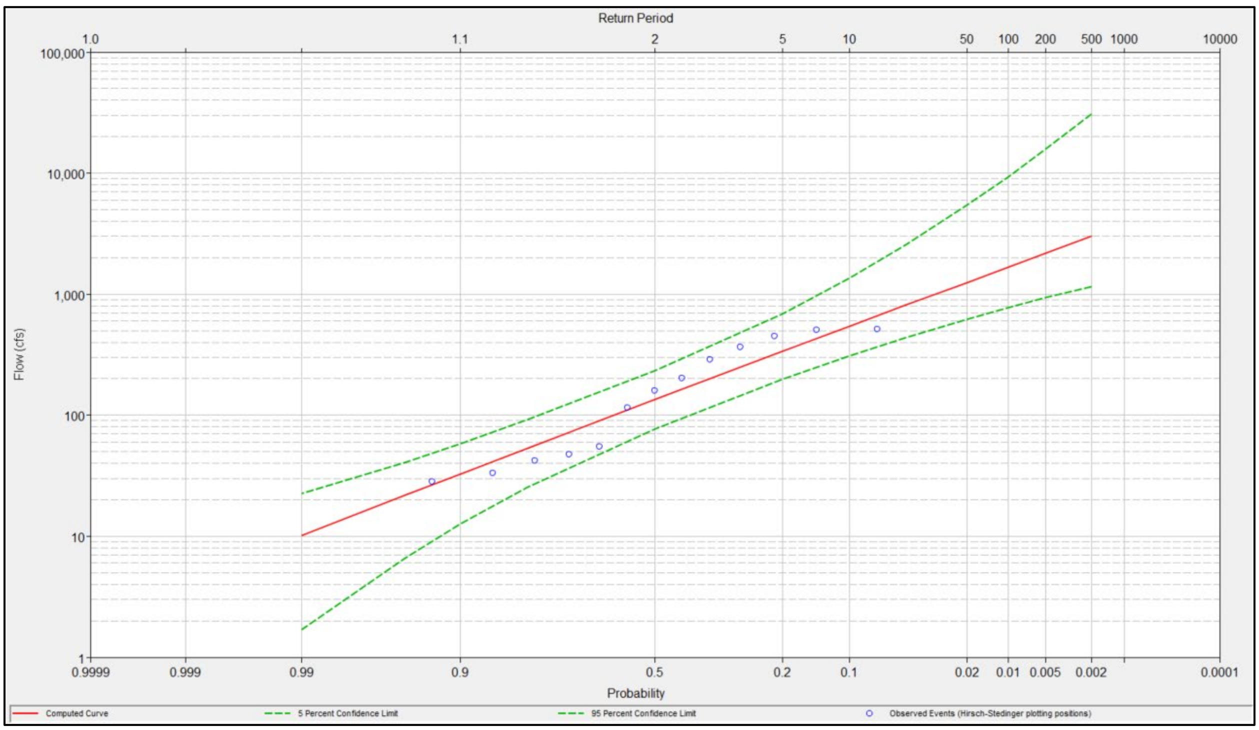


Figure 7. Lena Gulch at Highway 6 Flow Frequency Curve

The computed curve and existing 2021 FIS values (at Highway 6) agree relatively closely, especially for higher recurrence interval events (Figure 8). The FIS values all fall within the confidence limits of the flow frequency results. Future peak discharge values at Highway 6 from RESPEC's December 2020 hydrology study are also included for comparison. RESPEC's modeled values are below the computed curve, and some even fall outside the confidence limits of the computed curve. WWE does not recommend changing the FIS values. There is relative agreement between the FIS and computed results, and the inadvertent storage present in the watershed, which is not guaranteed to remain constant into the future, likely has an effect on making the calculated flows for a given frequency lower than those modeled for the FIS, which would not have taken inadvertent storage into modeling consideration. The fact that the FIS curve diverges more from the computed curve at the 10-year event may also be impacted by upstream interactions with ditches and inadvertent storage or diversions.

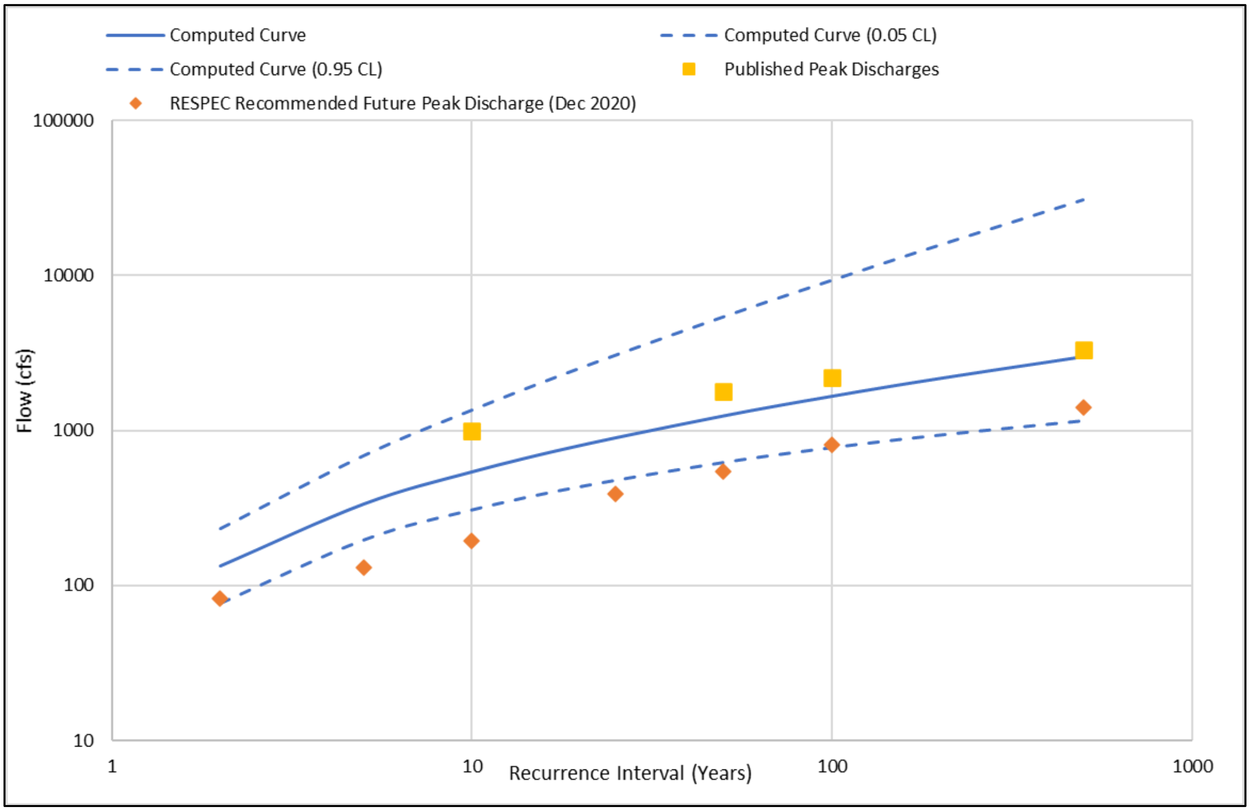


Figure 8. Lena Gulch at Highway 6 Comparison of Computed Frequency Curve with Published Frequency Curve and RESPEC Recommended Future Conditions Peak Discharges

4.2 Lena Gulch at Lakewood

Lena Gulch at Lakewood is USGS Gage Number 06719560, with a watershed area of 9.06 mi^2 (8.8 mi^2 from StreamStats). Crest Stage Indicator (CSI) peak flow data are available from 1973 through 2013, and discharge data are available from 2013 through the present. Station skew was used to compute the curve at this location given the long period of record. Four years of data (1973, 1979, 2002, and 2006) were flagged as low outliers by HEC-SSP using the Multiple Grubbs-Beck low outlier test. These data points were not included in analysis, following Bulletin 17C recommendation. The results are presented in Table 5 and Figure 9.

Table 5. Lena Gulch at Lakewood Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	0.95
2	252	210	300
5	410	345	492
10	518	434	645
20	622	516	829
50	756	610	1,133
100	856	670	1,416
200	955	723	1,755
500	1,085	782	2,315

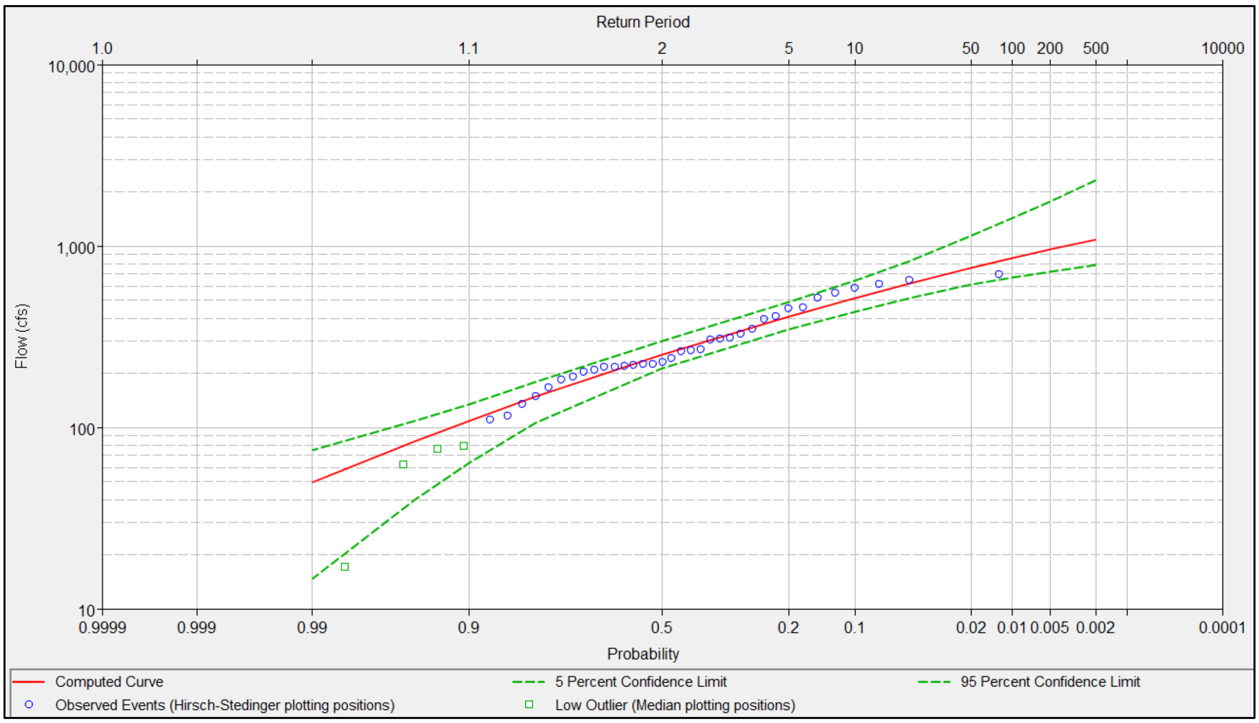


Figure 9. Lena Gulch at Lakewood Flow Frequency Curve

A comparison of the computed curve and the published flows at the approximate location of the Lakewood gage is presented in Figure 10. The 2021 Jefferson County FIS does not provide flows on Lena Gulch downstream of Highway 6. Similarly, it does not cite the hydrology reference for peak flows downstream of Highway 6. The published flows reported here are from the most recent FHAD, which was published in 1993 for Upper Lena Gulch.[2] The computed curve and confidence limits are below the published flow values. Again, it is WWE's recommendation that no changes be made to the published values. While the computed curve is well below the published curve, this may again be

[2] The published flows reported here for Lena Gulch at Lakewood and Lena Gulch at Nolte are both from the 1993 FHAD for Upper Lena Gulch (the most recent FHAD for this reach). The published flows for Lena Gulch at Maple Grove are from the 2007 FHAD for Lower Lena Gulch.

due to inadvertent storage or diversions in the watershed, which may decrease flows in the recent past and present but cannot be depended on going into the future. Future peak discharge values at Youngfield Street (approximately 0.4 miles downstream of the Lakewood gage) from RESPEC's December 2020 hydrology study are also included for comparison. RESPEC's values fall between the computed curve and the published peak discharges, although also greater than the computed confidence limits.

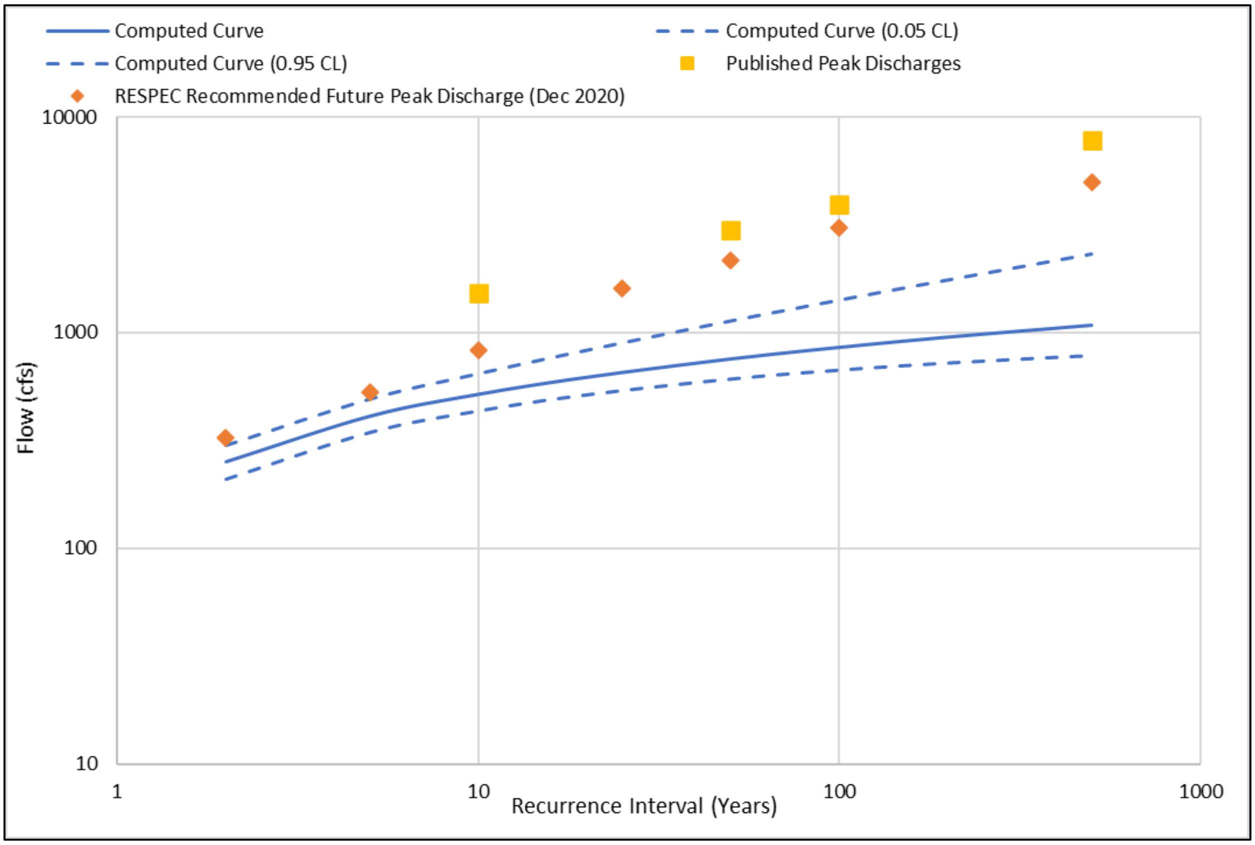


Figure 10. Lena Gulch at Lakewood Comparison of Computed Frequency Curve with Published Frequency Curve and RESPEC Recommended Future Conditions Peak Discharges

4.3 Lena Gulch at Nolte Pond

Lena Gulch at Nolte Pond is MHFD ALERT Gage Number 1023 with a watershed area of 9.45 mi^2 (StreamStats). The period of record is from 1986 to the present, with annual peaks missing for 1989, 1994, and 2014 due to insufficient data during those years. This gage presented some data quality issues, briefly summarized below. The gage measures stage in a small residential pond, the outlet of which can be controlled by a removable flashboard that allows the homeowner to change the stage in the pond by 1 to 1.5 feet. This results in two different ratings for the gage, one when the flashboard is installed and one when it is not. Unfortunately, no record is maintained of when the flashboard has historically been installed. The "without flashboard" rating was applied for the entire period, which means that there may be points in the historical record when discharges were slightly overpredicted. There is also a small pond upstream of Nolte that may provide some level of storage and regulation.

Finally, the data at Nolte Pond were found to not fit a log-normal distribution. The annual peak data was evaluated using HEC-SSP, and the results are presented in Table 6 and Figure 11. However, due to the data uncertainties and homogeneity issues discussed above, the results should not be adopted as official flow frequency values. Instead, the results are valuable for comparison with other gages in the watershed as well as a reasonableness check on the published values.

Table 6. Lena Gulch at Nolte Pond Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	0.95
2	147	118	187
5	280	217	389
10	405	302	630
20	561	399	1,008
50	827	546	1,863
100	1,084	674	2,956
200	1,401	818	4,682
500	1,934	1,035	8,579

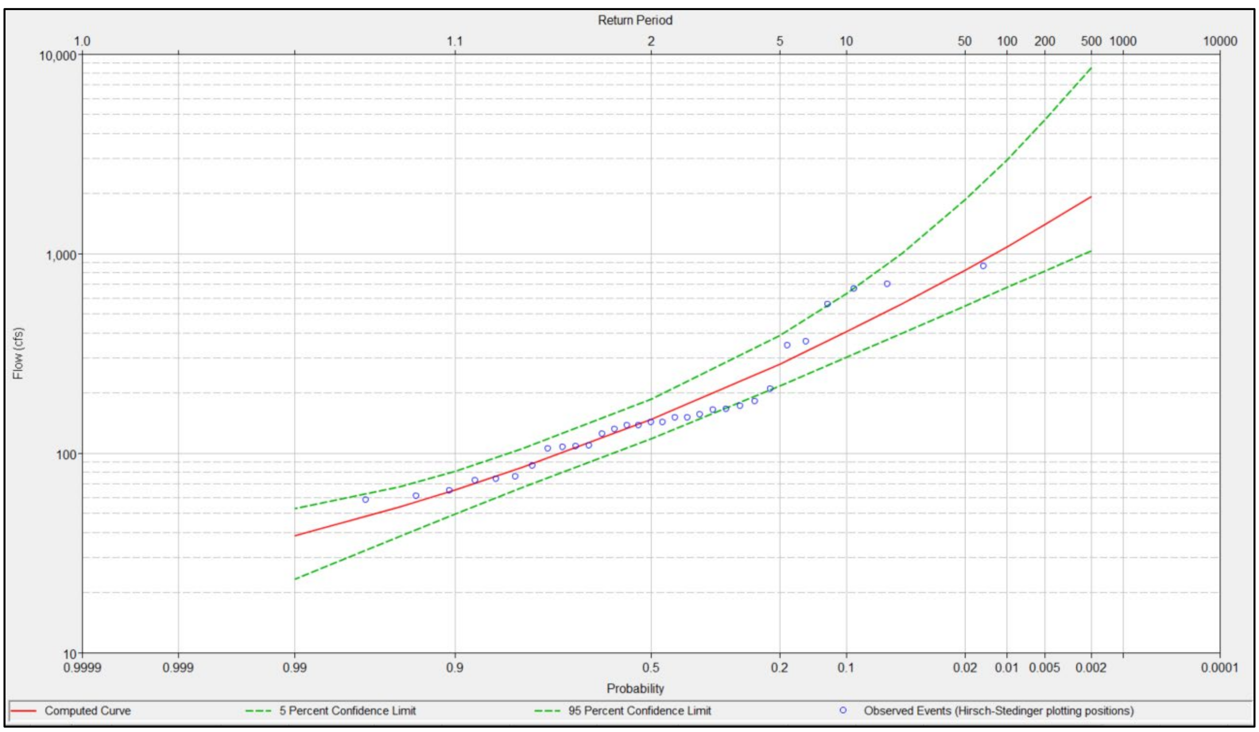


Figure 11. Lena Gulch at Nolte Pond Flow Frequency Curve

Finally, a comparison of the computed curve and the published flow frequency values (from the 1993 FHAD for Upper Lena Gulch, see footnote 2) at the approximate Nolte Pond location, are presented in Figure 12. Similar to the other gages, the computed curve is below the published flows. Only at the 500-year event is the published value within the computed confidence limits. Future peak discharge values at 20th Avenue (approximately 0.3 miles downstream of the Nolte Pond gage) from RESPEC's December 2020 hydrology study are also included for comparison. Again, RESPEC's values fall between the published peak discharges and the computed curve. Similar to the published value, only at the 500-year event is RESPEC's value within the computed confidence limit. As has been previously discussed related to inadvertent storage in the watershed, and based on data reliability questions, the computed curve does not provide any evidence that the published values should be adjusted.

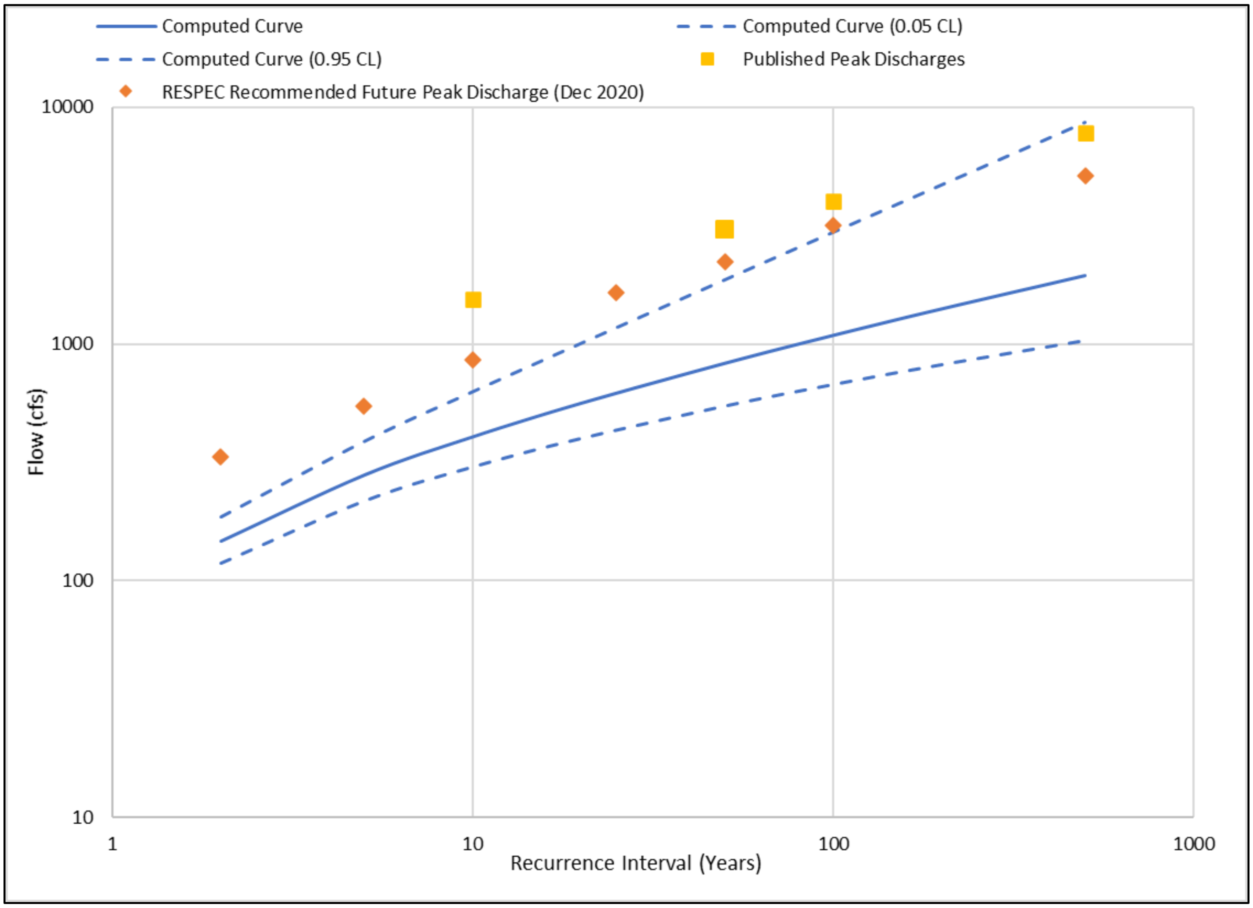


Figure 12. Lena Gulch at Nolte Pond Comparison of Computed Frequency Curve with Published Frequency Curve and RESPEC Recommended Future Conditions Peak Discharges

Depending on the intended use of the flow frequency values, it may be more appropriate to scale the results at the Lakewood Gage to downstream locations (based on watershed size), as opposed to using the results calculated at Nolte Pond. The difference in watershed area between Lena Gulch at Lakewood and Lena Gulch at Nolte Pond is less than 10%.

4.4 Lena Gulch at Maple Grove Reservoir

Lena Gulch at Maple Grover Reservoir is MHFD Alert Gage Number 1003 (drainage area of 10.5 mi^2, StreamStats) and has peak annual flows from 1987 through the present. Maple Grove Reservoir is highly influenced by regulation. WWE eliminated the years of zero flow from the dataset and then calculated the corresponding flow frequency using a station skew. The Bulletin 17C procedures were not designed for use with highly regulated datasets, and thus these results should be applied with caution. However, the fact that the emergency gates have never been lowered decreases the influence of policy and operation decisions on the computed results and makes the flow frequency results more representative of seasonal flow patterns than human decisions. The computed flow frequency curve and confidence limits are presented in Table 7 and Figure 13. Again, depending on the intended use of the calculated results, it may be better to use the results at the Lakewood gage and scale it to points just upstream of Maple Grove Reservoir based on watershed area. The difference in watershed area between these two points is less than 20%, and if the Lakewood gage results are scaled that will avoid the error associated with regulation at Maple Grove Reservoir.

Table 7. Lena Gulch at Maple Grove Reservoir Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	0.95
2	22	14	33
5	61	41	92
10	98	66	161
20	142	94	266
50	209	131	494
100	267	157	772
200	329	181	1,191
500	418	211	2,095

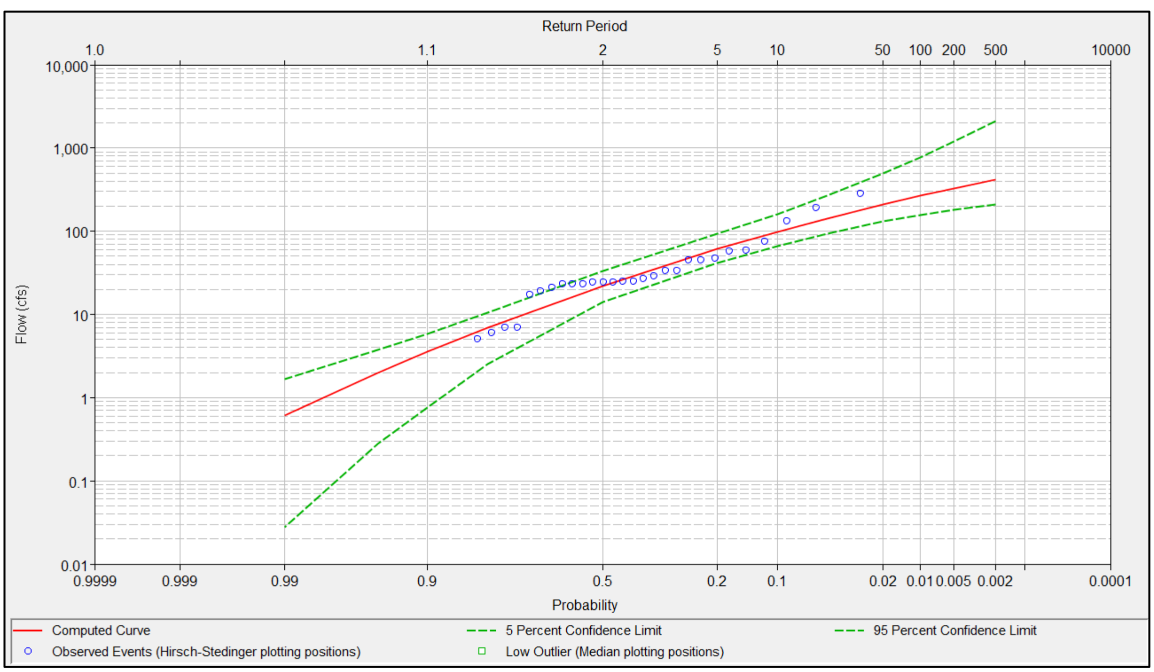


Figure 13. Lena Gulch at Maple Grove Reservoir Flow Frequency Curve

A comparison of the computed curve and the published flows (from the 2007 FHAD for Lower Lena Gulch, see footnote 2) is presented in Figure 14. For all flows above the 2-year event, the computed curve is lower than the published curve. Note that the 2-year flow from the MDP is only 1 cfs. The difference between the computed curve and the MDP flows is also of greater magnitude for the higher-recurrence interval events. It should be noted that the MDP assumed that the initial reservoir water surface elevation was the top of the conservation pool (elevation 5525.0 feet, volume is approximately 1,070 acre-feet). The Bulletin 17C analysis did not account for a constant starting reservoir pool but used the measured peak outflows, regardless of reservoir capacity at the start of the event. Future peak discharge values at the outlet of Maple Grove Reservoir from RESPEC's December 2020 hydrology study are also included for comparison. RESPEC's computed values are greater than the computed curve and confidence interval for all recurrence intervals, and greater than the published peak discharges at all recurrence intervals besides the 500-year event. Again, the computed curve and available data do not provide enough evidence for WWE to recommend changing the effective flows.

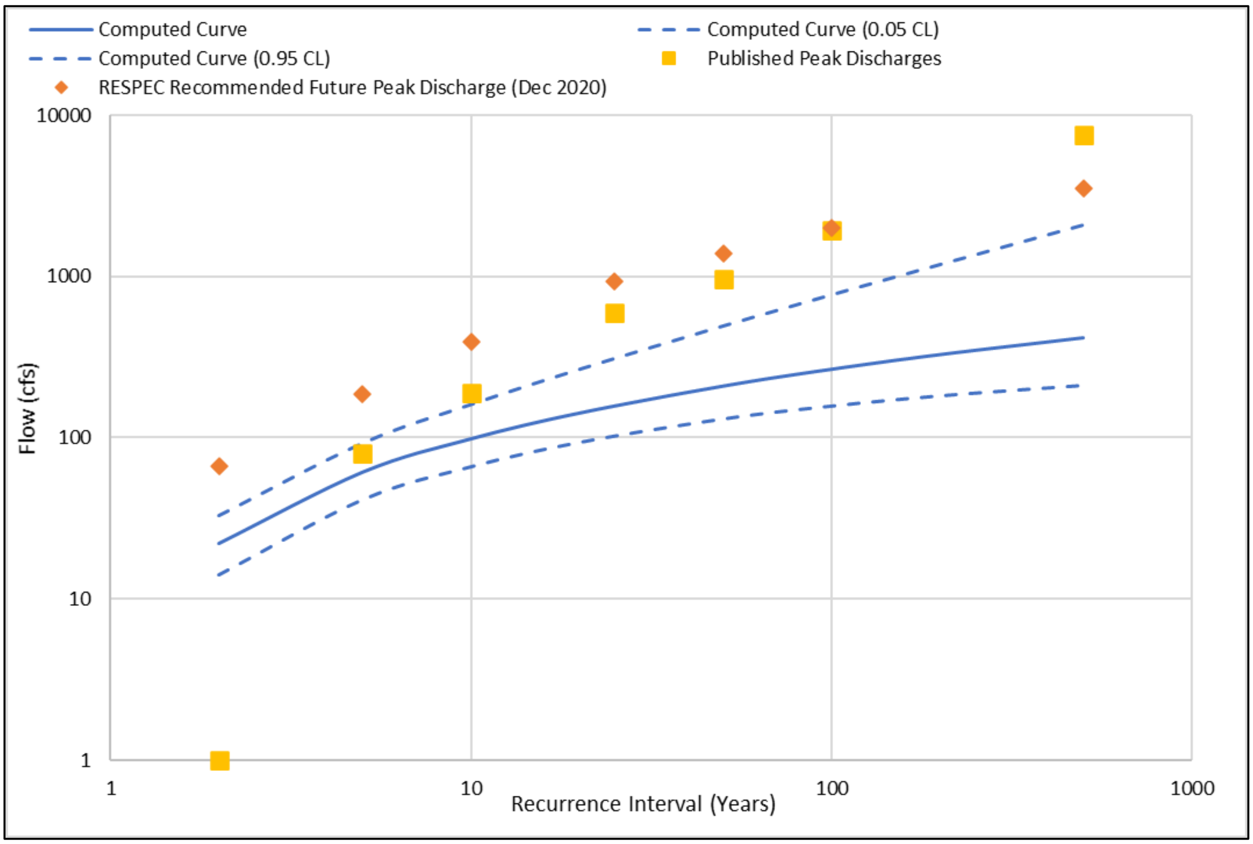


Figure 14. Lena Gulch at Maple Grove Reservoir Comparison of Computed Frequency Curve with Published Frequency Curve and RESPEC Recommended Future Conditions Peak Discharges

5.0 LITTLE DRY CREEK RESULTS

5.1 Little Dry Creek at Westminster

Little Dry Creek at Westminster is a USGS streamgage, number 06719840, and has a drainage area of 10.5 mi^2 (StreamStats). The Little Dry Creek at Westminster gage had 36 years of approved data for analysis, so a station skew was used to compute the flow frequency curve. The final results are summarized in Table 8 and the flow frequency curve and confidence limits shown in Figure 15.

Table 8. Little Dry Creek at Westminster Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	512	443	590
5	751	653	873
10	907	786	1,098
20	1,054	903	1,352
50	1,241	1,031	1,723
100	1,379	1,109	2,035
200	1,515	1,175	2,381
500	1,693	1,248	2,904

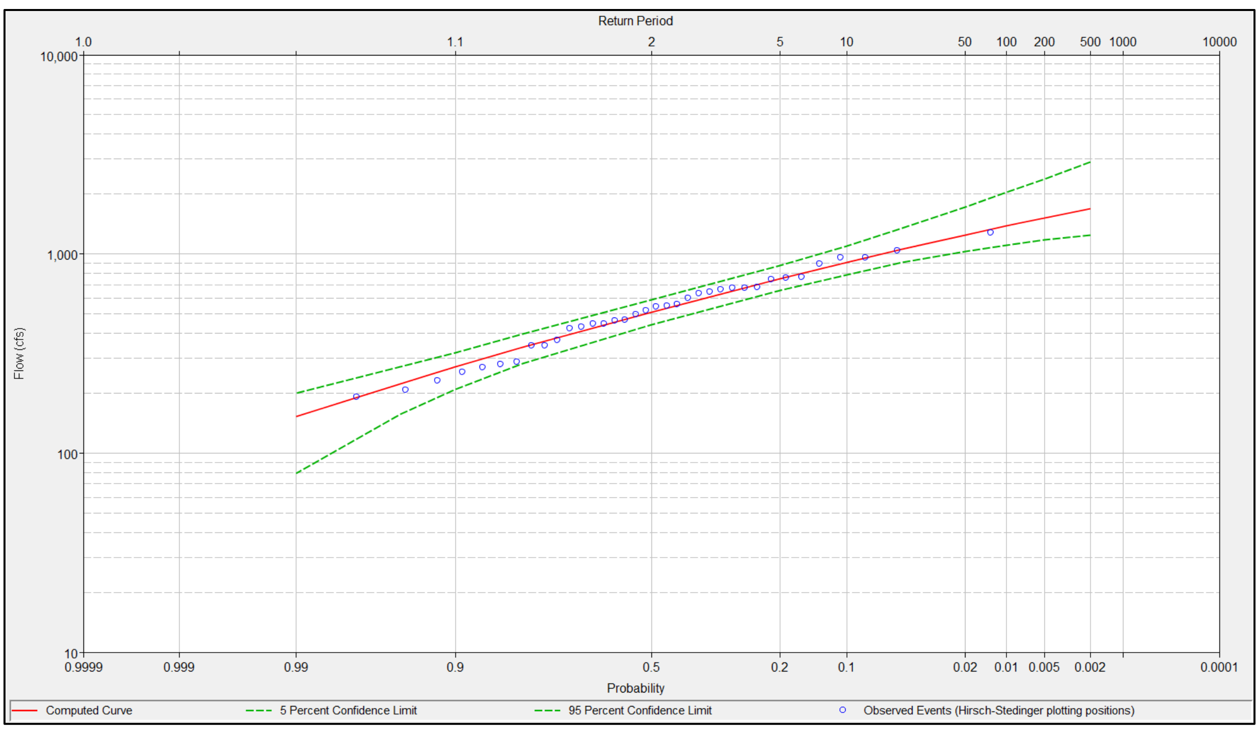


Figure 15. Little Dry Creek at Westminster Flow Frequency Curve

A comparison between the computed curve and the published peak discharges is presented in Figure 16. The published flows are from the 1978 Little Dry Creek FHAD.[3] The published flow values are all greater than both the computed curve and the confidence limits. WWE does not recommend changing the published values, for reasons discussed previously related to possible inadvertent storage and diversions. Future peak discharge values at West 72nd Avenue from Olsson's January 2020 hydrology study are also included for comparison. For all recurrence intervals, Olsson's values are greater than both the computed curve and the computed confidence intervals. For the 2-, 5-, and 10-year events, Olsson's modeled values are lower than the published peak discharges, whereas for the 25-, 50-, and 100-year events Olsson's modeled values are greater than the published peak discharges.

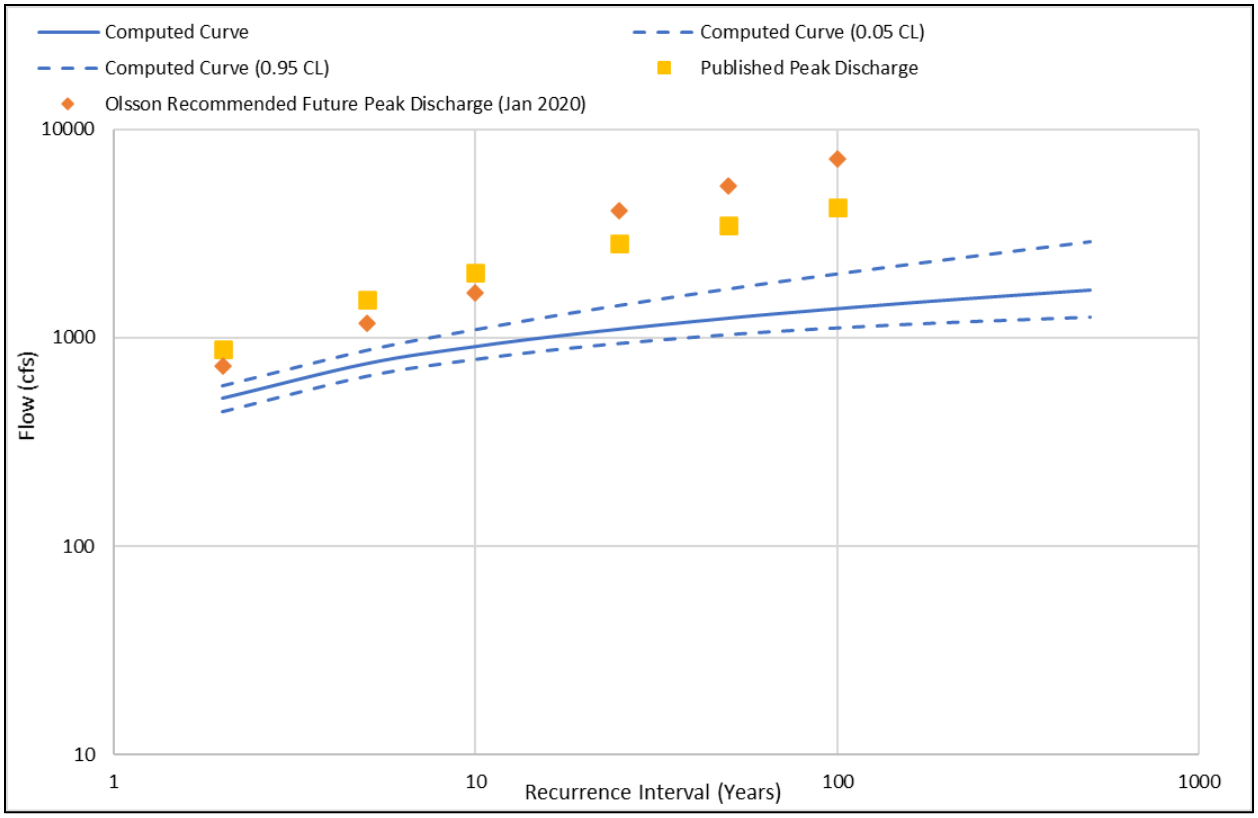


Figure 16. Little Dry Creek at Westminster Comparison of Computed Frequency Curve with Published Frequency Curve and Olsson Recommended Future Conditions Peak Discharges

5.2 Little Dry Creek at 64th Avenue

Little Dry Creek at 64th Avenue is MHFD ALERT Gage Number 1310 and has a continuous record from 2002 to 2019, or 18 years of data. However, the data collected at this gage represent only the portion of high flows that were diverted off of Little Dry Creek at a point approximately 0.3 miles upstream of the gage. Therefore, the results calculated using Bulletin 17C do not represent the actual

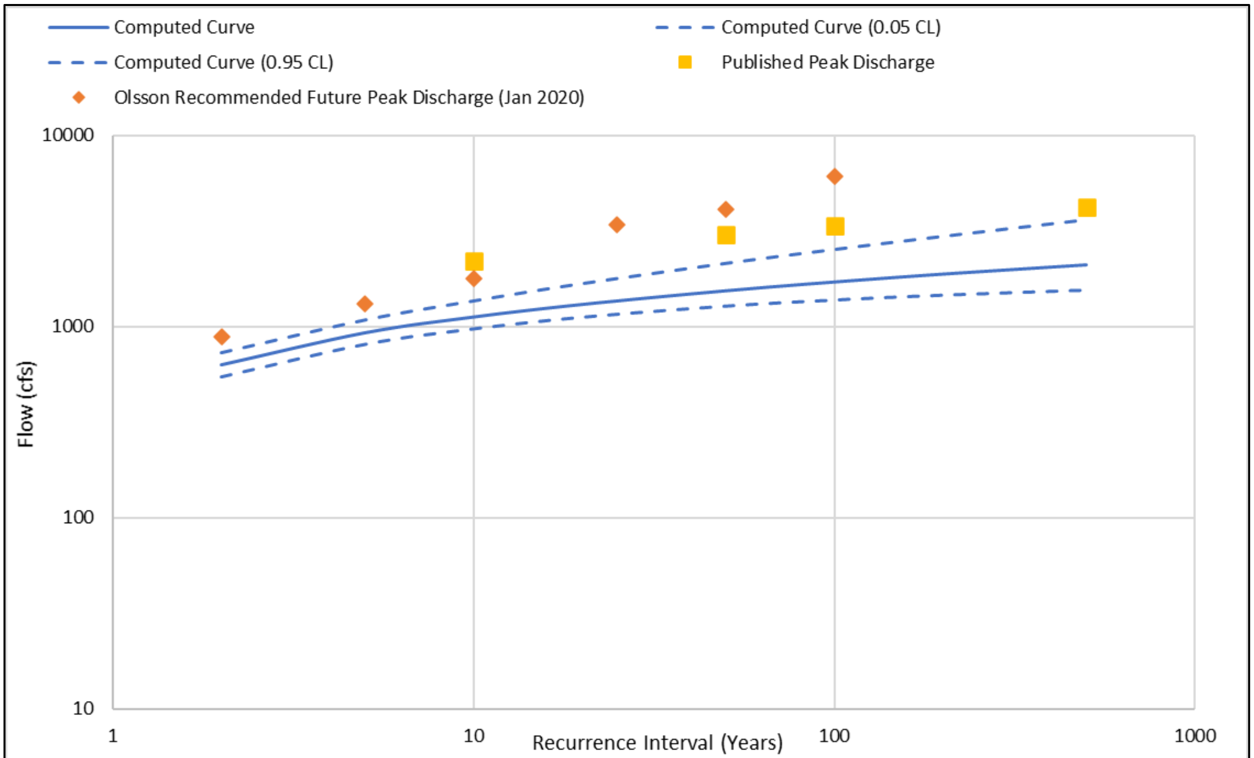
[3] On page 21 of the 2021 Jefferson County FIS, it states that "peak flows associated with Little Dry Creek and its tributaries can be obtained from the report entitled 'Flood Hazard Area Delineation, Little Dry Creek,' (Reference 49)." For this reason, the reported published flows are from the 1978 Little Dry Creek FHAD.

flow frequency on the mainstem of Little Dry Creek. For this reason, the results calculated at the Westminster gage were scaled based on watershed size to the diversion point on Little Dry Creek (watershed area of 13 mi^2, StreamStats). The results of this calculation are presented in Table 9. These results likely present a more accurate representation of flow frequency on the lower section of Little Dry Creek than do those calculated by Bulletin 17C due to the diversion.

Table 9. Little Dry Creek at 64th Avenue (Just Above Diversion) Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	634	548	731
5	929	809	1,081
10	1,123	973	1,360
20	1,305	1,118	1,673
50	1,536	1,276	2,133
100	1,707	1,373	2,519
200	1,875	1,455	2,948
500	2,095	1,546	3,596

A comparison of the above flow frequency values and the published flow frequency values at the approximate gage location is presented in Figure 17. The published peak discharge values presented are for Little Dry Creek at Confluence with Clear Creek, which is located approximately 0.4 miles downstream of the diversion point. At the confluence with Clear Creek, there is a watershed area of 13.1 mi^2 (compared to the watershed area at the diversion of 13 mi^2). Similar to the Westminster gage, the computed values scaled to the diversion point just above the 64th Avenue gage are below the published values. This may be caused by inadvertent storage or ditch diversions. Future peak discharge values at West 64th Avenue Detention Pond from Olsson's January 2020 hydrology study are also included for comparison. Olsson's values are greater than both the computed curve and confidence interval. It is WWE's recommendation that the published values not be adjusted.



Note: The computed curve shown is based on the flow frequency calculated at the upstream Westminster gage scaled by watershed area (as opposed to the Bulletin 17C results calculated at the gage, where are affected by a diversion).

Figure 17. Little Dry Creek at 64th Avenue (Just Above Diversion) Comparison of Computed Frequency Curve with Published Frequency Curve and Olsson Recommended Future Conditions Peak Discharges

6.0 CONCLUSION

The computed curves using gage data and Bulletin 17C procedures were almost always less than the existing flow frequency values. From a public safety standpoint, the existing values should not be decreased to match flows calculated from gage data, which are likely affected by inadvertent storage and diversions that cannot be depended on into the future. While WWE does not recommend decreasing the published flow values, there is also no evidence to suggest that the published flow values need to be increased at any of the analyzed points in the watershed. The gage analysis provides a useful “reality check” on the magnitude and frequency of flood flows that have occurred on these streams, in a context of statistical uncertainty, and indicates that flows used for purposes of regulating floodplains are reasonably conservative and protective of public safety. Ultimately, WWE’s analysis does not suggest that existing flow frequency values should be amended. However, additional analysis including model calibration efforts may provide supportive evidence for modifying existing flow frequency values and should be considered in the decision-making and policy formulation process.

Finally, it should be noted that as development in the watersheds continue, and interactions between watershed and streams change, it may be necessary to update these calculations accordingly. The results presented here represent calculations based on the best available data and may change as more years of data and additional gage information are collected. Continued implementation of stormwater management practices that promote runoff reduction and full spectrum detention will help to reduce the extent of changes in these flood flow frequency relationships over time.

Attachment A. Flow Frequency Memorandum for Van Bibber Creek

WWE
MEMORANDUM

To: Kevin Stewart, P.E.
Mile High Flood District

From: Wright Water Engineers, Inc.
Andrew Earles, Ph.D., P.E., D.WRE, and Haley Rogers

Date: December 16, 2020

Re: Flow Frequency Analysis for Van Bibber Creek

1.0 INTRODUCTION

WWE conducted a flow frequency analysis for two gages located on Van Bibber Creek (at Highway 93 and Sports Complex) as well as a downstream gage on Ralston Creek at Carr Street (below the confluence of Van Bibber Creek with Ralston Creek). WWE relied upon the peak annual flow data and corresponding reports provided by Water and Earth Technologies (WET) which included annual peak flow data and flagged potential data issues for each gage. This data set served as a basis for a Bulletin 17C flow frequency analysis (England et al., 2018).

For all three gages, the USACE's HEC-SSP software was used to perform a Bulletin 17C analysis. All analyzed gages had between 20 and 40 years of annual peak flow data, so a weighted skew was used for each analysis. The weighted skew was based upon the individually calculated station skew and a regional skew of 0.05 (with a regional skew mean square error [MSE] of 0.302), consistent with the Bulletin 17B Average Skew Coefficient By One Degree Quadrangles map. 5% and 95% confidence limits were also calculated. High and low outlier tests were conducted through HEC-SSP, and flagged data were evaluated on a case by case basis. Note that the annual peak flow data reported were based on the calendar year (as opposed to the water year). Individual results and analysis methods are presented for each gage in subsequent sections. The results are also compared to the most current FIS values and Olsson's modeled 100-year event and the implications discussed at the end of each section.

2.0 HOMOGENEITY CONSIDERATIONS

An underlying assumption of the Bulletin 17C analysis method is that the data analyzed are homogeneous. Before data were analyzed in HEC-SSP, it was necessary to confirm that the data were homogeneous, and therefore, it was appropriate to apply Bulletin 17C. Below are the major relevant considerations in evaluating each dataset:

- The earliest data analyzed as a part of the systematic record for the gages evaluated is from 1988. By this time, stormwater detention for peak flow attenuation was a requirement throughout the metropolitan Denver area. While the watersheds upstream of the gages evaluated undoubtedly have experienced increases in impervious area associated with new development and redevelopment over the periods analyzed, the widespread implementation of detention would have the result of diminishing the effects of changes to impervious area over time, resulting in data sets that would likely not be expected to violate the homogeneity assumptions of Bulletin 17C analysis.

- WWE performed statistical tests of each of the data sets to determine if they conform to the underlying log-normal statistical distribution used by Bulletin 17C. WWE applied the Shapiro-Wilk, Anderson-Darling, Lilliefors, and Jarque-Bera normality tests on log-transformed data. All four tests showed that the log-transformed datasets for Van Bibber Creek at Highway 93, Van Bibber Creek at Sports Complex, and Ralston Creek at Carr Street were normally distributed.

- On Ralston Creek, an issue related to homogeneity is the presence of storage in the watershed upstream of the gage, including the Ralston Reservoir, Arvada/Blunn Reservoir, and Leyden Lake/Reservoir. It is notable that the analysis for Ralston Creek uses data exclusively from the post-reservoir period, so the analysis avoids issues of homogeneity related to data from before and after reservoir construction. In addition, the statistical normality tests show that the Ralston Creek at Carr Street dataset follows a log-normal distribution.

- Finally, each dataset was evaluated for a mixed population of events caused by rainfall versus snowmelt driven hydrologic processes. The vast majority of events occurred between May and August (inclusive). Snowmelt events may have partially contributed to the events occurring in May, however across the three gages, the majority of the events occurred later in the summer when they were more likely due to rainfall events. Also, the maximum basin elevation for the two Van Bibber gages was 9,740 feet, and the maximum elevation for the Ralston at Carr St. gage was 10,500 feet with less than 50% of the basin above 7,500 ft. These metrics also indicate that there is likely less snowmelt impact and most events were rainfall driven. Finally, while a full meteorological analysis for each year was not conducted, as previously discussed, the datasets were determined to be log-normally distributed, and thus adjustments for mixed population were not needed.

Based on the above discussion, the data were determined to be appropriately homogeneous for use in accordance with the Bulletin 17C criteria. The individual analyses conducted for each gage are presented in the sections below.

3.0 VAN BIBBER CREEK AT HIGHWAY 93

The Van Bibber at Highway 93 gage is Mile High Flood District (MHFD) ALERT Gage Number 330 and has a period of record from 1991 to the present. It has a watershed area of 9.4 mi^2. A few data limitations exist for this gage. No data is available for the year 2006, so a perception threshold of 394 cfs to infinity (which translates to a potential flow range for that year between 0 and 394 cfs) was used for that year in HEC-SSP. 394 cfs was chosen as the upper bound of possible flow range for 2006 because that is the highest recorded historical flow at the Van Bibber at Highway 93 gage, and there is no evidence to indicate that 2006 exceeded the flow of record.

2013 was the flood of record at the Highway 93 gage. WET estimated that flows ranged between 272 and 394 cfs, so those values were used as the lower and upper flow bounds for that year in HEC-SSP. WET noted that the 2013 flood as well as construction on the culvert may have impacted the accuracy of gage measurements post-2013. In addition, in August of 2015, the stop bolt and pressure transducer (PT) height were changed which presented more questions about the reliability of the peak data for 2016 through the present. Ultimately, based on evaluation by WET and rainfall data and storm descriptions associated with the 2014 and 2015 events, data through 2015 were included in a second analysis, with the years after the PT was moved (2016–present) excluded. A comparison of the results for analyzing 1991 through present versus 1991 through 2015 are presented in Table 1 and demonstrate the effect that inclusion of data from 2016 through the present has on increasing the flows associated with each recurrence interval. Given the effect of including the data from 2016 through present and WET's assessment that these data may be inaccurately high, years from 2016 through the present were not used to calculate the final flow frequency curve, which still left 24 years of reliable data.

Table 1. Van Bibber Creek at Highway 93 Flow Frequency Values Using Data from 1991 Through Present versus 1991 Through 2015

Recurrence Interval (years)	Flow (cfs) [1991-Present]	Flow (cfs) [1991-2015]
2	54	45
5	132	107
10	211	169
20	311	248
50	481	384
100	643	515
200	839	675
500	1,159	938

Finally, a paleoflood peak value was also available, based on research conducted for a Master's Thesis by Natalie Trivino, titled *Paleoflood Hydrology and Basin Morphometric Characteristics Related to Flooding in the Colorado Front Range*. A paleoflood analysis based on paleoflood surveys estimated a maximum historic flow ranging between 2,280 cfs and 3,980 cfs, which Trivino proposed to be associated with a 1948 storm and flood event. However, this estimation corresponds to the range of estimated flows at six different cross-sections, all of which are approximately 1.5 miles upstream from the current gage location. The range of flows from the paleoflood location was scaled to the current gage location based on a comparison of the watershed areas. Based on a USGS StreamStats watershed delineation at the Highway 93 gage, the watershed area is 9.4 square miles. Trivino lists the watershed area for her study of Van Bibber Creek as 8.3 square miles (21.5 square kilometers). Therefore, the gage site has a watershed area of 113% of the paleo-watershed area. Applying this same ratio to the peak historical flow range estimated at the paleo cross sections gives a range of flows at the gage of 2,580 cfs to 4,500 cfs. All years between 1948 and 1990 (when the gage record begins) were assigned a perception threshold of 4,500 cfs to infinity, indicating that if flows had exceeded 4,500 cfs during this time period they would have been recorded and thus all years must have a flow less than or equal to 4,500 cfs. For 1948, the flow thresholds were set as 2,580 cfs to 4,500 cfs to represent that flows of record occurred this year. Based on historic descriptions of flooding in Golden, it is also possible that this historic peak occurred in 1896 instead. However, for the HEC-SSP analysis, the paleoflood was included as a 1948 event, consistent with Trivino's interpretations. 1948 was a significant flood year for adjacent watersheds, most notably Tucker Gulch. The 1948 flood in Tucker Gulch was well-documented in government records, and Trivino notes the 1948 storm peaked near the divide between Van Bibber Creek and Tucker Gulch. The peak discharge for Tucker Gulch during this event was estimated by Trivino to be over 9,000 cfs. A comparison of flow frequency results when the paleoflood information was and was not included in the HEC-SSP analysis is presented in Table 2. Inclusion of a perception threshold from 1948 to 1990 as well as the flow bounds for 1948 dramatically increased the high end flow frequency values.

Table 2. Van Bibber Creek at Highway 93 Flow Frequency Values Calculated Excluding and Including Paleoflood Information

Recurrence Interval (years)	Flow (cfs) [1991-2015, no paleoflood]	Flow (cfs) [1991-2015, with paleoflood]
2	45	50
5	107	152
10	169	284
20	248	486
50	384	906
100	515	1,391
200	675	2,079
500	938	3,422

While the paleoflood information had a significant effect on the upper end of the calculated flow frequency curve, the analysis is based on sound data and should incorporate as much available information as possible. Including data that represents such a major flood event occurring in the 20th century provides important context for evaluating flow frequency at the Highway 93 gage. The final flow frequency curve was based on the data set that excluded the potentially inaccurate 2016 through present data but did include the paleoflood information. The final flow frequency values and confidence limits are presented in Table 3, and the final flow frequency curve is shown in Figure 1.

Table 3. Van Bibber Creek at Highway 93 Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	50	31	80
5	152	92	273
10	284	162	568
20	486	255	1,121
50	906	419	2,676
100	1,391	582	5,132
200	2,079	784	9,799
500	3,422	1,118	22,994

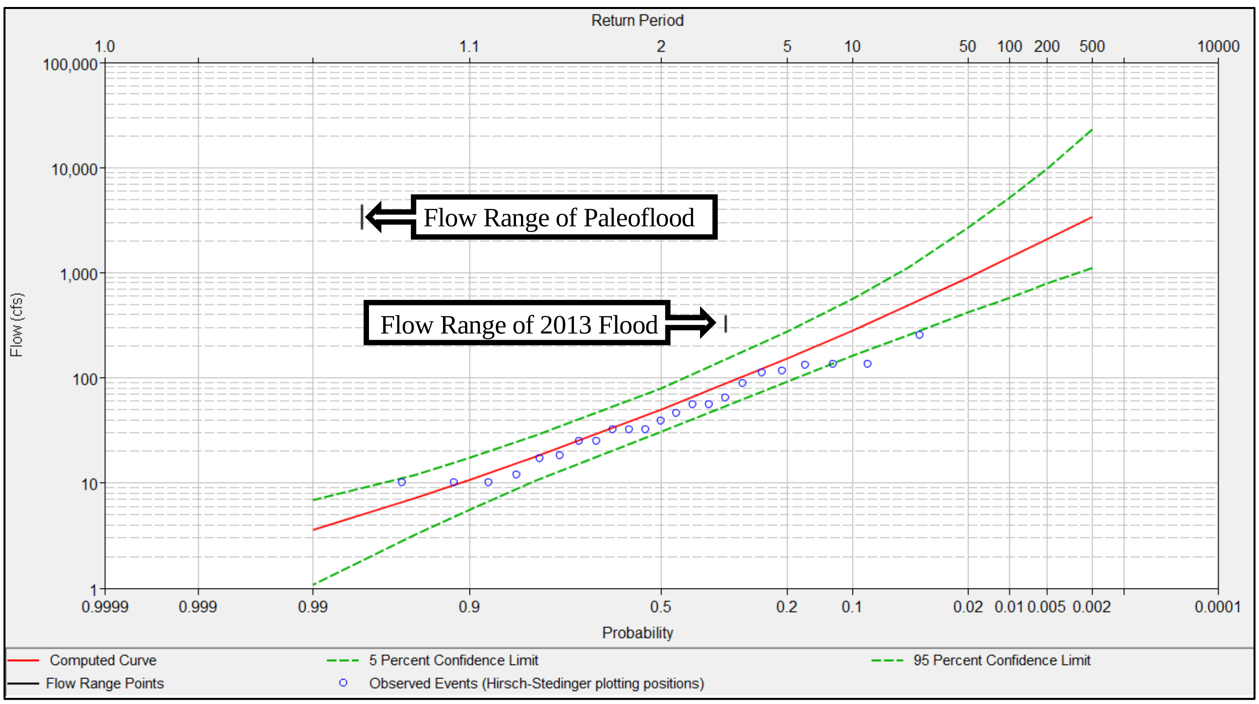


Figure 1. Van Bibber Creek at Highway 93 Flow Frequency Curve

On the upper end of the curve, two of the points plot below the confidence limits, and a third point plots on the confidence limit. This is due to the influence of the paleoflood event on the adopted skew. The regional skew value is 0.050, the calculated station skew is 0.846, and the adopted, weighted skew was 0.303. The paleoflood event causes the curve to be steeper at the upper end, and thus the recorded events are below the confidence intervals.

A comparison of the final results and currently existing FIS values (2019) as well as Olsson's recommended value for the 100-year event based on existing conditions is presented in Figure 2. The FIS and Olsson values for Van Bibber at the Hogback were scaled based on watershed area to the gage location. The computed curve is relatively similar to the FIS curve at higher recurrence interval events and diverges for the lower recurrence interval events. Olsson's 100-year event is significantly higher than both the FIS and computed curve values, and is approximately equal to the 5% confidence interval of the computer curve (in other words, based on the Bulletin 17C analysis, there is a 95% chance that the 100-year flow is below the value proposed by Olsson). Based on a comparison of the FIS values and the computed curve and confidence limits, WWE recommends that no changes to the FIS, specifically at the 100-year level for floodplain mapping purposes, be made.

Note that different horizontal scales are used in Figure 1 and Figure 2 (and later figures parallel to these). HEC-SSP output (which is used to generate Figure 1 and uses the red and green color scheme) uses a specialized probability scale that is unavailable in Excel.

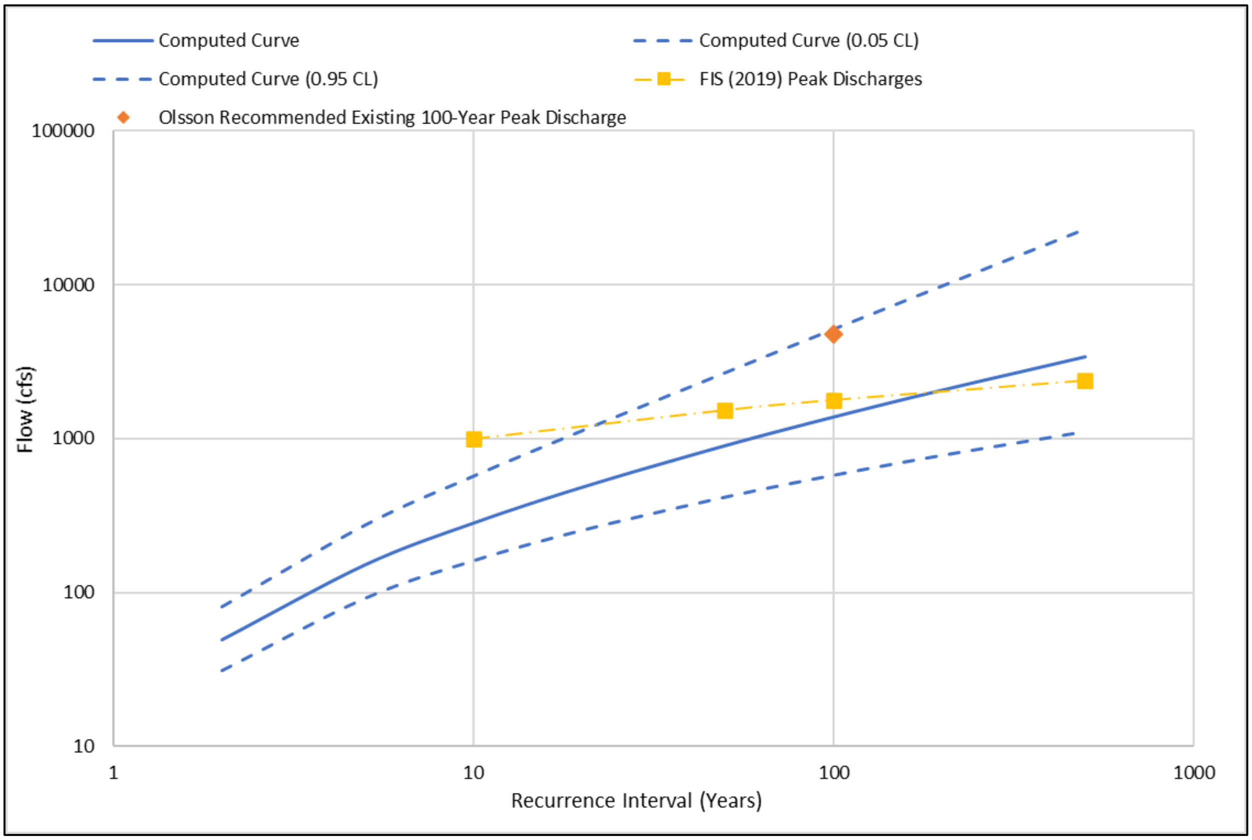


Figure 2. Van Bibber Creek at Highway 93 Comparison of Computed Frequency Curve with Existing FIS and Olsson Recommended Existing Conditions 100-yr Flow

4.0 VAN BIBBER CREEK AT SPORTS COMPLEX

The Van Bibber at Sports Complex gage is MHFD ALERT Gage Number 320 and has a period of record from 1990 through present (watershed area 17.5 mi^2). No data were available for the year 2006, when the gage's location was moved and channel realignment construction occurred. Also, there was insufficient data to determine an annual peak for 2012. For both years, a perception threshold of 286 cfs to infinity was applied, based on the assumption that flows were lower than the flood of record (286 cfs) in both those years, and therefore the flows must range between zero and 286 cfs.

WET's report acknowledged the influence of upstream canals which may divert flows out of Van Bibber Creek at the peak of high flow events. The exact extent to which canals interact with Van Bibber Creek is unknown, although there are most certainly interactions between Van Bibber Creek and the Croke Canal at high flows. The flow frequency analysis did not artificially account for these potential losses during high flow events in order to represent the flow frequency of actual conditions and waterway interactions as they currently exist. However, it should be noted that if the interaction between Van Bibber Creek and the upstream canals changes in the future, this flow frequency relationship may become a less accurate representation.

WET's report also noted that the gage was moved in 2006. While the distance between the two locations was not large, the physical arrangements and rating curves were different between gage locations. A 17C analysis was conducted using data from the entire period of record (1990-2019), as well as on data recorded at the gage's old location (1990-2005) and data recorded at the gage's new location (2007-2019). The computed curve for each analysis is presented in Table 4, and a graphical comparison of the curves and associated confidence intervals for the two different gage locations/periods of record is presented in Figure 3. While there are some differences between the results from the two periods of record, they are generally low, and differences may be partially due to the smaller data set available when the full record is divided into two periods of record. For all recurrence intervals, there is overlap between the confidence intervals, therefore this analysis does not provide evidence that data for the two periods are statistically different. It should be noted that at the lowest recurrence interval (2 years), the difference in the corresponding calculated flows between the two data sets was proportionally greater than at the highest recurrence intervals.

Ultimately, it was decided that despite the gage's relocation and slight differences in the calculated curves when the two data periods were compared, the full period of record should be used to maximize the data available for statistical analysis and because the purpose of this flow frequency analysis was more focused on larger recurrence interval events, where there was less discrepancy between the results of the two data sets.

Table 4. Van Bibber Creek at Sports Complex Flow Frequency Values for Different Periods of Record

Recurrence Interval (years)	Flow (cfs) [1990-2019]	Flow (cfs) [1990-2005]	Flow (cfs) [2007-2019]
2	88	76	106
5	144	127	162
10	186	167	205
20	229	210	251
50	290	270	317
100	339	320	372
200	391	374	432
500	465	452	520

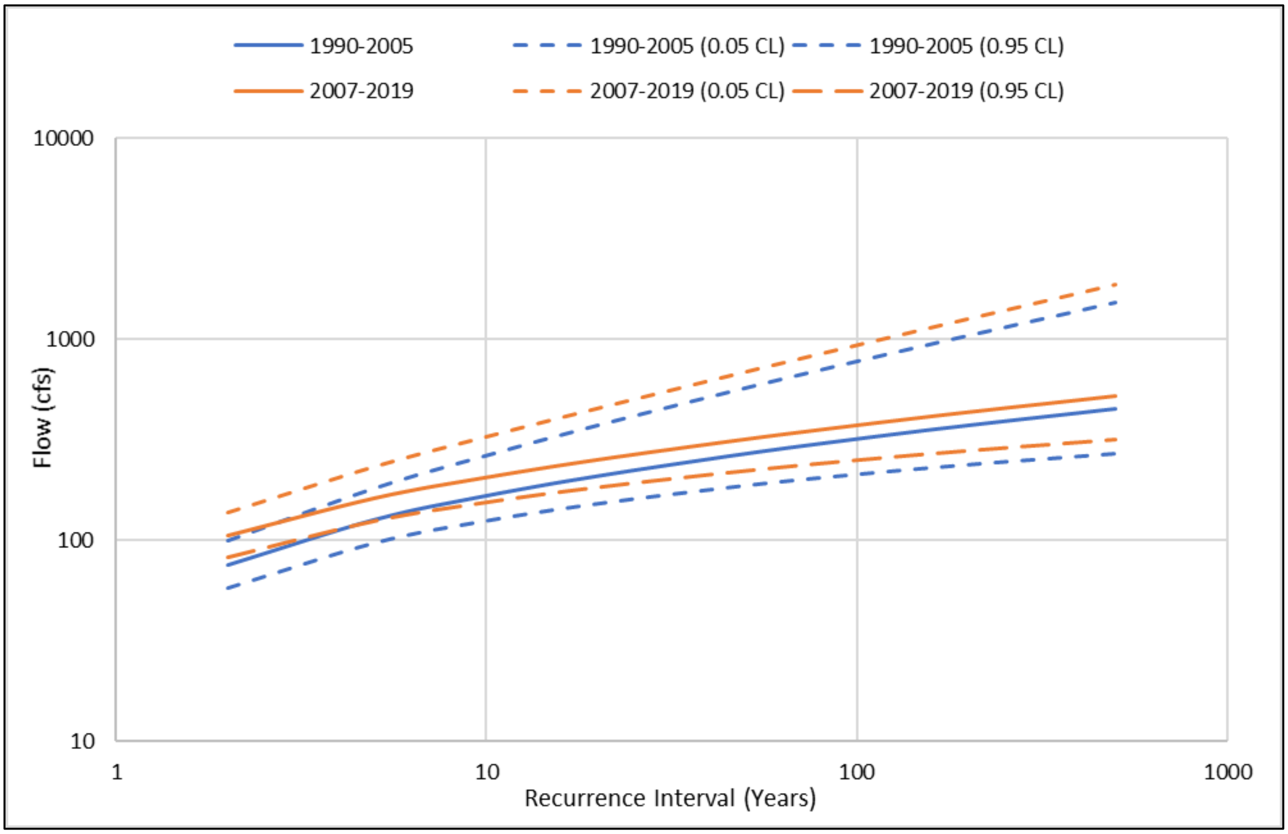


Figure 3. Van Bibber Creek at Sports Complex Comparison of Flow Frequency Curves and Confidence Limits Using Data from 1990-2005 versus 2007-2019

To be consistent with the upstream gage at Highway 93, paleoflood information was also included in the Van Bibber at Sports Complex analysis. It can be assumed that the high flows estimated at Highway 93 also would have occurred at the Sports Complex to some, unknown degree. The same perception thresholds and flow thresholds as Highway 93 were applied to the Sports Complex. The paleoflood peak was not increased or decreased because it is difficult to determine how factors such as the increased watershed area and possible canal diversions between Highway 93 and the Sports Complex interact to change peak flows. However, by virtue of the perception threshold and flow bounds, there is already some uncertainty factored into the analysis. A comparison of the results with and without the paleoflood information is included in Table 5.

Table 5. Van Bibber Creek at Sports Complex Flow Frequency Values Calculated Excluding and Including Paleoflood Information

Recurrence Interval (years)	Flow (cfs) [1990-2019, no paleoflood]	Flow (cfs) [1990-2019, with paleoflood]
2	88	94
5	144	200
10	186	304
20	229	436
50	290	664
100	339	887
200	391	1,163
500	465	1,629

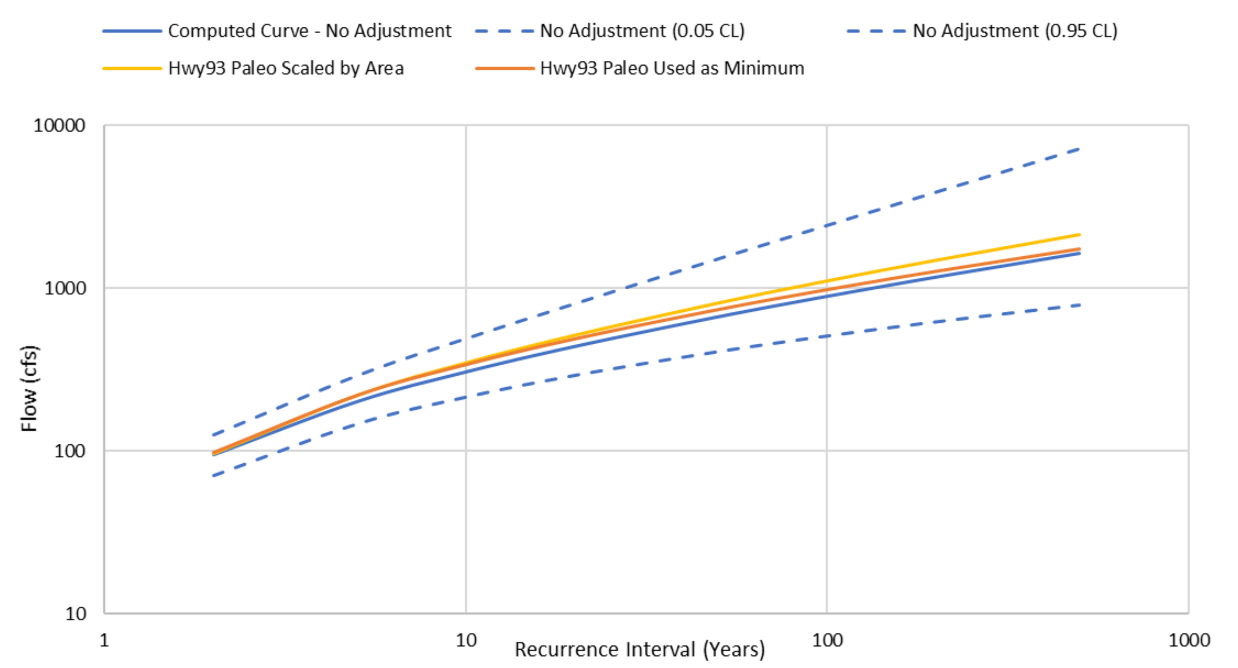
It should be noted that using the paleoflood peak flow estimated at Highway 93 for the flow at the Sports Complex may be underestimating the flow frequency values. Two different sensitivity analyses were conducted to better understand how various estimates of the paleoflood event at the Sports Complex gage would change the results.

One sensitivity run was made by scaling the paleoflood flows and corresponding perception thresholds by watershed area to the Sports Complex gage (increasing by a factor of approximately 1.86). This resulted in a flow range in 1948 of 4,800 cfs to 8,380 cfs, and a flow range of zero to 8,380 cfs for years between 1949 and 1989 when no measurements were made. This resulted in higher flow frequency values (a 100-year flow of 1,106 cfs versus 887 cfs).

A second sensitivity run was computed by setting the maximum paleoflood flow estimate at Highway 93 (4,500 cfs) as the minimum flow estimate for that year at the Sports Complex gage, so that possible flows in 1948 were between 4,500 cfs and infinity, and flows between 1949 and 1989 still had a flow range of zero to 4,500 cfs. This resulted in a flow frequency curve in between the two previously computed curves (a 100-year flow of 985 cfs).

The results of these two sensitivity analysis, along with the curve computed by making no adjustments to the Highway 93 paleoflood flow, are shown in Figure 4. It should be noted that the curves are not dramatically different using these three assumptions, and both sensitivity analyses fall within the confidence limits of the curve computed using no adjustments to the Highway 93 gage. However, the station skews computed for the sensitivity runs are large: 2.20 for the scaled by watershed area computation and 4.46 for the computation using the Highway 93 flow as a minimum. The station skew when no adjustment to the Highway 93 paleoflood peak flow is made has a value of 1.75. All three of these skews are much larger than the regional skew value of 0.05, although the latter is the closest.

Given the uncertainty in knowing the relationship between peak flows at Highway 93 and peak flows at the Sports Complex, especially for the largest-scale events, the flow frequency values computed using no adjustment to the paleoflood flow at Highway 93 are presented as the final values.



Note: The "Computed Curve - No Adjustment" was calculated using the same paleoflood peak flow value as was estimated at Highway 93. The "Hwy93 Paleo Scaled by Area" curve was computed by scaling the 1948 paleoflood minimum and maximum flow thresholds at Highway 93 to the Sports Complex site based on watershed area. The "Hwy93 Paleo Used as Minimum" curve was calculated by using the paleoflood maximum flow value at Highway 93 as a minimum flow value for 1948 at the Sports Complex. See main text for more information.

Figure 4. Van Bibber Creek at Sports Complex Computed Curves Using Different Paleoflood Estimates

The final flow frequency values and confidence limits for Van Bibber at Sports Complex, based on the full period of record as well as inclusion of the paleoflood estimate, are presented in Table 6, and the final flow frequency curve is shown in Figure 5.

Table 6. Van Bibber Creek at Sports Complex Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	94	71	126
5	200	147	292
10	304	214	492
20	436	291	806
50	664	407	1,521
100	887	507	2,443
200	1,163	618	3,905
500	1,629	784	7,214

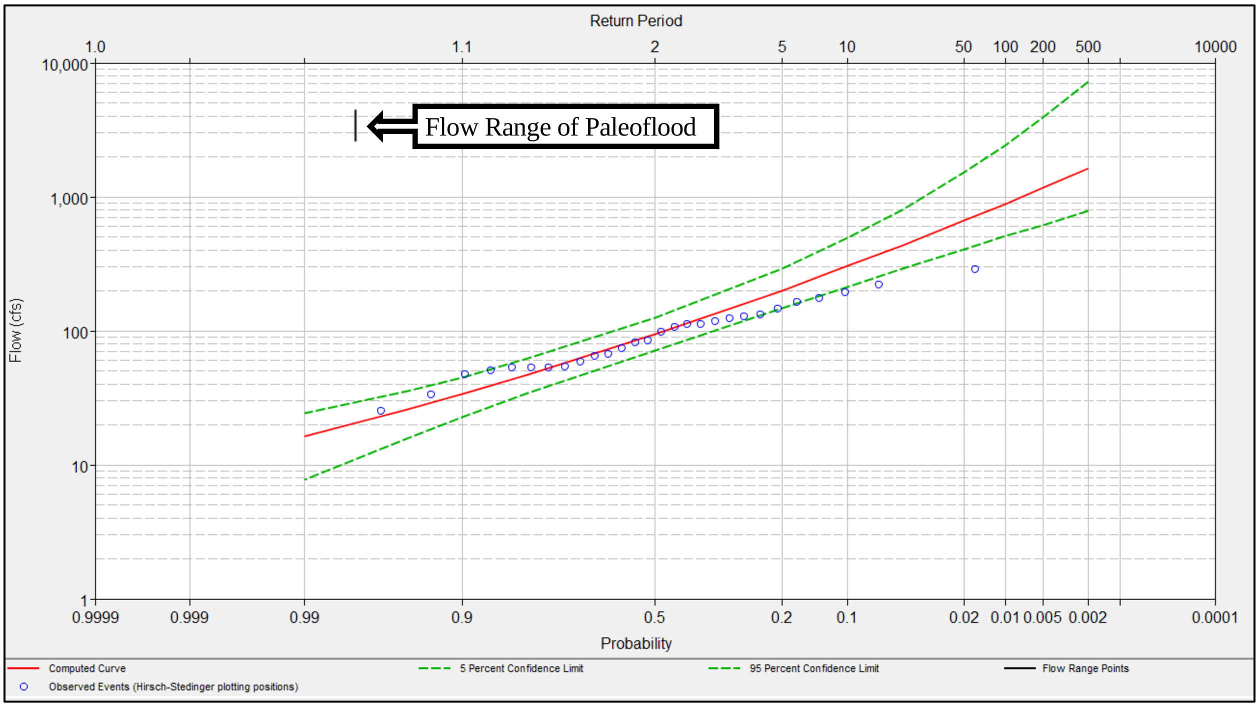


Figure 5. Van Bibber Creek at Sports Complex Flow Frequency Curve

A comparison between the computed curve and the 2019 FIS values plus the Olsson recommended 100-year flow is presented in Figure 6. Again, the FIS and Olsson values were scaled based on watershed area from the Van Bibber mouth to the gage location. The computed flow frequency curve is significantly lower than both the 2019 FIS frequency curve and Olsson's recommended 100-year flow. One contributing factor may be the possible diversions by canals upstream of the Sports Complex, which would likely not have been included in any FIS modeling, but which affect the measured flow values used for the computed curve. However, these canal diversions and any other inadvertent storage are not guaranteed to remain into the future. Therefore, for the purposes of design and flood mapping, the higher FIS values, which are likely less influenced by the current and unintentional diversions at high flows, should continue to be used. The Olsson 100-year flow, scaled to the gage location, is higher that the FIS 100-year peak discharge and is above the upper confidence limit of the flow frequency curve. The modeling by Olsson, in accordance with MHFD policies, would not have considered effects of diversions by ditches, inadvertent storage, and on site detention, so it is not surprising that the model results are significantly higher than the flow frequency results. As noted above, ditch diversions and inadvertent storage areas cannot be guaranteed to function as they currently do into the future, and this is a primary reason that WWE recommends continuing to use the current FIS 100-year peak discharge.

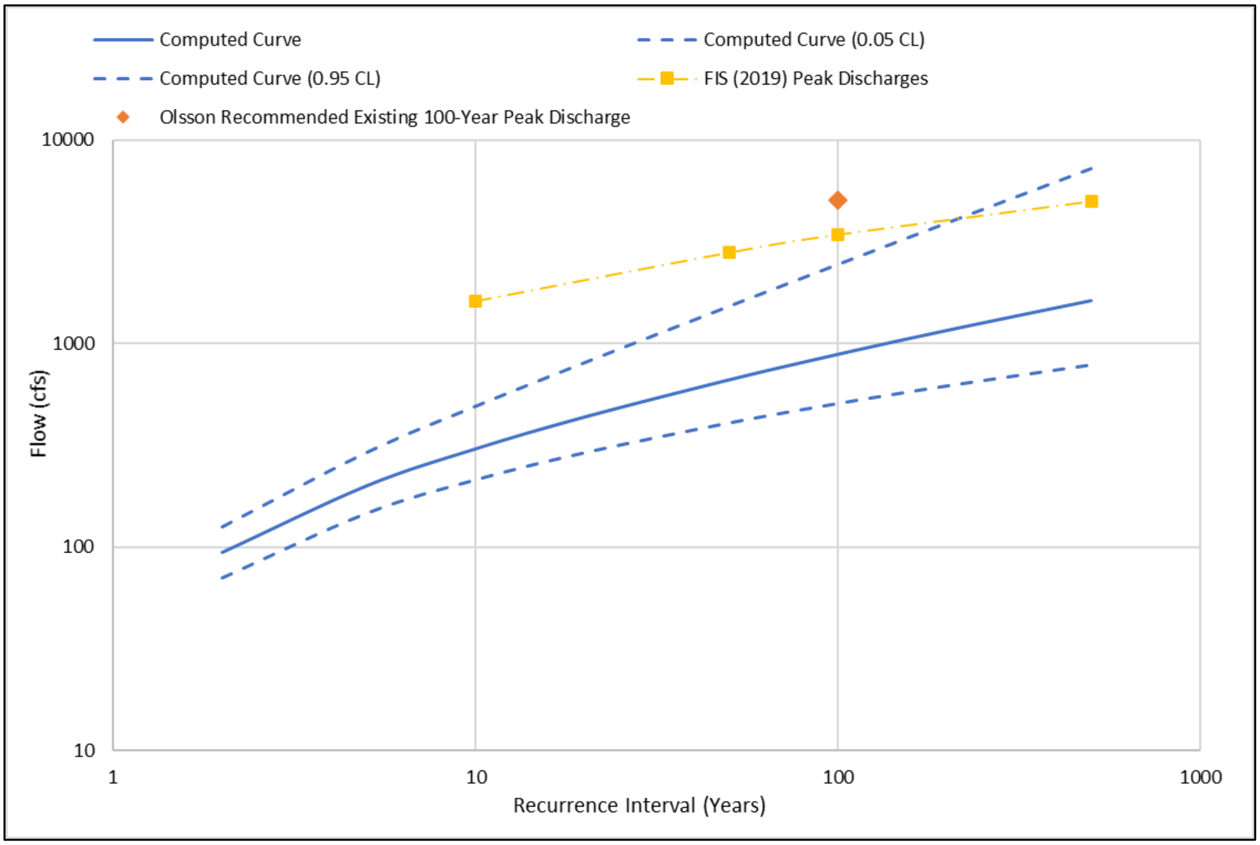


Figure 6. Van Bibber Creek at Sports Complex Comparison of Computed Frequency Curve with Existing FIS and Olsson Recommended Existing Conditions 100-yr Flow

5.0 RALSTON CREEK AT CARR STREET

The Ralston Creek at Carr Street gage is MHFD ALERT Gage Number 100 and is located below the confluence of Van Bibber Creek with Ralston Creek (watershed area of 89.1 mi^2). The period of record is from 1988 to the present, with no data for 2012. A perception threshold for 2012 of 3,010 cfs to infinity was used to represent that flows that year were assumed to be lower than the flood of record.

WET also noted that a new rating for the gage was implemented in 2014, following a channel improvement, which has resulted in a reduction in the average annual peak flows since then. The average annual peak flow pre-2014 is 1,075 cfs, and the average annual peak flow for the period since 2014 is 523 cfs. However, WET noted that when the pre-2014 and post-2014 annual peaks were plotted as a rating, no differences were readily observable.

When the 17C analysis was conducted using the entire period of record, 5 years were identified as low outliers by HEC-SSP: 2005, 2014, 2017, 2018, and 2019. The fact that four of the years since 2014 were flagged as low outliers is consistent with WET's observation that the new 2014 rating has decreased the recorded peak annual flows. A comparison of the flow frequency results when data from 2014 to the present are and are not included is presented in Table 7.

Table 7. Comparison of Ralston Creek at Carr Street Flow Frequency Results With and Without Years Since 2014 Included

Recurrence Interval (years)	Flow (cfs) [1988-Present]	Flow (cfs) [1988-2013]
2	823	949
5	1,333	1,450
10	1,712	1,812
20	2,105	2,179
50	2,655	2,683
100	3,099	3,083
200	3,568	3,502
500	4,233	4,089

Inclusion of the 2014 through 2019 data is most influential on the results for lower recurrence interval events. A comparison of flows at the upstream Van Bibber Gage and the Ralston Creek at Carr Street gage also indicates that some post-2014 peak flow measurements at the Ralston Creek at Carr Street gage may be inaccurately low. For example, the peak flow in 2018 at both gages was measured on September 5th. Upstream at Van Bibber, peak flows were recorded at 221 cfs, whereas downstream after these flows had confluenced with Ralston Creek the peak flow was measured at 238 cfs. This difference of only 17 cfs between the two gages seems uncharacteristically low even when considering the potential for offset peaks, especially when compared with differences in pre-2014 years which were on the order of several hundred to over a thousand cfs different. For these reasons, the final flow frequency values and confidence limits were calculated using only the data prior to 2014, the results of which are presented in Table 8 and the corresponding graphical curve is presented in Figure 7. No paleoflood information is available for this site.

Table 8. Ralston Creek at Carr Street Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	949	794	1,134
5	1,450	1,209	1,812
10	1,812	1,487	2,403
20	2,179	1,750	3,115
50	2,683	2,078	4,292
100	3,083	2,314	5,400
200	3,502	2,541	6,738
500	4,089	2,829	8,939

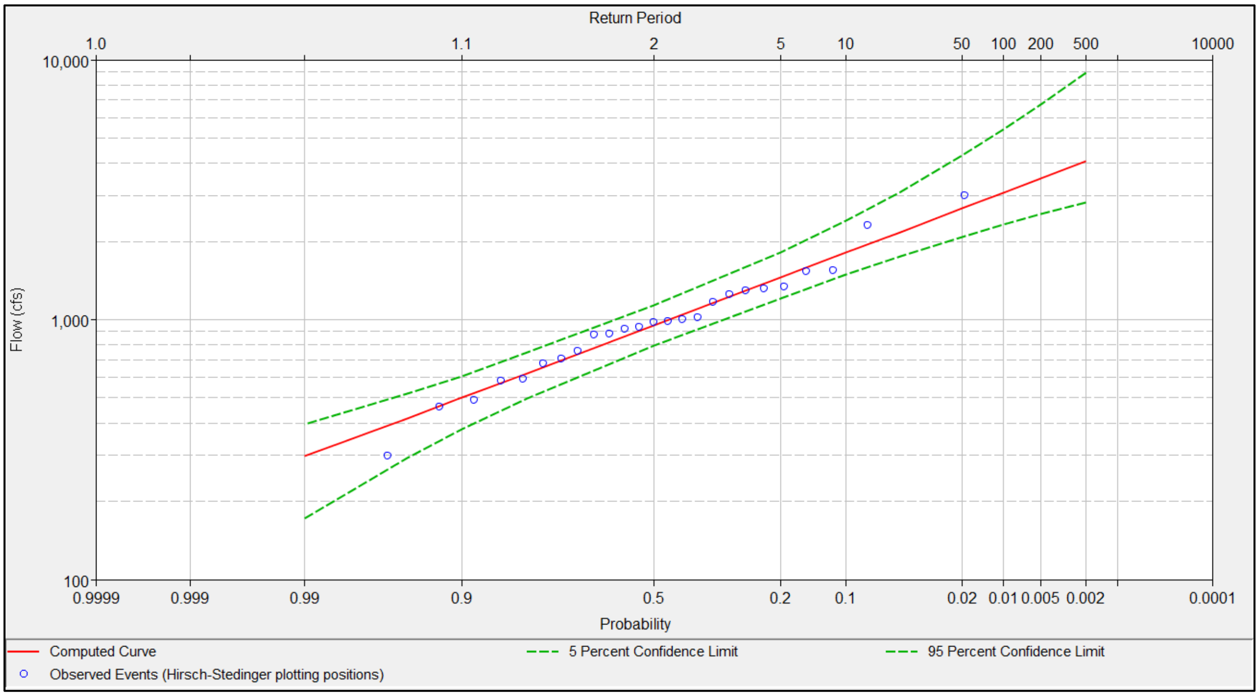


Figure 7. Ralston Creek at Carr Street Flow Frequency Curve

Note that the two highest flow events fall above the computed curve in Figure 7. One way to obtain a better fit at the top end of a curve is to exclude low values from the analysis by increasing the low outlier threshold. An analysis was conducted with the low outlier threshold set to 500 cfs so that the three lowest flow values were excluded. However, this had minimal effect on the results, and the two highest points were still above the computed curve. When the new low-outlier threshold was applied, the 100-year flow was 3,135 cfs compared with 3,083 cfs when no artificial low outlier was applied. Ultimately, because the points fall within the confidence limits, and the exclusion of the three lowest points did not cause a significantly better fit at the upper end, no artificial low outlier was applied (all points in the dataset from 1988-2013 pass the Multiple Grubbs-Beck test following standard Bulletin 17C procedures).

Finally, a comparison of the computed curve and the 2019 FIS values for Ralston Creek below the confluence with Van Bibber Creek is presented in Figure 8 (no 100-year flow value from Olsson exists at this location). Again, the computed curve and confidence intervals are below the FIS values. However, from a planning and design perspective, WWE does not recommend decreasing the FIS values, especially for the 100-year event, due to the effects of inadvertent storage and upstream diversions that are reflected in the gage record but that may not be present in the future.

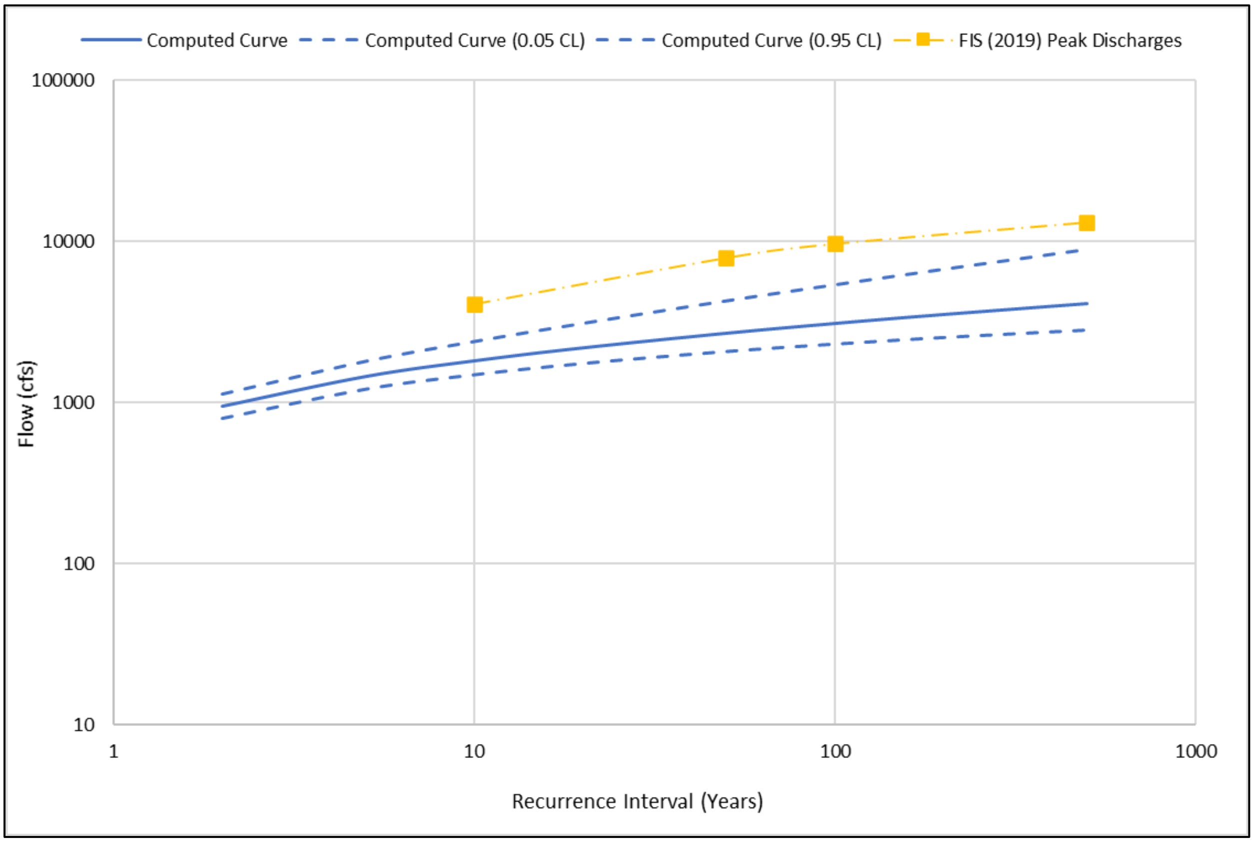


Figure 8. Ralston Creek at Carr Street Comparison of Computed Frequency Curve with Existing FIS

6.0 SUMMARY/CONCLUSION

Flow frequency analysis using HEC-SSP and following Bulletin 17C procedure was conducted for two gages along Van Bibber Creek and a third gage on Ralston Creek below its confluence with Van Bibber Creek. Annual peaks and potential data accuracy concerns were provided in an attached report by WET and were factored into the data used in HEC-SSP to generate flow frequency estimates. The final analysis methods and final results are summarized below for each gage.

For the Van Bibber at Highway 93 gage, data from 2016 through the present were excluded from the analysis because they were determined to likely be inaccurately high following a change in the PT height in August of 2015. However, a paleoflood value from 1948 was included in the analysis. The final results are summarized in Table 9.

Table 9. Van Bibber Creek at Highway 93 Flow Frequency Values and Confidence Limits (same as Table 3)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	**0.05**
2	50	31	80
5	152	92	273
10	284	162	568
20	486	255	1,121
50	906	419	2,676
100	1,391	582	5,132
200	2,079	784	9,799
500	3,422	1,118	22,994

For the Van Bibber at Sports Complex gage, the entire period of record was used and a paleoflood estimate parallel to that used at Highway 93 was included in the analysis. The resulting flow frequency estimates are summarized in Table 10. For events with a recurrence interval of 100 years and above the associated flows are lower at Sports Complex than at Highway 93. This may be due to the influence of canals upstream of the Sports Complex which may catch and divert flows out of Van Bibber Creek before they reach the Sports Complex gage.

Table 10. Van Bibber Creek at Sports Complex Flow Frequency Values and Confidence Limits (same as Table 6)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	**0.05**
2	94	71	126
5	200	147	292
10	304	214	492
20	436	291	806
50	664	407	1,521
100	887	507	2,443
200	1,163	618	3,905
500	1,629	784	7,214

Finally, for the Ralston Creek at Carr Street gage, data from 2014 through the present were based off a different gage rating than the previous years. This resulted in lower than average peak flow values, many of which were identified as low outliers by HEC-SSP. For this reason, and after examination of individual yearly data, the final analysis was based only on data up to 2013. The final flow frequency estimates are summarized in Table 11.

Table 11. Ralston Creek at Carr Street Flow Frequency Values and Confidence Limits (same as Table 8)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	949	794	1,134
5	1,450	1,209	1,812
10	1,812	1,487	2,403
20	2,179	1,750	3,115
50	2,683	2,078	4,292
100	3,083	2,314	5,400
200	3,502	2,541	6,738
500	4,089	2,829	8,939

While the computed curves do not perfectly match the effective curves used in the 2019 FIS for Van Bibber and Ralston Creek, it is WWE's recommendation that no changes be made to the effective curve. The FIS values were generally higher than the curves computed using Bulletin 17C procedure, and using these FIS values is conservative for planning and design purposes. However, WWE's analysis did not provide any evidence that the FIS values should be even higher than they already are, as the values modeled by Olsson may suggest.

As development in the watershed continues, and interactions between waterways change (such as the canals upstream of the Van Bibber at Sports Complex gage), it may be necessary to update these calculations accordingly. The results presented here represent calculations based on the best available data and may change as more years of data are collected.

cc:
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Kate Malers, WET, kmalers@water-and-earth.com

Attachment B. Flow Frequency Memorandum for Lena Gulch

WWE

MEMORANDUM

To: Kevin Stewart, P.E.
Mile High Flood District

From: Wright Water Engineers, Inc.
Andrew Earles, Ph.D., P.E., D.WRE, and Haley Rogers

Date: December 4, 2020

Re: Flow Frequency Analysis for Lena Gulch

1.0 INTRODUCTION

WWE conducted a flow frequency analysis for four gages located along Lena Gulch. WWE relied upon the peak annual flow data and corresponding reports provided by Water and Earth Technologies (WET) which included annual peak flow data and flagged potential data issues for each gage. This dataset served as a basis for a Bulletin 17C flow frequency analysis (England et al., 2018).

For all four gages, the USACE's HEC-SSP software was used to perform a Bulletin 17C analysis. All analyzed gages had between 13 and 40 years of annual peak flow data, so either a weighted skew or station skew was used for each analysis (although for the Lena Gulch at Highway 6 gage, which only had 13 years of data, an analysis using the regional skew was also evaluated). The weighted skew was based upon the individually calculated station skew and a regional skew of 0.05 (with a regional skew mean square error [MSE] of 0.302), consistent with the Bulletin 17B Average Skew Coefficient By One Degree Quadrangles map. 5% and 95% confidence limits were also calculated. High and low outlier tests were conducted through HEC-SSP, and flagged data were evaluated on a case by case basis. Note that the annual peak flow data reported were based on the calendar year (as opposed to the water year). Individual results and analysis methods are presented for each gage in subsequent sections. The results are also compared to the most current FIS and Major Drainageway Plan (MDP) values.

2.0 HOMOGENEITY CONSIDERATIONS

An underlying assumption of the Bulletin 17C analysis method is that the data analyzed are homogeneous. Before data were analyzed in HEC-SSP, it was necessary to confirm that the data were homogeneous, and therefore, it was appropriate to apply Bulletin 17C. Below are the major relevant considerations in evaluating each dataset:

- WWE performed statistical tests of each of the datasets to determine if they conform to the underlying log-normal statistical distribution used by Bulletin 17C. WWE applied the Shapiro-Wilk, Anderson-Darling, Lilliefors, and Jarque-Bera normality tests on log-transformed data. For the Highway 6 gage, all four tests found that the data were log-normally distributed. For the Lakewood gage, when the four years identified as low outliers (see discussion in Section 4.0) were removed from the dataset, all four tests found that the data were log-normally distributed. For the Nolte Pond gage, three of the four tests found that the data were not log-normally distributed. As discussed in Section 5.0, this gage has both data uncertainty issues and is potentially affected by regulation and therefore may not be the most appropriate dataset for use in Bulletin 17C. For Maple Grove Reservoir, when the zero flow years were removed (see explanation in Section 6.0), the data were found to be log-normally distributed by three of the four tests. However, this gage is influenced by regulation.

- The earliest data analyzed as a part of the systematic record for the gages evaluated were from 1973, at the Lakewood gage. Requirements for stormwater detention for peak flows was common in many parts of the Denver metropolitan area by the mid-1980s, which means that for three of the four gages (all of which have periods of record starting in 1986 or later) the widespread implementation of detention would have the result of diminishing the effects of changes to impervious area over time. For the Lakewood gage, there are 7 years of data between 1973 and 1987 which may be more affected by the impacts of watershed development. Two of the seven years were identified as low outliers and were not included in the frequency analysis (see discussion in Section 4.0). It should be noted that the second and third highest peaks on record occurred early in the record, in 1975 and 1977, respectively. However, as discussed above, the normality tests for this gage (once low outliers were removed) showed that the data for the entire period of record were log-normally distributed. Therefore, all four of the datasets would not be expected to violate the homogeneity assumptions of Bulletin 17C analysis due to watershed development.

- Both Lena Gulch at Nolte Pond and Lena Gulch at Maple Grove Reservoir are affected by regulation (Maple Grove Reservoir to a much greater degree). The regulation-influenced observed peak flows were used for flow frequency calculations, although specific adjustments and considerations are discussed in more detail for each gage (Section 5.0 for Nolte Pond and Section 6.0 for Maple Grove Reservoir). In general, the results at both these gages should be carefully considered due to the regulated nature of the input data. A discussion comparing all gages across the watershed is presented in Section 7.0.

- Finally, each dataset was evaluated for a mixed population of events caused by rainfall versus snowmelt driven hydrologic processes. The vast majority of events occurred between May and August (inclusive). Snowmelt events may have partially contributed to the events occurring in May, however across the four gages, the majority of the events occurred later in the summer when they were more likely due to rainfall events. In addition, the maximum elevation in the watersheds encompassing each of the four gages is 7,580 feet. These metrics also indicate that there is likely less snowmelt impact and most events were rainfall driven.

The individual analyses conducted for each gage are presented in the sections below. Additional discussion on the homogeneity of each dataset and appropriate adjustments and interpretation in accordance with Bulletin 17C are also included, where applicable.

3.0 LENA GULCH AT HIGHWAY 6

The Lena Gulch at Highway 6 gage is MHFD ALERT Gage Number 1043 and has a watershed area of 3.54 mi^2 (StreamStats). It has a period of record from 1985 through the present. However, for years prior to 1998, as well as 2007 through 2013, and 2018, the data are insufficient to determine an annual peak. This results in only 13 years of available annual peaks for analysis. Missing years of data during the systematic record (2007-2013 and 2018) were given a perception threshold of infinity to infinity. It is possible that 2013 was the peak of record (based on the Maple Grove Reservoir gage and other local gages and the historic storm experienced that year). Since the peak of record could not confidently be determined, all years of missing data were not capped with a maximum or minimum possible flow.

Because only 13 years of data were available for the Highway 6 gage, the Bulletin 17C analysis was conducted using both a weighted skew and a regional skew. Bulletin 17C recommends that a station have at least 10 years of data before the analysis methods can be utilized, which this gage does. However, sometimes for gages with only slightly over 10 years of data, the regional skew instead of the station skew is used because there are fewer data points with which to calculate a reliable station skew. For the Highway 6 gage, an analysis run was conducted using both skew methods, which is presented in Table 1.

Table 1. Comparison of Lena Gulch at Highway 6 Calculated with Weighted versus Regional Skew

Recurrence Interval (years)	Flow (cfs) [Weighted Skew]	Flow (cfs) [Regional Skew]
2	135	133
5	338	337
10	545	550
20	806	828
50	1,251	1,316
100	1,674	1,795
200	2,184	2,388
500	3,010	3,379
Adopted Skew	-0.037	0.050

The use of the weighted versus station skew did not have a major effect on the computed flow frequency curve. However, the weighted skew resulted in tighter confidence limits on the higher end (see Figure 1). For this reason, the final flow frequency curve was calculated using the weighted skew. The final results are presented in Table 2 and the curve is shown in Figure 2. Note that different horizontal scales are used in Figure 1 and Figure 2 (and later figures parallel to these). HEC-SSP output (which is used to generate Figure 2 and uses the red and green color scheme) uses a specialized probability scale that is unavailable in Excel.

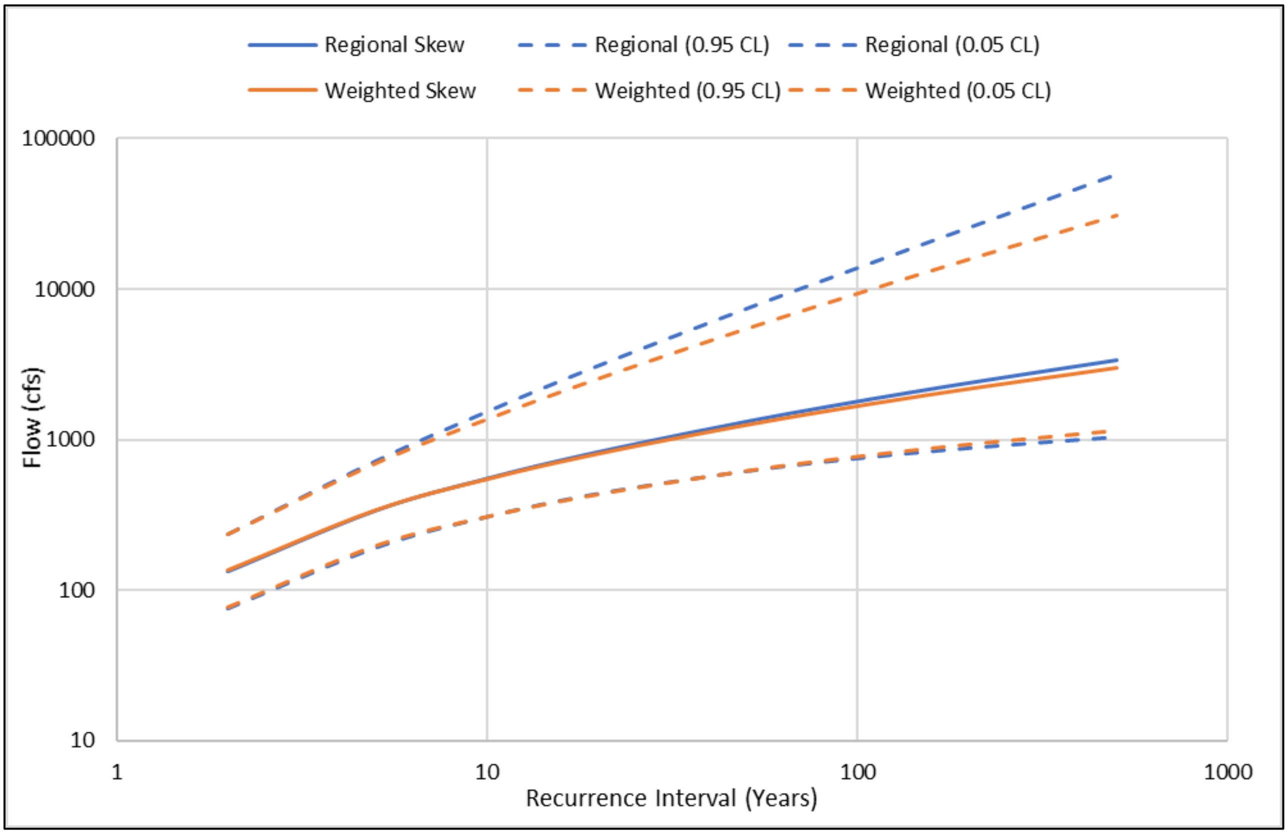

Figure 1. Flow Frequency Curves and Confidence Limits for Lena Gulch at Highway 6 using Weighted versus Regional Skew

Table 2. Lena Gulch at Highway 6 Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	**0.05**
2	135	77	234
5	338	198	691
10	545	308	1,359
20	806	435	2,538
50	1,251	622	5,433
100	1,674	775	9,345
200	2,184	935	15,811
500	3,010	1,154	30,911

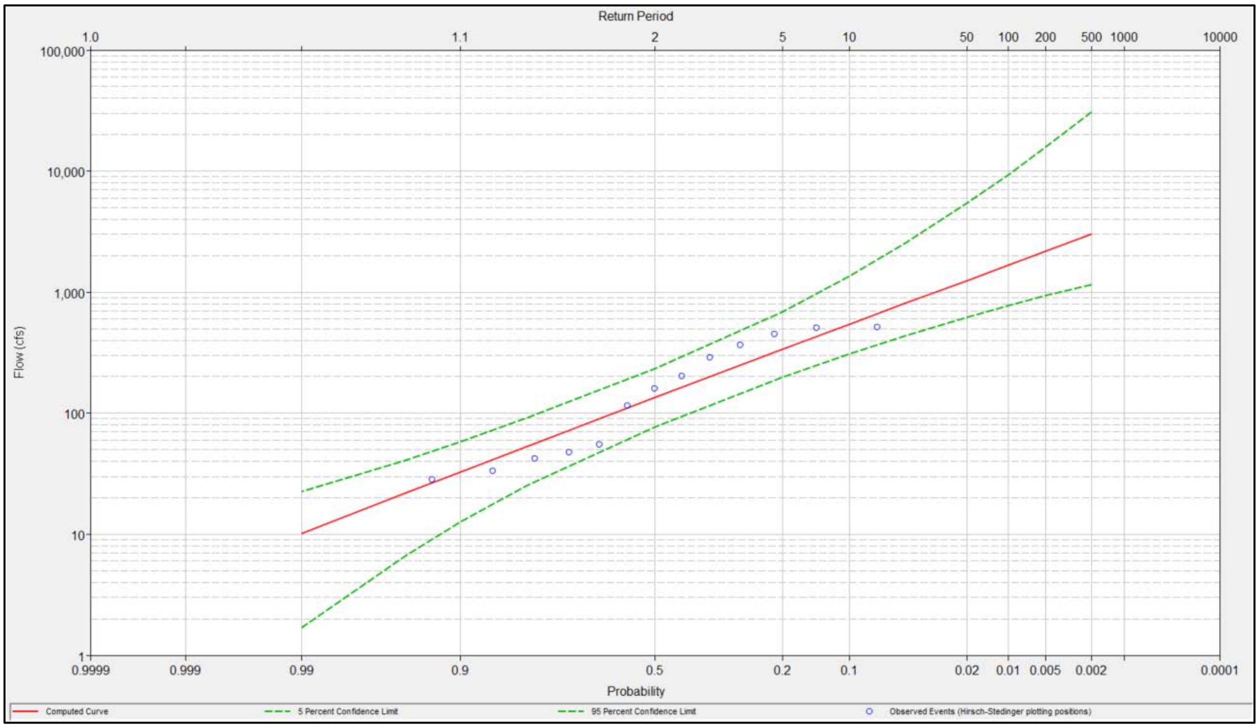

Figure 2. Lena Gulch at Highway 6 Flow Frequency Curve

Finally, a comparison of the computed flow frequency curve and the existing FIS (2019) peak discharges for Lena Gulch at U.S. Highway 6 is presented in Figure 3. The computed curve and existing FIS values agree relatively closely, especially for higher recurrence interval events. The FIS values all fall within the confidence limits of the flow frequency results. WWE does not recommend changing the FIS values. There is relative agreement between the two results, and the inadvertent storage present in the watershed, which is not guaranteed to remain constant into the future, likely has an effect on making the calculated flows for a given frequency lower than those modeled for the FIS, which would not have taken inadvertent storage into modeling consideration. The fact that the FIS curve diverges more from the computed curve at the 10-year event may also be impacted by upstream interactions with ditches and inadvertent storage or diversions.

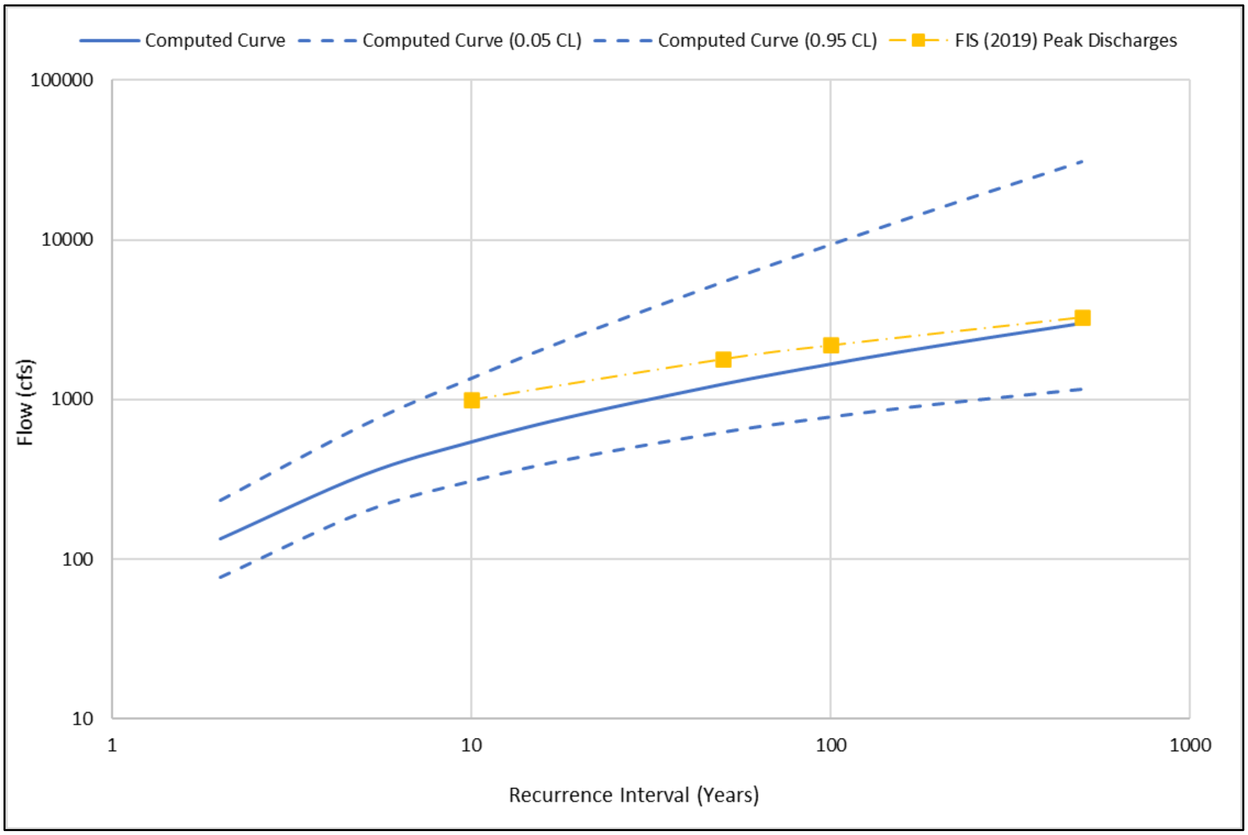

Figure 3. Lena Gulch at Highway 6 Comparison of Computed Frequency Curve with Existing FIS

The 2019 FIS does not present flow frequency data for any points on Lena Gulch further downstream than Highway 6, so no additional comparisons between the Bulletin 17C calculated results and the FIS values are presented. For the other gages, comparisons were made to the flood frequencies in the MDP for Lena Gulch. For the Lena Gulch at Highway 6 location, WWE compared values from the 2019 FIS with the most recent MDP values for Lena Gulch. The 2019 FIS used flows from the 1975 MDP for Lena Gulch. These flow frequencies were similar, but not identical, to the more recent 1994 MDP for the 10-year through 100-year events. For the 500-year event, the more recent MDP flow is over 30% larger than the 2019 FIS reported value. For all other gages, only values from the most recent MDP are discussed (since no FIS flow values were reported at these locations).

4.0 LENA GULCH AT LAKEWOOD

Lena Gulch at Lakewood is USGS Gage Number 06719560, with a watershed area of 9.06 mi^2 (8.8 mi^2 from StreamStats). Crest Stage Indicator (CSI) peak flow data are available from 1973 through 2013, and discharge data are available from 2013 through the present. This gage presented few data quality issues, although USGS did include a data flag for 2006 through 2019 that states: "All or part of the record affected by Urbanization, Mining, Agricultural changes, Channelization, or other" for those years. As discussed in Section 2.0, these changes may have an impact on flows throughout time, but it is likely minimized by the presence of stormwater detention for peak attenuation, which has been a requirement for all of the 2006 through 2019 period that was flagged by USGS.

The gage has missing data for 1974, 1981 through 1986, and 2008. The perception thresholds for 1974 and 1981 through 1986 were set to be infinity to infinity. Other gages along Lena Gulch do not have a record that extends back to these years, so there is no nearby gage evidence to determine whether a flood of record could have occurred in one of these years. For 2008, the perception threshold was set to be 693 cfs to infinity, with a corresponding flow threshold of 0 cfs to 693 cfs, based on the assumption that flows in 2008 were below the peak flow of record.

39 years with recorded data are available. A comparison of results using weighted skew versus station skew was conducted because there is an appropriately long period of record to potentially use station skew. The results are presented in Table 3. The calculated station skew was -0.363, compared with the weighted skew value of -0.164. The calculated flow frequency curves are relatively similar, although the confidence limits calculated using station skew are tighter at the upper end of the flow frequency curve than with weighted skew, and thus the station skew was used.

Table 3. Comparison of Lena Gulch at Lakewood Calculated with Weighted versus Station Skew

Recurrence Interval (years)	Flow (cfs) [Weighted Skew]	Flow (cfs) [Station Skew]
2	248	252
5	406	410
10	522	518
20	638	622
50	796	756
100	920	856
200	1,048	955
500	1,225	1,085
Adopted Skew	-0.164	-0.363

4 years of data were flagged as low outliers: 1973, 1979, 2002, and 2006, based on the Multiple Grubbs-Beck low outlier test. Because the purpose of the flow frequency analysis is to determine flows associated with the relatively higher recurrence interval events, the low outlier years were eliminated from the analysis. This allowed for a better fitting curve for the higher recurrence interval events. The four years, which are termed Potentially Influential Low Floods (PILFs) in Bulletin 17C, were instead each given a low perception threshold based on the low outlier Grubbs-Beck critical value calculated by HEC-SSP (in this case 110 cfs). This process is done by HEC-SSP automatically when low outliers are detected, so the results in Table 3 represent the values when the PILFs are excluded. For comparison purposes, a flow frequency curve with the PILFs included was also computed. The results are shown in Table 4, and demonstrate the large effect the low-outlier years have on the upper end of the computed curve when they are included in the analysis.

Table 4. Comparison of Lena Gulch at Lakewood Calculated with Low Outliers Included and Excluded (Using Station Skew)

Recurrence Interval (years)	Flow (cfs) [PILFs Excluded]	Flow (cfs) [PILFs Included]
2	252	270
5	410	429
10	518	509
20	622	567
50	756	621
100	856	650
200	955	672
500	1,085	693

The final results were selected to be those calculated using station skew with the PILFs excluded to best fit the upper ends of the flow frequency curve. The results, including the computed curve and confidence limits, are presented in Table 5 and Figure 4. The PILFs are shown in the figure as green boxes.

Table 5. Lena Gulch at Lakewood Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	**0.05**
2	252	210	300
5	410	345	492
10	518	434	645
20	622	516	829
50	756	610	1,133
100	856	670	1,416
200	955	723	1,755
500	1,085	782	2,315

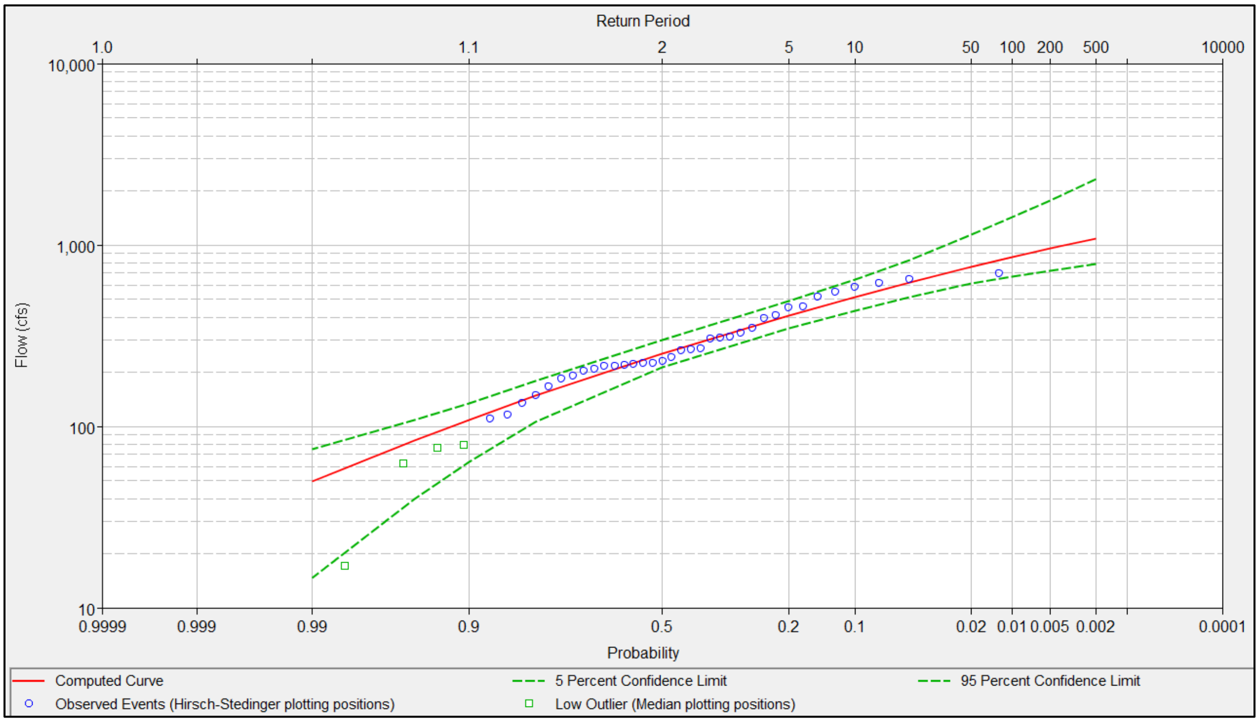

Figure 4. Lena Gulch at Lakewood Flow Frequency Curve

Note that the Lena Gulch at Lakewood gage presents the longest period of record, as well as the gage with the most reliable data in the watershed. A comparison and discussion of results across gages is presented in Section 7.0. The flow frequency values calculated at Highway 6 are higher than those calculated at Lakewood, although there is notable overlap in the confidence interval between the two computed curves. This could be due to a few factors. Differences in the period of record could impact the computed curve and confidence intervals (the Lakewood gage, which has many more years of data also has much tighter confidence intervals). Also, possible diversions between the two gages could contribute to the lower values calculated at Lakewood. Finally, WET noted that the Lena Gulch at Highway 6 gage's pressure transducer riser pipe may be measuring stage in turbulent water or possibly in a hydraulic jump, which could impact measurements. This could in turn contribute to some of the difference between these two gages. A comparison of the computed curves for the two gages and their corresponding confidence intervals is shown in Figure 5.

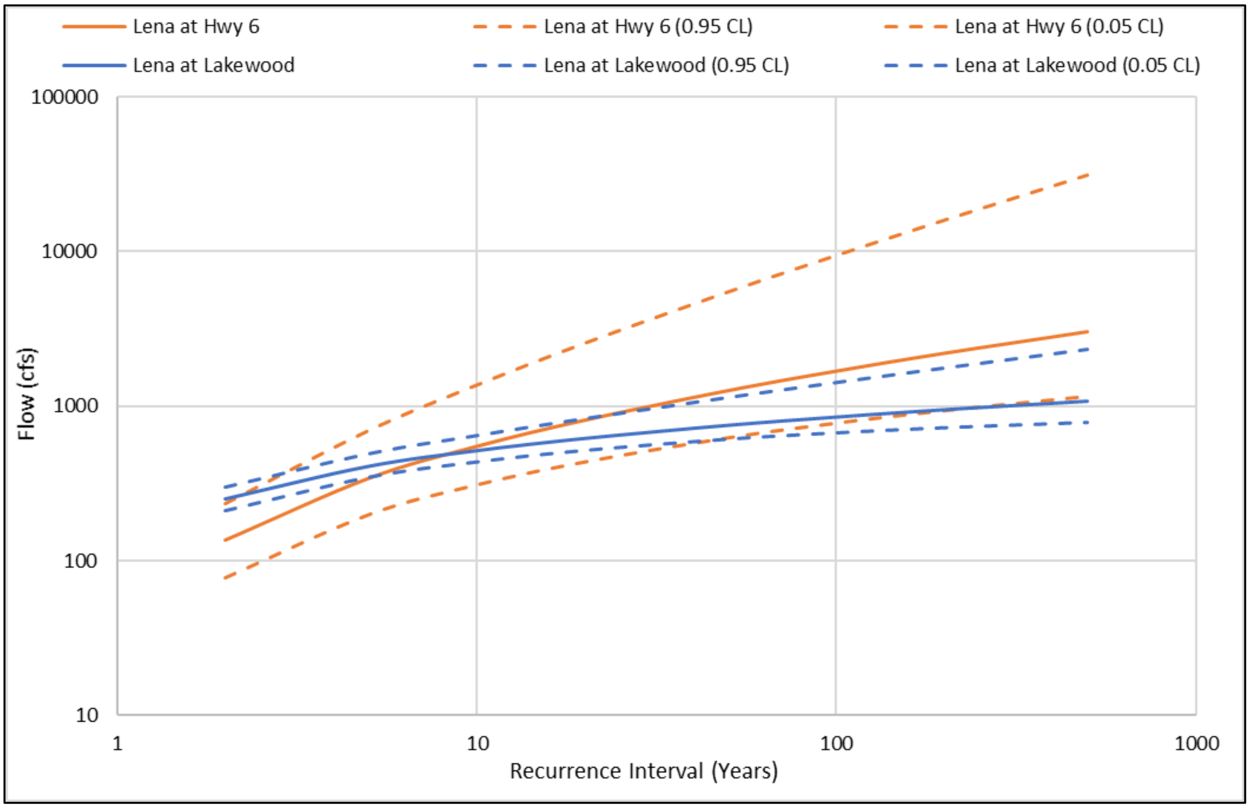

Figure 5. Comparison of Lena Gulch at Lakewood and Lena Gulch at Highway 6

A comparison of the computed curve and the MDP flows (*Major Drainageway Planning, Upper Lena Gulch*, 1994) at the approximate location of the Lakewood gage is presented in Figure 6. The computed curve and confidence intervals are below the MDP flow values. Again, it is WWE's recommendation that no changes be made to the effective MDP values. While the computed curve is well below the MDP, this may again be due to inadvertent storage or diversions in the watershed, which may decrease flows in the recent past and present but cannot be depended on going into the future.

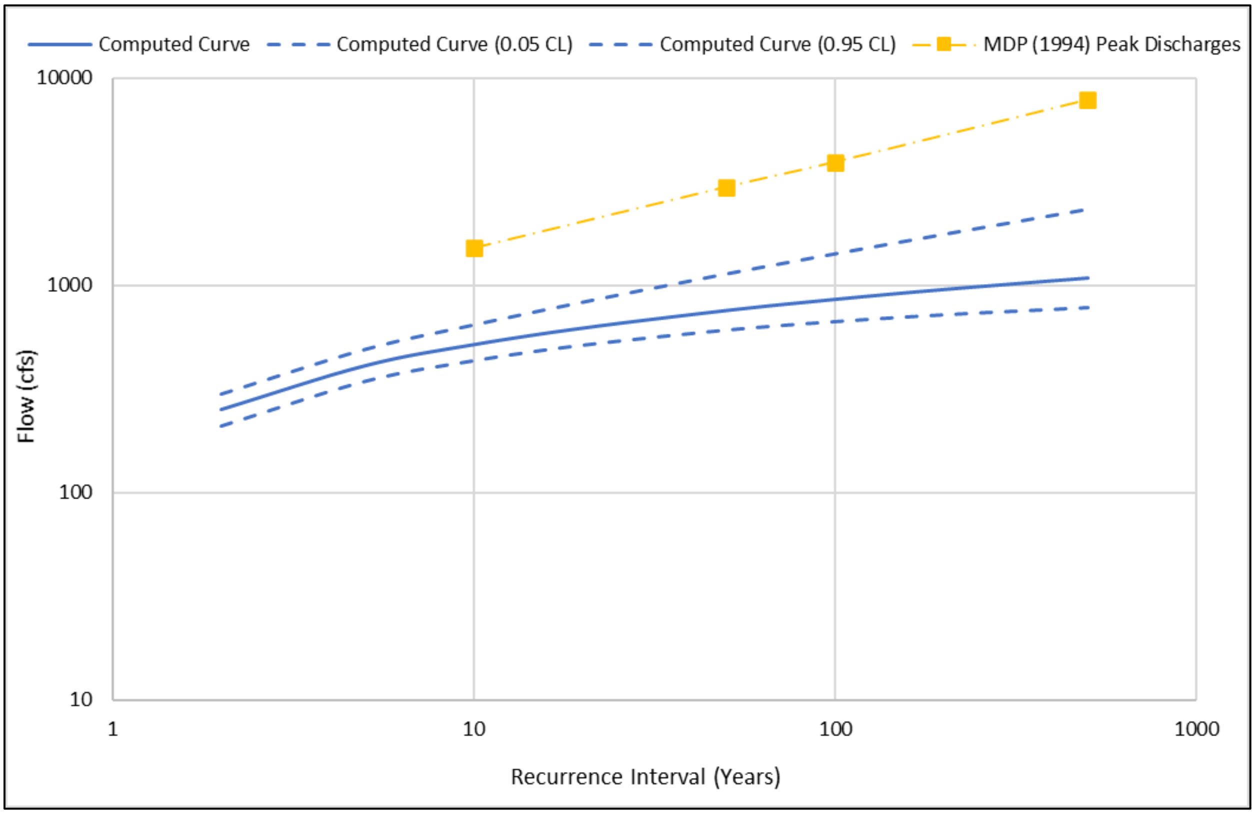

Figure 6. Lena Gulch at Lakewood Comparison of Computed Frequency Curve with Major Drainageway Plan

5.0 LENA GULCH AT NOLTE POND

Lena Gulch at Nolte Pond is MHFD ALERT Gage Number 1023 with a watershed area of 9.45 mi^2 (StreamStats). The period of record is from 1986 to the present, with annual peaks missing for 1989, 1994, and 2014 due to insufficient data during those years. The years with missing flow data were assigned a perception threshold of 865 cfs to infinity, and corresponding flow ranges of zero to 865 cfs in HEC-SSP. It was assumed that in all these years, the flow was less than the peak of record (865 cfs), which occurred in 2004.

This gage presented some data quality issues, evaluated by WET and briefly summarized below. The gage measures stage in a small residential pond, the outlet of which can be controlled by a removable flashboard that allows the homeowner to change the stage in the pond by 1 to 1.5 feet. This results in two different ratings for the gage, one when the flashboard is installed and one when it is not. Unfortunately, no record is maintained of when the flashboard has historically been installed, and thus one of the ratings must be applied with the knowledge that it may not be the most accurate one at all times. The “without flashboard” rating was applied for the entire period, which means that there may be points in the historical record when discharges were slightly overpredicted because the flashboard was in fact installed. Fortunately, the difference between the with and without flashboard flows are most dramatic at low flows, and the flow frequency analysis is based on peak flows which have generally less discrepancy between the two ratings. Possible over-estimation of flows ranges from zero cfs (for the high magnitude events) to 40 cfs (for the lower magnitude events), based on the two relevant rating curves.

In addition to uncertainty related to the flashboard, WET also noted another pond immediately upstream of the gage location which has a powered gate structure. The upstream pond could be drained by the homeowners to provide storage in anticipation of a large storm event. This upstream pond may provide some level of attenuation and regulation to flows measured at the Nolte Pond location, although only slight, as described by WET, and is likely not a major influence on the measured peak discharges.

Finally, as discussed in Section 2.0, the log-transformed data at Nolte Pond were found to not follow a log-normal distribution by three of the four statistical tests. It is difficult to say whether issues with the flashboard, potential upstream regulation, or some other factor is the cause for why the data are not log-normally distributed. The data were analyzed in HEC-SSP, but due to the data uncertainties and homogeneity issues discussed above, the results should not be adopted as official flow frequency values. Instead, the results are valuable for comparison with other gages in the watershed as well as a reasonableness check on the effective MDP values. The computed curve and confidence limits are presented in Table 6 and Figure 7.

Table 6. Lena Gulch at Nolte Pond Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	**0.05**
2	147	118	187
5	280	217	389
10	405	302	630
20	561	399	1,008
50	827	546	1,863
100	1,084	674	2,956
200	1,401	818	4,682
500	1,934	1,035	8,579

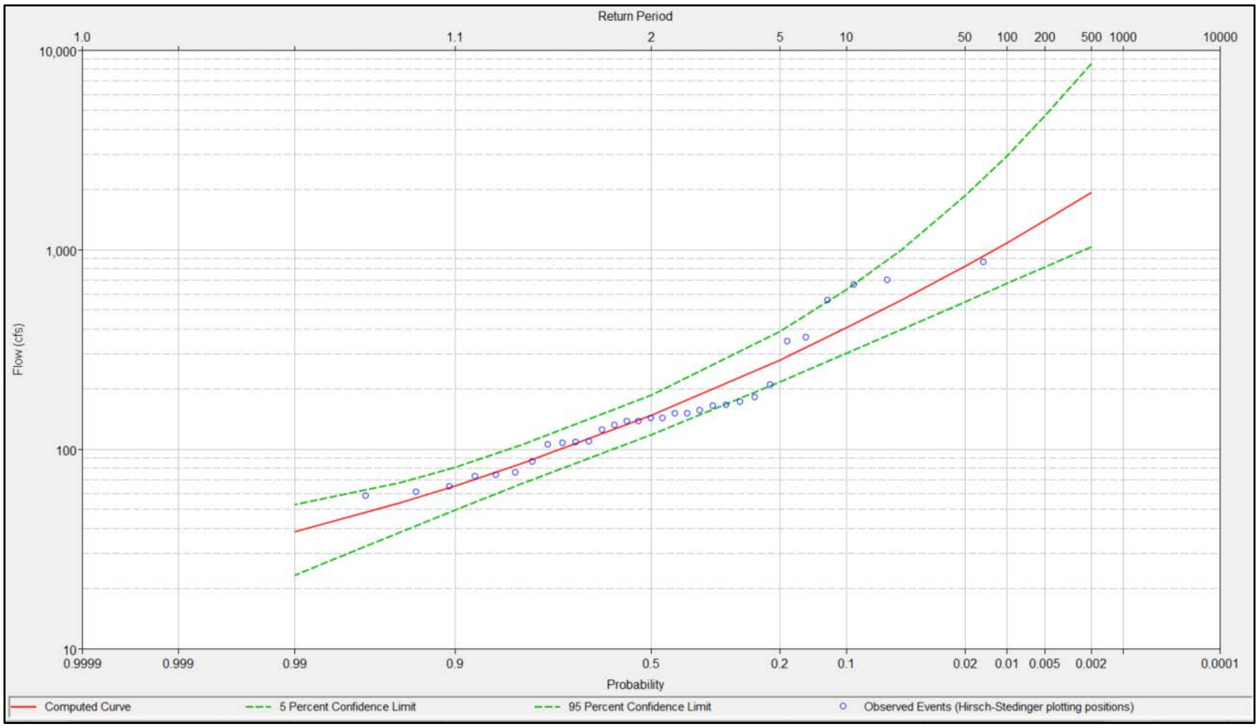

Figure 7. Lena Gulch at Nolte Pond Flow Frequency Curve

Considering that the Nolte Pond gage is less than a mile downstream of the Lakewood gage, and the difference in watershed area (as calculated by StreamStats) is less than 10%, from a planning and design perspective it makes most sense to use the flow frequency values calculated for the Lakewood gage, which has fewer potential data issues and less potential regulation influence. A comparison of the computed curves for Lena Gulch at Lakewood, scaled based on StreamStats calculated watershed area to the Nolte Pond location, versus the computed curve for Lena Gulch at Nolte Pond is presented in Table 7. For events with recurrence intervals below 50 years, the computed flows at Nolte Pond are lower than at Lakewood, however the opposite is true for the 50-year and higher recurrence interval events.

Table 7. Comparison of Lena Gulch at Lakewood and Lena Gulch at Nolte Pond Computed Curves

Recurrence Interval (years)	Computed Curve at Lakewood, scaled to Nolte by watershed area (cfs)	Computed Curve at Nolte Pond (cfs)
2	271	147
5	440	280
10	556	405
20	668	561
50	812	827
100	919	1,084
200	1,026	1,401
500	1,165	1,934

Finally, a comparison of the computed curve and peak discharge values from the MDP (*Major Drainageway Planning, Upper Lena Gulch*, 1994) at the approximate Nolte Pond location, are presented in Figure 8. Similar to the other gages, the computed curve is below the effective MDP flows. Only at the 500-year event is the MDP value within the computed confidence limits. As has been previously discussed related to inadvertent storage in the watershed, and based on data reliability questions, the computed curve does not provide any evidence that the MDP values should be adjusted.

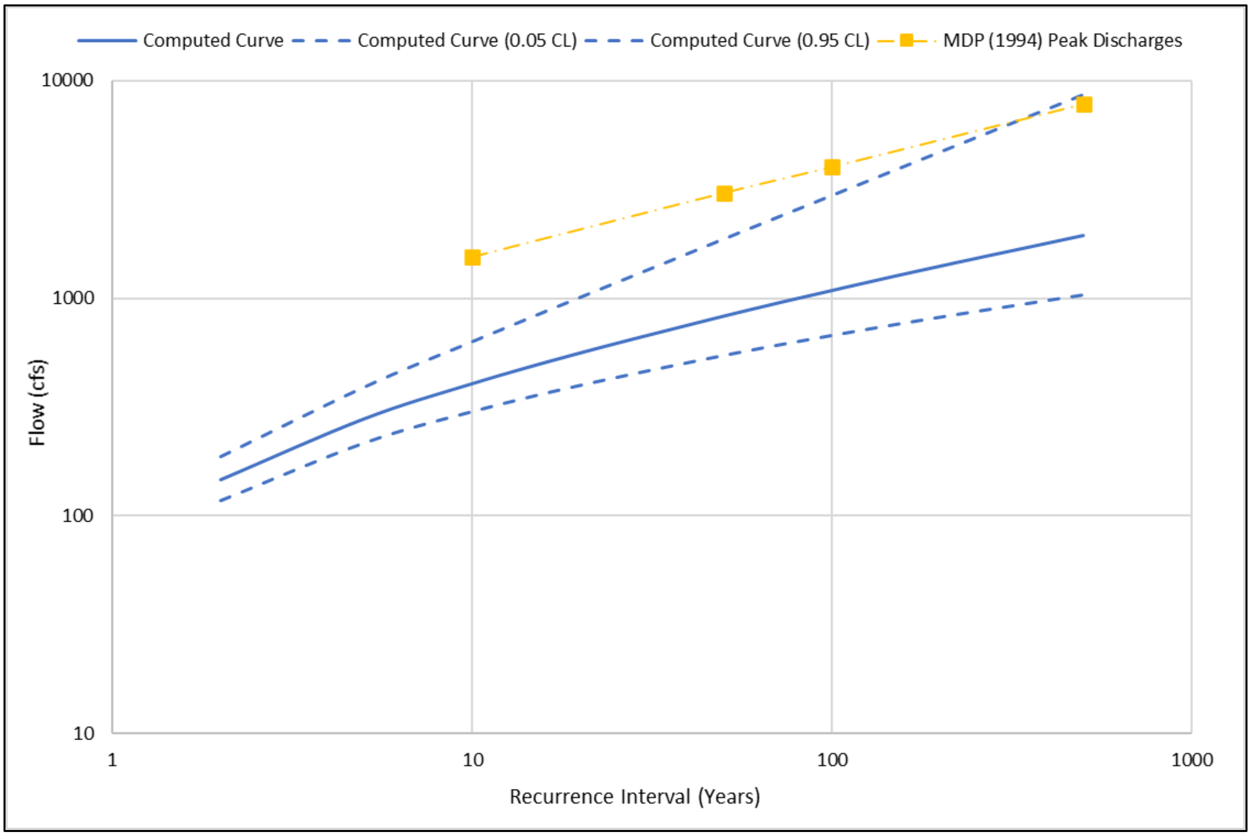

Figure 8. Lena Gulch at Nolte Pond Comparison of Computed Frequency Curve with Existing Major Drainageway Plan

6.0 LENA GULCH AT MAPLE GROVE RESERVOIR

Lena Gulch at Maple Grover Reservoir is MHFD Alert Gage Number 1003 (drainage area of 10.5 mi^2, StreamStats) and has peak annual flows from 1987 through the present. These peak flows are estimated using the reservoir water level and thus observed flows are affected by the reservoir storage and attenuation. This effect is immediately visible when the raw data for Lena Gulch at each gage are reviewed: peak flows at Maple Grove Reservoir are lower than peak flows at the other gages, with zero flow recorded in some years. In general, Bulletin 17C procedures were not developed to address "watersheds where flood flows are appreciably altered by reservoir regulation" (pg. 2). One technique to handle flow frequency analysis that is affected by regulation is to construct a hypothetical dataset approximating peak flows each year without the effect of regulation. In the case of Maple Grove Reservoir, this unregulated dataset could be developed by computing the change in reservoir elevation and storage volume on days of peak flows, then calculating the approximate inflows that would be associated with this change in storage volume. A flow frequency analysis could be conducted on these unregulated flows, and then a relationship developed to convert the unregulated flow frequency curve to a regulated flow frequency curve. However, that was not within the scope of the current assessment and may be unnecessary because the data, when years of zero outflow were removed, were found to be log-normally distributed by three of the four statistical tests (see discussion in Section 2.0). In addition, the computed curve fits the data relatively well for recurrence-intervals greater than 2-years.

Instead of going through a regulated-unregulated flow adjustment, HEC-SSP was run on the regulated data with slight modifications. Years with zero flow were eliminated and instead replaced with perception thresholds based on the Multiple Grubbs-Beck critical value calculated by HEC-SSP (equal to 5 cfs for Maple Grove Reservoir), following standard Bulletin 17C procedures. This is comparable to the elimination of PILFs at the Lakewood gage, and prevents low flow years from over-influencing the upper end of the frequency curve, which is the focus of the analysis. The flow frequency curve was also computed using the station skew. Regional skew values (which affect the results when either a regional or a weighted skew is used) were calculated based on unregulated streams in the region, and thus are not representative of skew patterns for regulated streams. Therefore, the station skew is used to reduce the influence of the unregulated flow assumption that is implicit in the regional skew coefficients. This difference in relationship between regulated and unregulated skews is evident when comparing the skew values. The regional skew, which was developed based on unregulated streams in the region, has a value of 0.05. However, the station skew for Maple Grove Reservoir has a value of -0.461. The results of this flow frequency calculation, with years of zero flow removed following Bulletin 17C procedure and computed using station skew, are presented in Table 8 and Figure 9.

Table 8. Lena Gulch at Maple Grove Reservoir Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	22	14	33
5	61	41	92
10	98	66	161
20	142	94	266
50	209	131	494
100	267	157	772
200	329	181	1,191
500	418	211	2,095

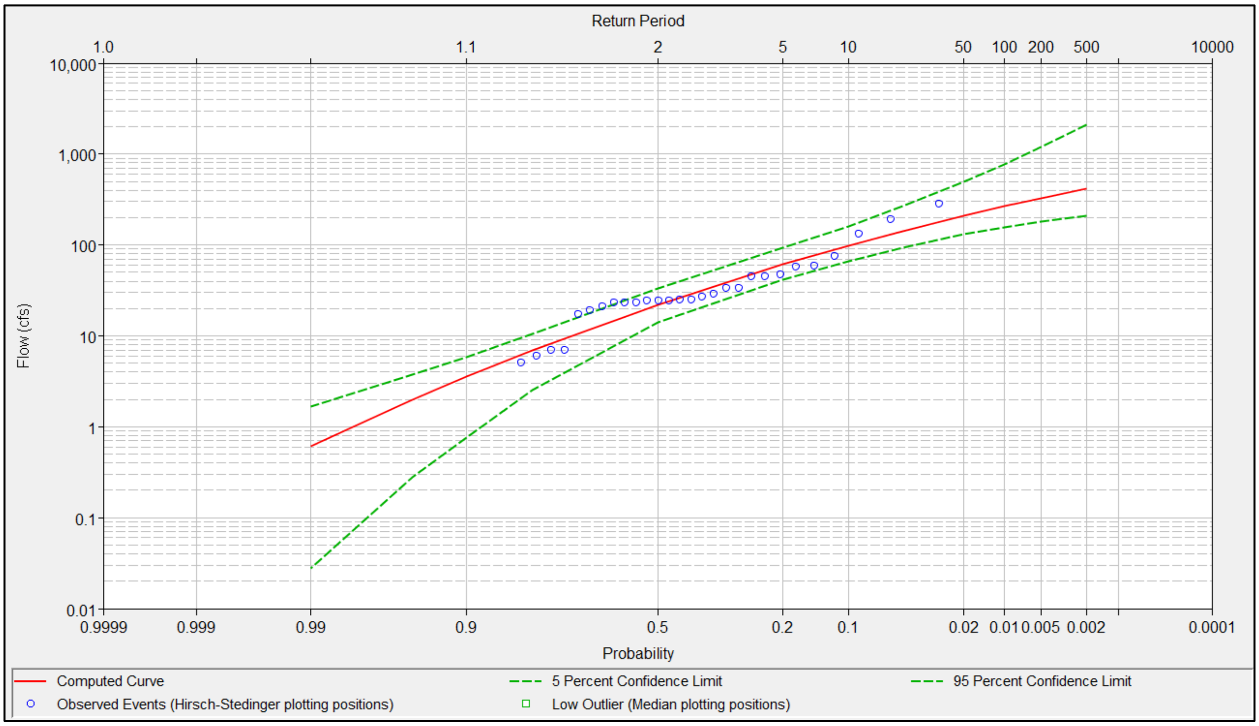

Figure 9. Lena Gulch at Maple Grove Reservoir Flow Frequency Curve

As noted by WET, the release gates have never been lowered for emergency operations during the period of record. Therefore, peak flows for each year are more a reflection of the annual variability across multiple storm events (because of the effect of storage) than policy or operation decisions about emergency flow releases. For this reason, the flow frequency results still provide valuable information for flows at this location, however they should be used with caution and the application of the results carefully considered. Additionally, if gate operation procedures change, corresponding flow frequency will be similarly affected.

A comparison of the computed curve and the MDP values (*Major Drainageway Planning, Lower Lena Gulch*, 2007) is presented in Figure 10. For all flows above the 2-year event, the computed curve is lower than the MDP curve. Note that the 2-year flow from the MDP is only 1 cfs. The difference between the computed curve and the MDP flows is also of greater magnitude for the higher-recurrence interval events. It should be noted that the MDP assumed that the initial reservoir water surface elevation was the top of the conservation pool (elevation 5525.0 feet, volume is approximately 1,070 acre-feet). The Bulletin 17C analysis did not account for a constant starting reservoir pool but used the measured peak outflows, regardless of reservoir capacity at the start of the event.

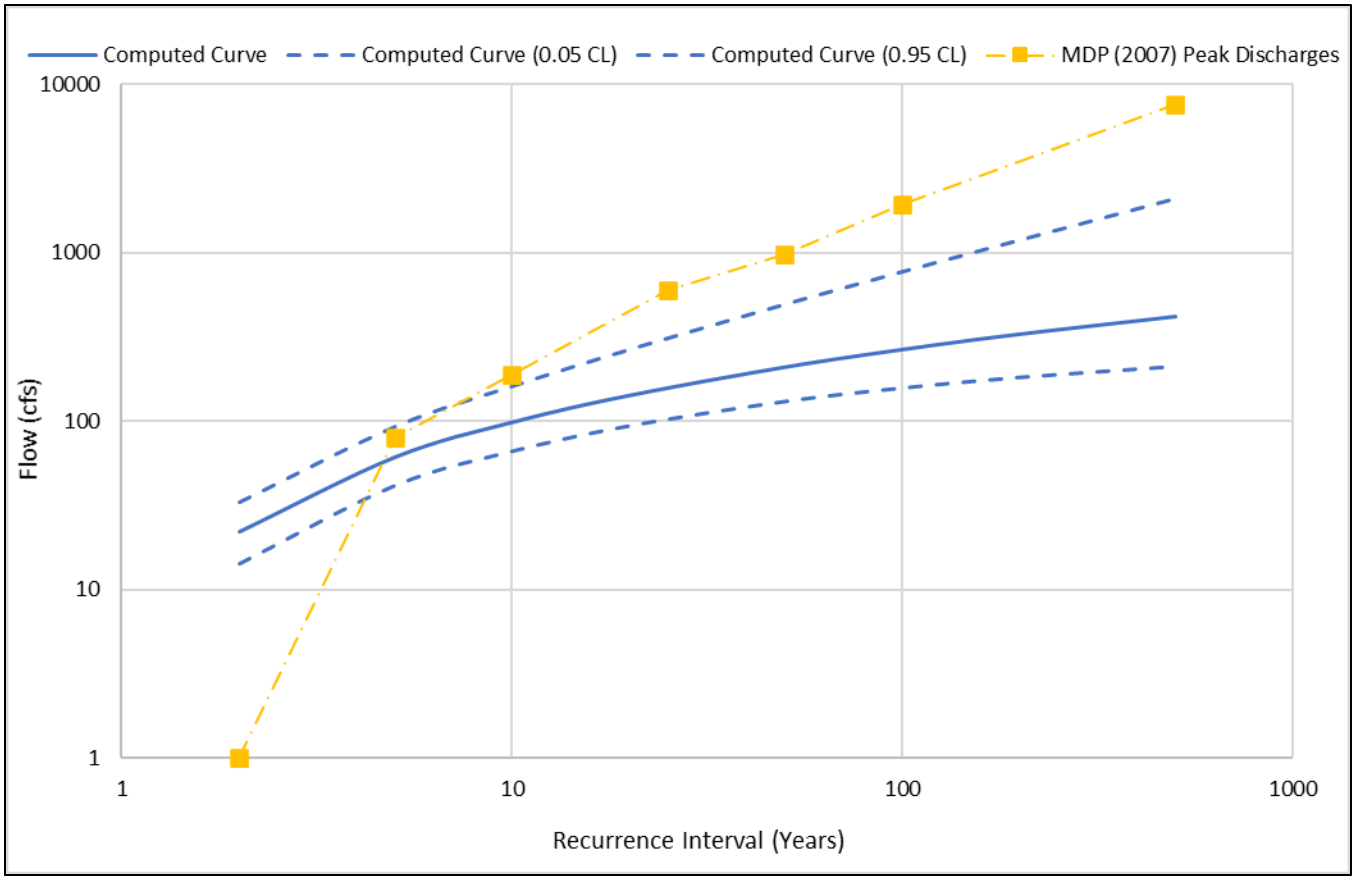

Figure 10. Lena Gulch at Maple Grove Reservoir Comparison of Computed Frequency Curve with Existing Major Drainageway Plan

7.0 SUMMARY/CONCLUSION

Flow frequency analysis using HEC-SSP and following Bulletin 17C procedures was conducted for four gages along Lena Gulch. Annual peaks and potential data accuracy concerns were provided in the associated WET report. The final analysis methods and results are reprinted below for each gage, as well as a discussion on the results across the watershed due to data and regulation issues at some of the gages.

For the Lena Gulch at Highway 6 gage, the full period of record was used (13 years) with a weighted skew, and there are no major data or homogeneity issues. The results are reprinted in Table 9.

Table 9. Lena Gulch at Highway 6 Flow Frequency Values and Confidence Limits (same as Table 2)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	0.95
2	135	77	234
5	338	198	691
10	545	308	1,359
20	806	435	2,538
50	1,251	622	5,433
100	1,674	775	9,345
200	2,184	935	15,811
500	3,010	1,154	30,911

The Lena Gulch at Lakewood gage has the longest period of record in the watershed with no major data reliability issues. Four years of data were flagged as low outliers by the Multiple Grubbs-Beck low outlier test and were modified using perception thresholds following Bulletin 17C procedure. The computed flow frequency, based on a station skew, is presented in Table 10. Depending on the intended use of the flow frequency results, it may be most appropriate to scale the computed curve at Lakewood for downstream locations due to data and possible regulation issues at Nolte Pond as well as regulation at Maple Grove Reservoir. The difference in watershed area (calculated by StreamStats) between Lakewood and Nolte Pond is less than 10% and the difference between Lakewood and Maple Grove Reservoir is less than 20%.

Table 10. Lena Gulch at Lakewood Flow Frequency Values and Confidence Limits (same as Table 5)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	0.95
2	252	210	300
5	410	345	492
10	518	434	645
20	622	516	829
50	756	610	1,133
100	856	670	1,416
200	955	723	1,755
500	1,085	782	2,315

The Lena Gulch at Nolte Pond gage is located at the outlet of a residential pond, which is controlled by a flashboard that can be removed by the homeowner. Unfortunately, no record is available for the times when the flashboard was or was not installed, and therefore the assumed rating curve may be slightly overpredicting values, although this overprediction is most influential at lower flows which are not the focus of this analysis. WET also noted an upstream pond that may provide some level of regulation for the flows measured at this gage, although likely a relatively minor influence. Finally, the data for Nolte Pond were found to not fit a log-normal distribution by three of the four statistical tests performed by WWE. Taken together, WWE notes that it may be more appropriate to scale the calculated flows at the Lakewood gage to the appropriate location, as opposed to using the calculated curve at Lena Gulch. However, the Bulletin 17C results are presented in Table 11 for comparison purposes.

Table 11. Lena Gulch at Nolte Pond Flow Frequency Values and Confidence Limits (same as Table 6)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	**0.95**
2	147	118	187
5	280	217	389
10	405	302	630
20	561	399	1,008
50	827	546	1,863
100	1,084	674	2,956
200	1,401	818	4,682
500	1,934	1,035	8,579

Finally, the computed curve and confidence limits for Maple Grove Reservoir are reprinted in Table 12. Maple Grove Reservoir is highly influenced by regulation. WWE eliminated the years of zero flow from the dataset and then calculated the corresponding flow frequency using station skew. The Bulletin 17C procedures were not designed for use with highly regulated datasets, and thus these results should be applied with caution. However, the fact that the emergency gates have never been lowered decreases the influence of policy and operation decisions on the computed results and makes the flow frequency results more representative of seasonal flow patterns than human decisions.

Table 12. Lena Gulch at Maple Grove Reservoir Flow Frequency Values and Confidence Limits (same as Table 8)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.05	**0.95**
2	22	14	33
5	61	41	92
10	98	66	161
20	142	94	266
50	209	131	494
100	267	157	772
200	329	181	1,191
500	418	211	2,095

For all four gages, WWE recommends that no changes in flows be made to the FIS value (for the Highway 6 gage) and the MDP values (for the Lakewood, Nolte Pond, and Maple Grove Reservoir gages). The fact that the computed curves using Bulletin 17C were lower than the current effective flows for all gages provides valuable information that watershed changes have not drastically increased flow frequency in the watershed compared to the conditions assumed for modeling.

As development in the watershed continues, and interactions between waterways change, it may be necessary to update these calculations accordingly. The results presented here represent calculations based on the best available data and may change as more years of data and additional gage information are collected.

cc:
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Attachment C. Flow Frequency Memorandum for Little Dry Creek

WWE

MEMORANDUM

To: Kevin Stewart, P.E.
Mile High Flood District

From: Wright Water Engineers, Inc.
Andrew Earles, Ph.D., P.E., D.WRE, and Haley Rogers

Date: December 18, 2020

Re: Flow Frequency Analysis for Little Dry Creek

1.0 INTRODUCTION

WWE conducted a flow frequency analysis for two gages located on Little Dry Creek (Westminster and 64th Avenue). WWE relied upon the peak annual flow data and corresponding reports provided by Water and Earth Technologies (WET) which included annual peak flow data and flagged potential data issues for each gage. This dataset served as a basis for a Bulletin 17C flow frequency analysis (England et al., 2018).

For both gages, the USACE's HEC-SSP software was used to perform a Bulletin 17C analysis. The Westminster and 64th Avenue gages had a period of record of 36 years, and 18 years, respectively. A station skew was used for the Westminster gage, and a weighted skew was used for the 64th Avenue gage. The weighted skew was based upon the individually calculated station skew and a regional skew of 0.05 (with a regional skew mean square error [MSE] of 0.302), consistent with the Bulletin 17B Average Skew Coefficient by One Degree Quadrangles map. 5% and 95% confidence limits were also calculated. High and low outlier tests were conducted through HEC-SSP, and flagged data were evaluated on a case-by-case basis. Note that the annual peak flow data reported were based on the calendar year (as opposed to the water year). Individual results and analysis methods are presented for each gage in subsequent sections. The results are also compared to the effective FIS values (2018) and the implications discussed at the end of each section.

Note that another USGS gage, Little Dry Creek Below Federal Boulevard at Westminster (number 06719845) also exists within the watershed. However, this site has very limited data and therefore was not analyzed in this report.

2.0 HOMOGENEITY CONSIDERATIONS

An underlying assumption of the Bulletin 17C analysis method is that the data analyzed are homogeneous. Before data were analyzed in HEC-SSP, it was necessary to confirm that the data were homogeneous, and therefore, it was appropriate to apply Bulletin 17C. Below are the major relevant considerations in evaluating each dataset:

- WWE performed statistical tests of each of the datasets to determine if they conform to the underlying log-normal statistical distribution used by Bulletin 17C. WWE applied the Shapiro-Wilk, Anderson-Darling, Lilliefors, and Jarque-Bera normality tests on log-transformed data. All four tests showed that the log-transformed datasets for Little Dry Creek at Westminster and 64th Avenue were normally distributed.

- The earliest data analyzed as a part of the systematic record for the gages evaluated were from 1982. Stormwater detention for peak flow attenuation was common in many parts of the Denver metropolitan area by the mid-1980s, which means that there is only minimal gage data prior to the widespread implementation of detention. While the watersheds upstream of the gages evaluated undoubtedly have experienced increases in impervious area associated with new development and redevelopment over the periods analyzed, the widespread implementation of detention would have the result of diminishing the effects of changes to impervious area over time, resulting in datasets that would not be expected to violate the homogeneity assumptions of Bulletin 17C analysis.

- In the Little Dry Creek watershed, upstream of both gages, are Lake Arbor and the Pomona Lakes which provide storage for the watershed. It is notable that the analysis for Little Dry Creek uses data exclusively from the post-reservoir period (based on the existence and discussion of both lakes in the 1979 Major Drainageway Plan for Little Dry Creek), so the analysis avoids issues of homogeneity related to data from before and after reservoir construction. The role of these reservoirs as detention facilities also partially offsets the effects of development and increased impervious land through time, further maintaining homogeneity across the data. In addition, the statistical normality tests show that both datasets follow a log-normal distribution.

- Finally, each dataset was evaluated for a mixed population of events caused by rainfall versus snowmelt driven hydrologic processes. The vast majority of events occurred between June and August (inclusive). Snowmelt events may have partially contributed to the few events occurring in May, however across the two gages, the majority of the events occurred later in the summer when they were more likely due to rainfall events. Also, the maximum basin elevation for both gages is 5,820 feet. This maximum elevation indicates that there is likely relatively low snowmelt impact and most events were rainfall driven. Finally, while a full meteorological analysis for each year was not conducted, as previously discussed, the datasets were determined to be log-normally distributed, and thus adjustments for mixed population were not made.

Based on the above discussion, the data were determined to be appropriately homogeneous for use in accordance with the Bulletin 17C criteria. The individual analyses conducted for each gage are presented in the sections below.

3.0 LITTLE DRY CREEK AT WESTMINSTER

Little Dry Creek at Westminster is a USGS stream gage, number 06719840, and a drainage area of 10.5 mi^2 (StreamStats). It has a period of record from 1982 through present. No data were available for the years 1998 and 2008, resulting in 36 years of data. For both 1998 and 2008, a perception threshold of 1,280 cfs to infinity (corresponding to a flow range of 0 to 1,280 cfs) was applied. The flood of record occurred in 1991 and there is no indication that flows in 1998 or 2008 exceeded the 1,280 cfs measured in that year. For all years of data, USGS included a data flag that states: “All or part of the record affected by Urbanization, Mining, Agricultural changes, Channelization, or other.” As discussed in Section 2.0, these changes may have some impact on flow frequencies throughout time, but it is likely minimized by the presence of stormwater detention for peak attenuation, which has been a requirement for nearly the entire period of record. Thus, this data flag does not present a major consideration for homogeneity.

Because there is a relatively long period of record, a flow frequency analysis was conducted using both the weighted skew and the station skew. A comparison of the results is presented in Table 1 and Figure 1. The results are nearly identical.

Table 1. Little Dry Creek at Westminster Weighted versus Station Skew

Recurrence Interval (years)	Flow (cfs) [Weighted Skew]	Flow (cfs) [Station Skew]
2	507	512
5	749	751
10	912	907
20	1,070	1,054
50	1,275	1,241
100	1,431	1,379
200	1,588	1,515
500	1,797	1,693
Skew Value	**-0.149**	**-0.257**

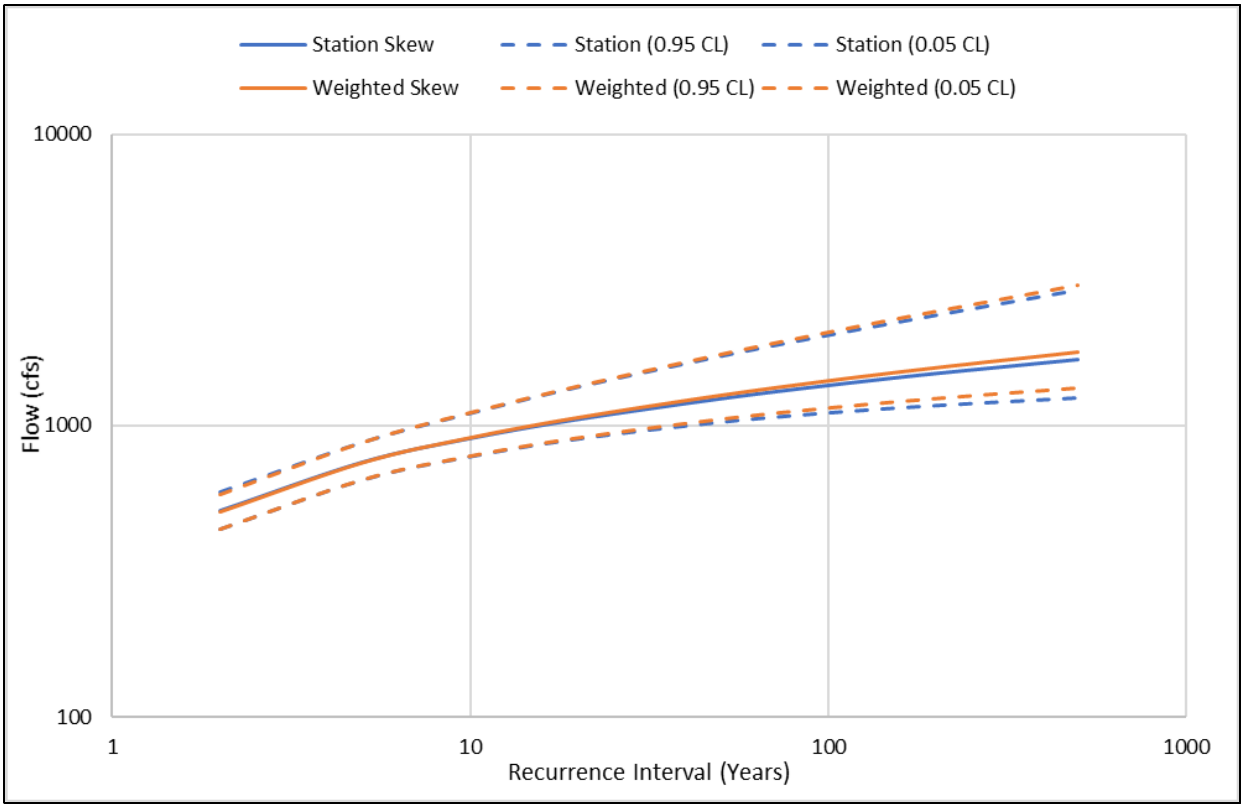

Figure 1. Comparison of Flow Frequency Curves and Confidence Limits Using Station Skew versus Weighted Skew

Because the period of record is 36 years and there was no significant difference between the weighted and station skews, the station skew was used to compute the final flow frequency curve. The final results are presented in Table 2 and Figure 2. Note that different horizontal scales are used in Figure 1 and Figure 2 (and later figures parallel to these). HEC-SSP output (which is used to generate Figure 2 and uses the red and green color scheme) uses a specialized probability scale that is unavailable in Excel.

Table 2. Little Dry Creek at Westminster Flow Frequency Values and Confidence Limits

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	512	443	590
5	751	653	873
10	907	786	1,098
20	1,054	903	1,352
50	1,241	1,031	1,723
100	1,379	1,109	2,035
200	1,515	1,175	2,381
500	1,693	1,248	2,904

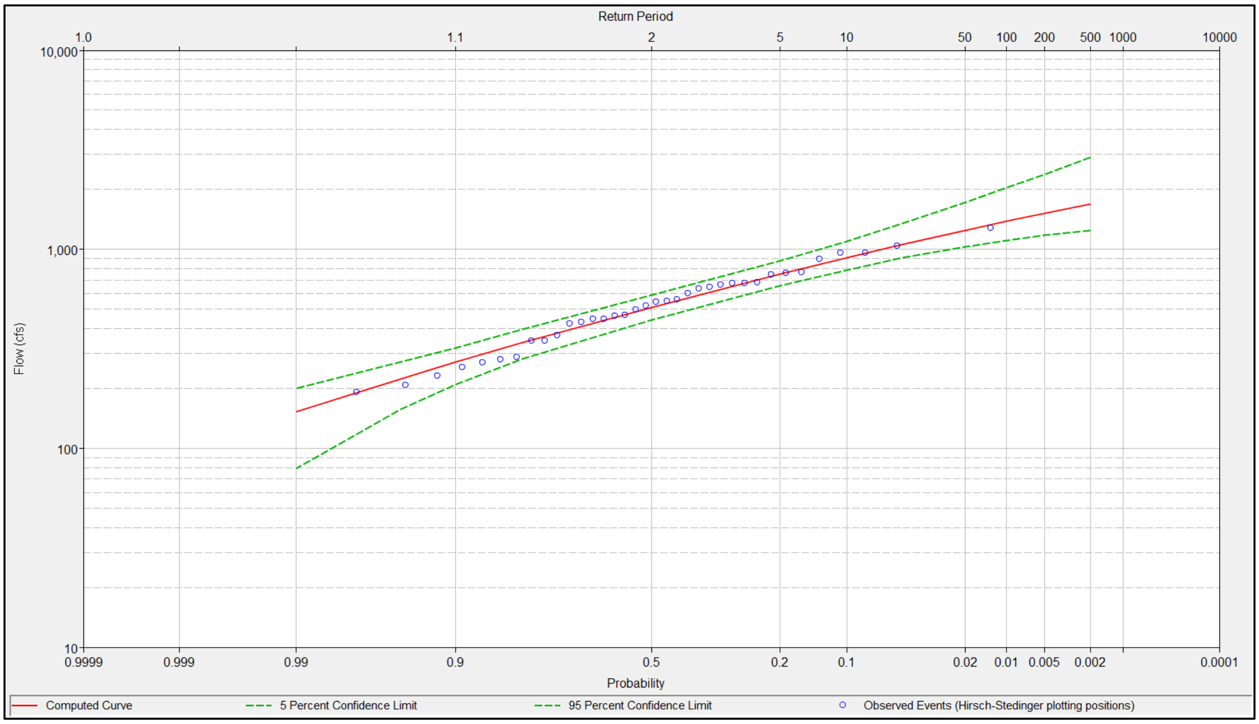

Figure 2. Little Dry Creek at Westminster Flow Frequency Curve

A comparison between the computed curve and the effective FIS (2018) peak discharges is presented in Figure 3. The FIS values were given for Little Dry Creek upstream of Federal Boulevard, which were scaled down based on the difference in watershed area between the FIS location and the gage. The FIS values are all greater than both the computed curve and the confidence limits. WWE does not recommend changing the FIS values. One factor potentially contributing to the lower calculated flows for each recurrence interval is inadvertent storage in the watershed and/or diversion of runoff by irrigation ditches, which would not be included in the FIS modeling. This inadvertent storage cannot be relied upon for future planning and development, and therefore the FIS values should not be decreased to match the calculated curve, which may be affected by past and current inadvertent storage.

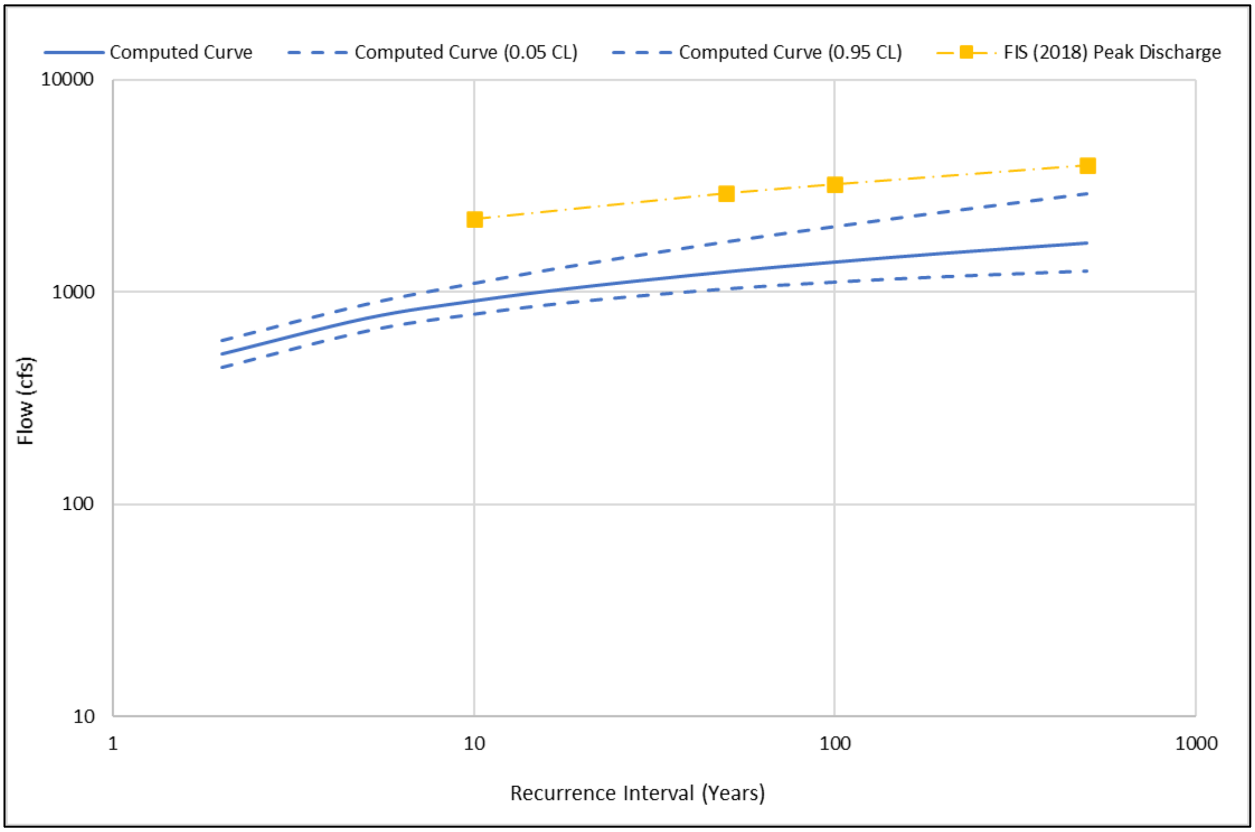

Figure 3. Little Dry Creek at Westminster Comparison of Computed Frequency Curve with Existing FIS

4.0 LITTLE DRY CREEK AT 64TH AVENUE

Little Dry Creek at 64th Avenue is MHFD ALERT Gage Number 1310 and has a continuous record from 2002 to 2019, or 18 years of data. No years of data were missing, and thus no perception thresholds were applied. A weighted skew was used, and WET did not identify any potential issues with the peak flow record. Therefore, the final results are presented in Table 3 and Figure 4.

It should be noted that a diversion is present approximately 0.3 miles upstream from the gage. This diversion splits flows and directs high flows from Little Dry Creek to a detention basin, at the inlet of which the gage is located. Therefore, the flows recorded at this gage and the associated flow frequency results are only a part of the full flows in Little Dry Creek just above the diversion point.

Table 3. Little Dry Creek at 64th Avenue Flow Frequency Values and Confidence Limits, Based on Bulletin 17C Gage Analysis

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	134	96	185
5	254	185	376
10	350	250	576
20	455	316	847
50	607	401	1,348
100	733	462	1,872
200	869	520	2,555
500	1,065	593	3,791

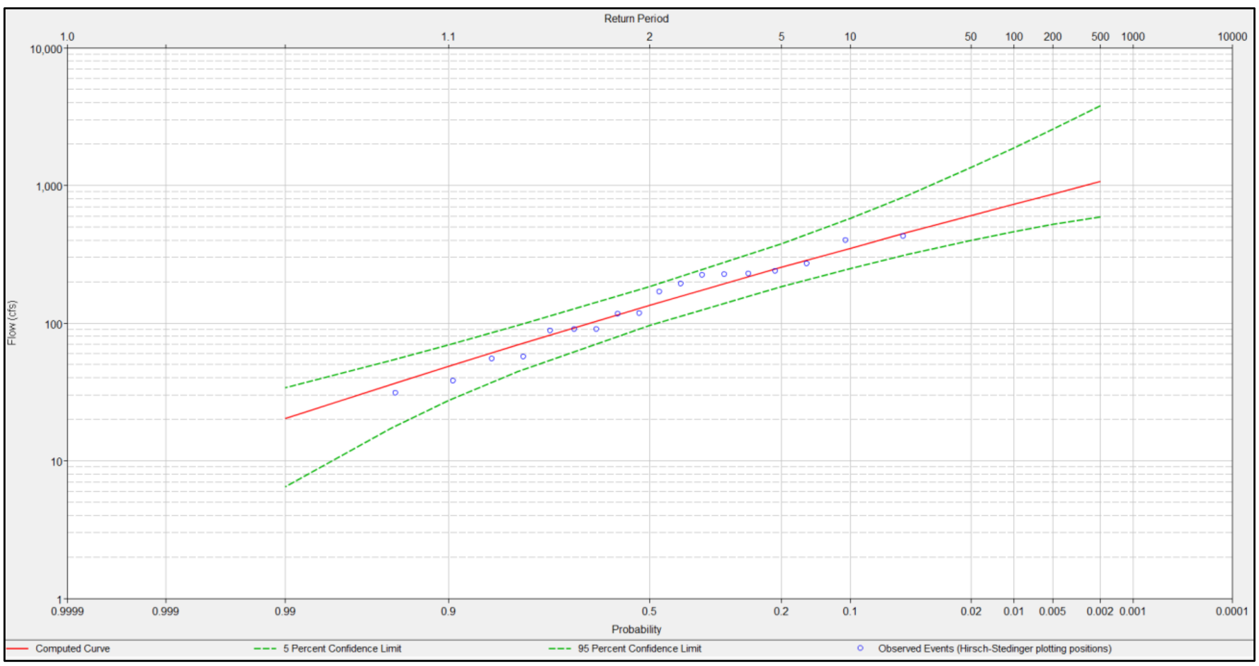

Figure 4. Little Dry Creek at 64th Avenue Flow Frequency Curve

A comparison between the computed curves for the Westminster gage and the 64th Avenue gage is presented in Table 4 and Figure 5. As discussed above, the computed flows at the Westminster gage are significantly higher than those computed for the 64th Avenue gage, due to the diversion occurring just upstream of the latter gage. The difference in computed flow frequencies is generally consistent for recurrence intervals above the 10-year event. Note that the logarithmic vertical scale in Figure 5 distorts this pattern.

Table 4. Comparison of Computed Frequency Curves for Little Dry Creek at Westminster and 64th Avenue

Recurrence Interval (years)	Flow (cfs) [Westminster Gage]	Flow (cfs) [64th Ave Gage]	Difference (cfs) [Westminster – 64th Ave]
2	512	134	378
5	751	254	497
10	907	350	557
20	1,054	455	599
50	1,241	607	634
100	1,379	733	646
200	1,515	869	646
500	1,693	1,065	628

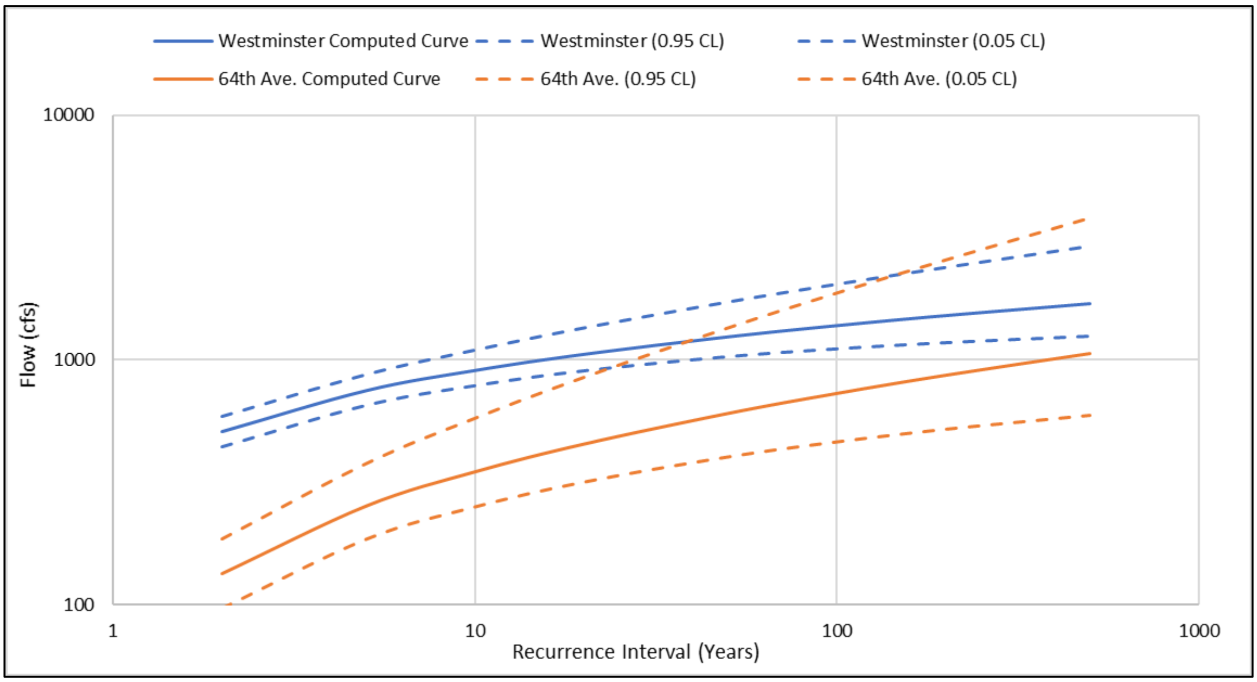

Figure 5. Comparison of Computed Frequency Curves for Little Dry Creek at Westminster and 64th Avenue

Given the issues with high flow diversions at the 64th Avenue gage, applying the results of the Westminster gage, adjusted for additional watershed area at the 64th Avenue gage, may be a more reliable estimate of the flow frequency relationship at this location than analysis of the gage data from 64th Avenue. The watershed area at the upstream diversion point is 13 mi^2. The ratio of watershed areas between the 64th Avenue gage diversion and the Westminster gage is 1.2. The Westminster gage computed curve and confidence limits were multiplied by this factor to determine the flow frequency values at the diversion point (0.3 miles upstream of the actual gage). The results are presented in

Table 5. These results likely present a more accurate representation of flow frequency on the lower section of Little Dry Creek than do those calculated by Bulletin 17C due to the diversion.

Table 5. Little Dry Creek at 64th Avenue Flow Frequency Values and Confidence Limits, Based on Scaling Upstream Gage

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	**0.05**
2	634	548	731
5	929	809	1,081
10	1,123	973	1,360
20	1,305	1,118	1,673
50	1,536	1,276	2,133
100	1,707	1,373	2,519
200	1,875	1,455	2,948
500	2,095	1,546	3,596

A comparison of the above flow frequency values (based on scaling the results of the Westminster gage) and the currently existing FIS peak discharge values (2018) is presented in Figure 6. The peak discharge values presented are for Little Dry Creek at Confluence with Clear Creek, which is located approximately 0.4 miles downstream of the diversion point. At the confluence with Clear Creek, there is a watershed area of 13.1 mi^2. The FIS values were scaled slightly to account for the difference in watershed area (13.1 versus 13 mi^2). Similar to the Westminster gage, the computed values scaled to the diversion point just above the 64th Avenue gage are below the FIS values. This may be caused by inadvertent storage or ditch diversions, and it is WWE’s recommendation that the FIS values not be adjusted.

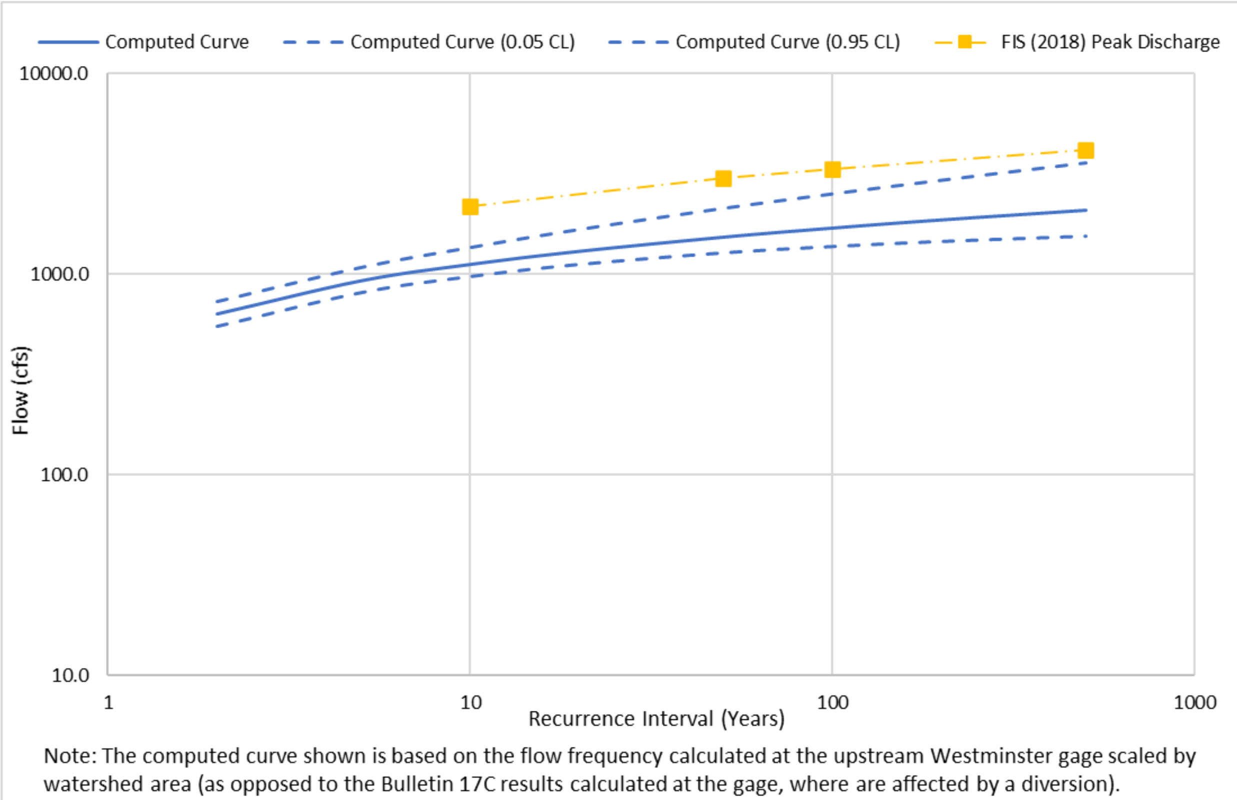

Figure 6. Little Dry Creek at 64th Avenue Comparison of Computed Frequency Curve with Existing FIS

5.0 SUMMARY/CONCLUSION

Flow frequency analysis using HEC-SSP and following Bulletin 17C procedure was conducted for two gages along Little Dry Creek. Annual peaks and potential data accuracy concerns were provided in the corresponding report by WET. The final analysis methods and final results are summarized below for each gage.

The Little Dry Creek at Westminster gage had 36 years of approved data for analysis, so a station skew was used to compute the flow frequency curve. The final results are summarized in Table 6.

Table 6. Little Dry Creek at Westminster Flow Frequency Values and Confidence Limits (same as Table 2)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	512	443	590
5	751	653	873
10	907	786	1,098
20	1,054	903	1,352
50	1,241	1,031	1,723
100	1,379	1,109	2,035
200	1,515	1,175	2,381
500	1,693	1,248	2,904

For the Little Dry Creek at 64th Ave, the entire period of record and a weighted skew were used for the Bulletin 17C analysis. However, the results of the Bulletin 17C analysis represent only the portion of high flows that was diverted off of Little Dry Creek at a point just upstream of the gage. Instead, the flow frequency results from the Westminster gage were scaled up to the diversion point on Little Dry Creek based on a comparison of watershed size. The resulting flow frequency estimates are summarized in Table 7. This is likely more representative of the Little Dry Creek flow frequency just above the confluence with Clear Creek than the estimate made using Bulletin 17C, which was based solely on diverted flows.

Table 7. Little Dry Creek at 64th Avenue Flow Frequency Values and Confidence Limits (same as Table 5)

Recurrence Interval (years)	Computed Curve (cfs)	Confidence Limits (cfs)	
		0.95	0.05
2	634	548	731
5	929	809	1,081
10	1,123	973	1,360
20	1,305	1,118	1,673
50	1,536	1,276	2,133
100	1,707	1,373	2,519
200	1,875	1,455	2,948
500	2,095	1,546	3,596

While the computed curves do not perfectly match the effective curves used in the 2018 FIS for Little Dry Creek, it is WWE's recommendation that no changes be made to the effective curve. The FIS values were higher than the curves computed using Bulletin 17C procedure, and using these FIS values is conservative for planning and design purposes.

As development in the watershed continues, and interactions between waterways change, it may be necessary to update these calculations accordingly. The results presented here represent calculations based on the best available data and may change as more years of data and additional gage information are collected.

cc:
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Kate Malers, WET, kmalers@water-and-earth.com

Attachment D. WET Gage Report for Van Bibber Creek

Van Bibber Creek Watershed Annual Peak Discharge Analysis

Summary:

Two stream level measurement points exist on Van Bibber Creek; one high in the watershed (330 Van Bibber at Hwy 93 for the majority of the period of record, recently relocated to Crestone St. as ALERT2 RADAR gage 10064), and 320 Van Bibber at the Sports Complex, near the Kipling Street crossing and about one mile of stream length above the confluence with Ralston Creek (Figure 1). Periods of record are 1991 and 1990 to the present, respectively. No USGS or DWR gages are sited on Van Bibber Creek. At the start of this Annual Peak Discharge Analysis, both MHFD gages presented some issues in terms of their suitability for providing flow estimates (and in some cases, stage records) that can be used for a statistical analysis of annual peak flows in the watershed. These issues have been addressed, as described here.

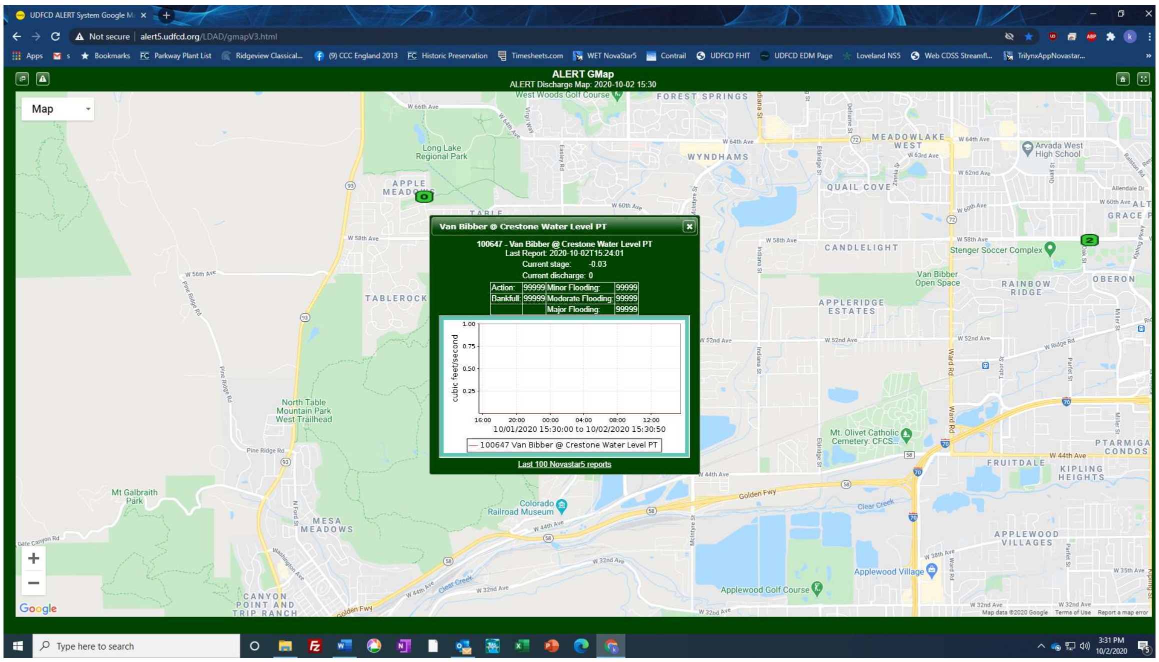


Figure 1. Location of ALERT Stations 330 Van Bibber Creek @ Crestone St. and 320 Van Bibber Creek @ Sports Complex from GMap. Prior to the 2018 flood season, the upper watershed gage was located immediately to the west, at Highway 93.

Annual peaks for these two Van Bibber gages do not frequently coincide; this occurred 8/24/1992 (39 cfs @ 93 and 53 cfs at Sports Complex, two hours later) and 5/8/2015 (132 cfs @93 and 286 cfs at Sports Complex; this time the downstream gage peaked more than 6 hours *before* the upstream gage). It may be that the annual peaks occurred the same day on 6/17/1993 as well (Sports Complex's 6/7/1993 peak is questionable; without it, the peak becomes 6/17/1993), with Sports Complex's peak of 45 cfs occurring an hour *before* 93's of 46 cfs. Peaks for 1997 and 2013 are a day apart. Variations in the spatial distribution of rainfall likely explain many of the instances in which annual peaks do not coincide or do not neatly represent a slug of peak flow moving downstream. However, there is another potential explanation in that Van Bibber Creek crosses several irrigation canals between the upstream and downstream gage locations, and it is known that during high flow events there can be/has been diversion of flows from Van Bibber into the canals (which convey the captured flows northward, out of the Van Bibber watershed). Additional information on this issue is provided later in this document. Much better correlation of annual peak flows occurs between the Van Bibber gage at the Sports Complex and the closest downstream gage, on Ralston Creek @ Carr (100). For most years, the annual peak flow at Sports Complex occurs on the same day, and often about 40 minutes before, the annual peak at Carr St. Therefore, some discussion of annual peaks at Carr St. is also included here.

Table 1 below shows the range of Qp values for various locations on Van Bibber and Ralston Creeks from the USGS StreamStats tool. Qps are calculated using three different methods (area-adjusted Mountain Region Peak Flow produces the low estimates; area-adjusted Foothills region Peak Flow 2016 produces the high estimate, and Area-Averaged values are always in between) for eight recurrence frequencies. The StreamStats reports are included in the Appendix.

Table 1. Range of Values from Streamstats for Selected Watershed Locaitons

Location	Drainage Area, miles^2	Q2 cfs	Q5 cfs	Q10 cfs	Q25 cfs	Q50 cfs	Q100 cfs	Q200 cfs	Q500 cfs
Van Bibber @ 93	9.4	58-71	88-186	109-305	136-507	166-698	190-937	211-1,210	253-1,640
Van Bibber @ Crestone	10.2	60-82	92-217	114-357	142-597	173-823	198-1,110	220-1,430	264-1,950
Van Bibber @ Sports Complex	17.5	75-178	115-494	146-832	181-1,420	222-1,980	258-2,680	289-3,510	345-4,820
Van Bibber @ Mouth	20.3	79-209	123-585	156-989	193-1,690	238-2,360	276-3,200	311-4,200	371-5,770
Ralston above Van Bibber Confluence	68.4	250-390	376-1,010	470-1,660	563-2,770	689-3,820	791-5,110	881-6,630	1,040-8,970
Ralston @ Carr St. Gage	89.1	292-479	440-1,240	552-2,050	660-3,420	808-4,720	931-6,340	1,040-8,220	1,220-11,100
Ralston @ Mouth	91.5	295-515	445-1,350	559-2,230	668-3,740	819-5,160	943-6,940	1,050-9,020	1,240-12,300

Gage 330 Van Bibber @ Hwy 93/10064 Van Bibber @ Crestone St.

The gage's period of record is from 5/16/1991 to the present. Gage 330 Van Bibber @ Hwy 93 was moved to Crestone St. (Figure 1) prior to the 2019 flood season because of significant long-standing issues with sedimentation at the Highway 93 location and for safety reasons.

This discussion will focus first on the Highway 93 location, and then describe the Crestone St. location.

From 5/16/1991 through the 2018 season, the upper Van Bibber gage was installed on the right wingwall upstream of the double-barrel concrete culvert under Highway 93. The riser pipe that housed the PT is still bolted to the wingwall. Because the channel bank is higher at the wingwall than at the thalweg, the stop bolt upon which the PT seats in the riser pipe sits higher than the adjacent channel invert, which varies with deposition/erosion. All stages reported for the PT represent elevation above the concrete culvert entrance just downstream of the PT cross section. The stop bolt for the PT at this gage was moved several times over the course of the gage's record; on 7/7/1992 it was lowered 6" from its initial installation location; on August 13, 2015 it was raised 1.08 ft because it had become silted in by deposition. These changes are reflected in the "reference level" values that are added to measured stages over the period of record.

The configuration of the channel reach also changed during the period of record. Specifically, the culvert was significantly altered when residential development downstream of the culvert occurred throughout 2013, and then the flood of record on September 13, 2013 also caused changes to the channel cross sectional geometry in the vicinity of the gage. It is not known precisely when the culvert alterations were initiated and completed, but it seems safe to assume that the changes had been made by the September high flow event (evidenced by the very heavy deposition upstream of and inside the culvert that is currently visible). Construction began after the close of the 2012 flood season and was well underway by August 2013, based upon dated historic photographs from GoogleEarth (Figures 2 and 3).

An initial depth-discharge rating for the gage was based upon hydraulics at the downstream culvert entrance. However, it provided poor estimates of discharge because it did not account for the effect of significant roughness elements in the culvert (concrete energy dissipation baffles, now buried under gravel and cobble deposition) on low to moderate flows. WET was contracted to replace the initial rating in 2006 and the site was surveyed for that rating development on January 23, 2006. However in 2012, in the course of E-19 development work to determine “critical stages” for alarms at ALERT gages, it was discovered that the rating developed in 2006 was never implemented. The 2006 rating was implemented on 7/27/2012 and used to back-calculate discharges for the entire record; the initial rating is retained in NS5 as an historical archive but was not used to calculate any flows in the record.

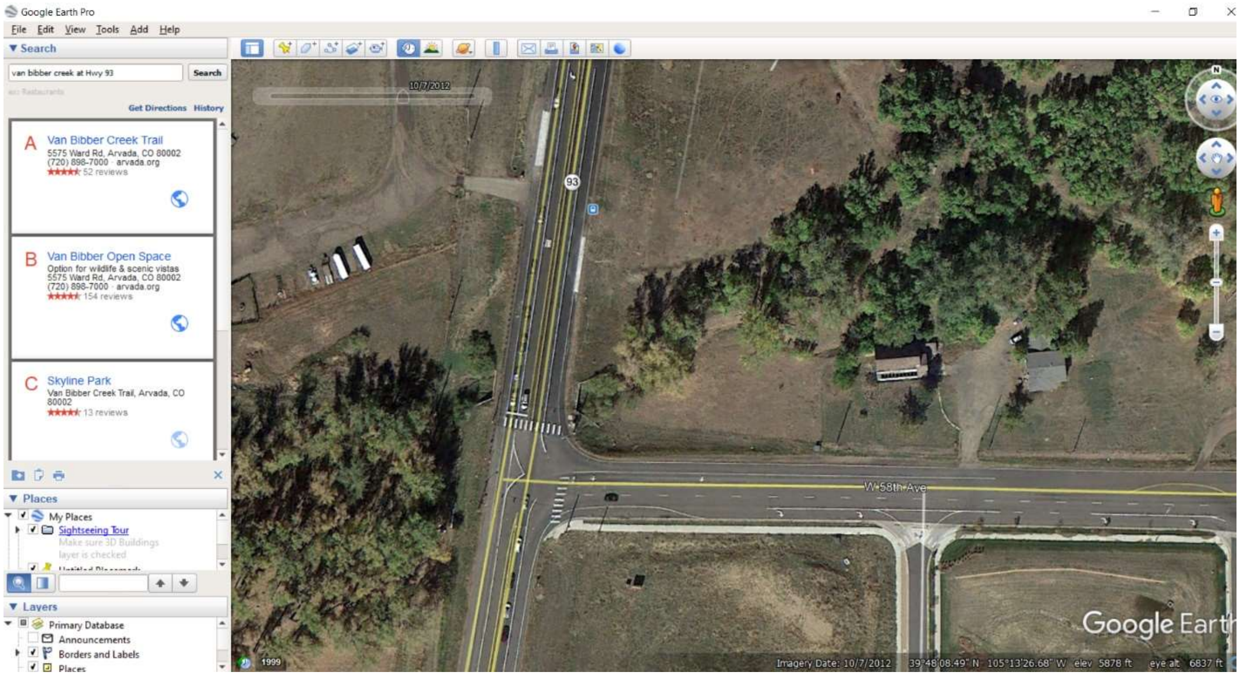


Figure 2. Google Earth image of the 330 Van Bibber @ Hwy 93 Culvert, 10/7/2012. Last available image of the pre-development culvert configuration.

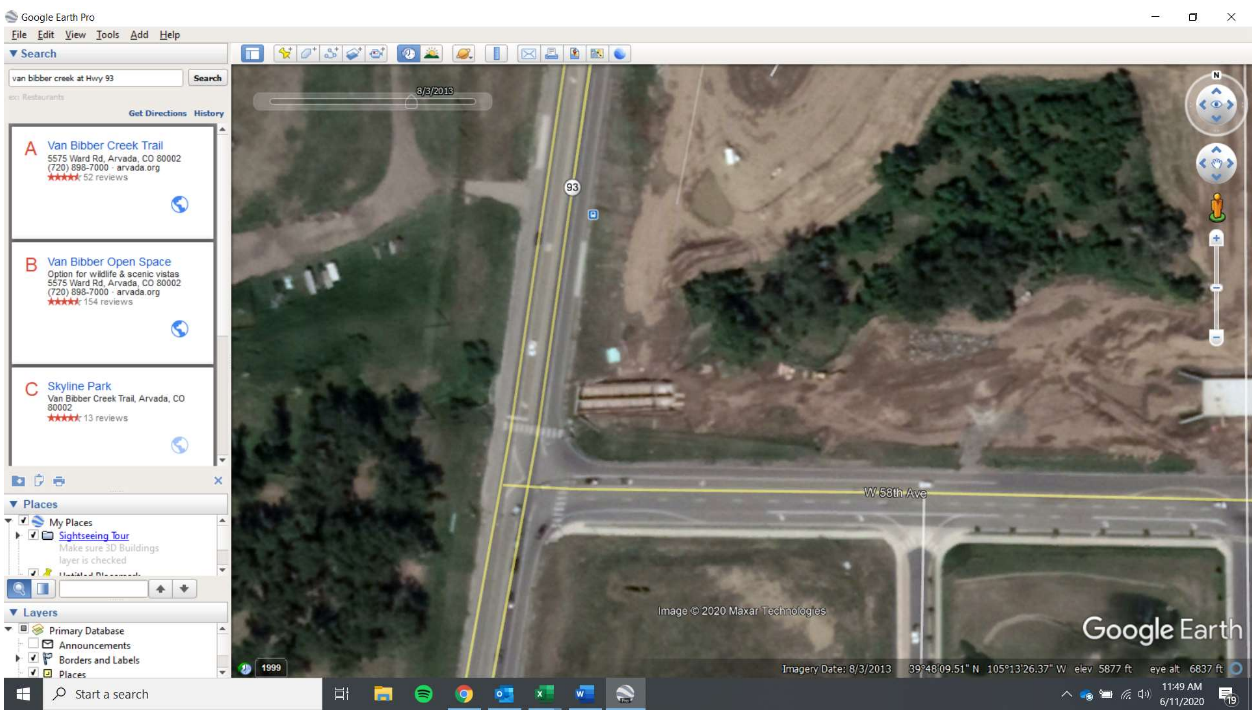


Figure 3. First available Google Earth Image of the culvert reconstruction; 8/3/2013

The likelihood that the culvert alterations during 2013 potentially rendered the 2006 gage rating obsolete was not recognized until 2015, likely because the alterations affected the downstream end of the culvert and it was assumed that backwater at the gage would be under inlet control. By then there were already plans in motion to re-locate the gage, and a new rating was not undertaken.

The 2013 construction elongated the culvert and realigned the channel downstream of the culvert without actively disturbing the gage site upstream of the culvert. However, the post-development, much longer culvert includes 2.5-foot-high weir walls near the exit of the culvert in both culvert barrels. Figure 4 shows the wall in the southern barrel (and the substantial deposition behind it). Orifice openings in the north wall of each barrel divert low flow into a remnant reach of the original channel alignment, likely to maintain a small gallery of cottonwood trees that were not removed with development. Higher flows would overtop the weir walls in the culvert and then exit the culvert over an armored drop structure/detention pond.

Figure 4. Photograph of the diversion wall inside the reconstructed culvert downstream of gage 330 at Hwy 93.

To determine if the weir walls would provide hydraulic control of depths at the gage site, in 2020 the weir walls (and two upstream cross sections) were surveyed relative to the gage datum. The 2006 HEC-RAS model was altered to reflect the culvert extension and weir walls. This analysis produced the 2020A rating. Which reflects the reconstruction of the culvert but NOT the deposition that was surveyed in 2020. A model utilizing the current, 2020 cross sections at the culvert entrance and at the monitored cross section, with the reconstructed culvert/weir walls downstream, was used to develop the 2020B rating. Deposition at the culvert entrance was determined to be approximately 0.6 ft deep. The monitored cross section is also significantly different than in 2006, with a narrow, deep low flow channel at river right and the entrance to the left barrel of the culvert entirely inaccessible except at high flows. The deposition that exists in the culvert behind the wing walls and extends upstream to the gaged cross section has changed the relationship between water level measured at the gage and discharge very substantially from either the 2020A rating or the 2006 rating, and would be expected to apply if the gage were in operation now. However, some judgement is required in determining the period of record for which each of the 2020 ratings should be used.

It is not possible to determine from the GoogleEarth satellite images if the weir walls were in place during the flood event of September 12-13, 2013 (since the weir walls are inside the culvert). It seems likeliest that they were, because the significant deposition behind the weir walls in the culvert includes a significant proportion of larger particle size sediments including both gravels and cobbles that were most likely moved by a more significant storm event. It is likely that the weir walls were in place to exacerbate deposition from the September 12-13 flow events, and that most of the deposition now visible happened during that single 2013 event. In fact photographs of the channel shot in 2015 look very similar to what is present in 2020.

It is difficult to recommend a single peak flow value for the 2013 flow event that is the annual peak for 2013 as well as the flow of record for the gage, because that event likely changed the relationship between water surface at the gage and discharge in the channel as the flow event occurred. The solidification of that deposition may not have preceded or coincided with the peak flow. Material was likely being actively distributed over the course of the flow event. Peak flow for the 2013 event was estimated using all three ratings. Assuming the weir walls were NOT in place and the 2006 rating best describes the relationship between water level and discharge on Sept. 12, 2013, the estimated flow would be 593 cfs. Since it is likely that the weir walls were in place, this is an overestimate.

Assuming that the weir walls in the culvert were in place, but the upstream cross sections remained as they were surveyed in 2016, then the 2013 peak flow would be estimated at associated heavy deposition, now cemented and vegetated, was not yet in place, the 2013 peak flow would be estimated at 394 cfs. With the current, cemented and vegetated deposition, the discharge for the 2013 measured water level would be just 272 cfs. These latter values are proposed as bracketing values for the estimated flow for the 2013 event.

The annual peaks for years 2014-2018 are potentially unrealistically high (average annual peak of 187 cfs for those five years, compared to an annual average peak of only 56 cfs for the 21 year 1991-2012, excluding 2006). Confidence in the estimates for 2014-2018 are impacted by two issues described next.

Discharge estimates for 2014 and most of 2015 may be impacted by inaccurate *stage* measurement because sedimentation in the gage riser pipe itself was hampering hydraulic response for the column of water in the riser pipe that is measured by the gage PT. On August 13, 2015, the gage installation was altered to address sedimentation/siltation issues at the gage. In an 8/14, 2015 email from Scott Bores at OneRain, it was described that, “the riser was heavily silted in with mud and debris. Our team had to dig out the bottom of the riser and cut the bottom off to free sediment. Approximately one foot of sediment is encased around where the bottom of the riser once rested. In order to secure hydraulic contact a new stop bolt was placed 1.08 feet above where the original was located.” The sedimentation problem at the riser pipe, like the sedimentation in the culvert and channel downstream of the gage, likely occurred during the 2013 high flow event.

After the stop bolt was raised on 8/13/2015, the PT rested considerably higher than the channel invert and the culvert entrance. Water level measurements after this date would not have been impacted by the hydraulic connectivity issue anymore, but two other issues cause uncertainty about the reliability of these discharge estimates. First, the expected result of raising the PT stop bolt another foot+ higher than the channel invert would be to have many records reflecting a water level too low to register any depth over the higher PT, i.e. with a numeric value equal to the reference level of 1.935 ft less the calibration offset (-.245) = 1.69 ft. Instead, the minimum value observed in any of the years after the gage alteration is 1.70 ft stage. Sometimes PTs are known to produce anomalous readings in hot, dry locations, and there may still have been issues with the water in the riser pipe tracking correctly with the water level in the stream. The whole that was dug to free the bottom of the riser pipe remains as a feature of the channel bottom, so that the riser pipe sits behind and below a ridge of material that separates it from the active channel. Finally, after the stop bolt was raised, it does not appear that any new holes were drilled in the riser pipe to allow water to enter. Although the base of the riser pipe was left open to allow water to enter and exit, there is no redundant path for water to move freely in the riser pipe.

Since 2019, the new gage location is on the upstream side of the bridge carrying Crestone St. over Van Bibber Creek. The rating was developed based upon a reach and cross section survey and theoretical hydraulic modeling, calibrated with a single instantaneous (low flow) discharge measurement with associated water surface elevation. Downstream of the bridge, the channel has a lot of capacity-reducing components including a sinuous path around a significant left-bank berm, heavy vegetation and other channel and overbank roughness elements including a fence, an elevated utility crossing and a pedestrian footbridge. In the development of the rating it was assumed that these conditions downstream of the bridge may be more significant in developing backwater upstream of the bridge, at the gage, than the bridge itself. The single year of existing data does not provide a lot of insight (2019 was a relatively dry year in many front range watersheds and produced few new high water records), but hopefully the rating development effort reflects these channel characteristics well enough to provide reasonable estimates of discharge going forward.

Annual peaks (for the two gage locations high on Van Bibber Creek) are shown in Table 1. Annual peaks for the 22-year period of record from 1991-2012 are most likely useable to analyze recurrence. Peaks for the period 2013-2018 are questionable because they do not reflect post-2013 deposition and due to stage measurement issues. The shaded portion of the flow table reflects values using the 2019 rating for the new location at Crestone St.

Table 1. Van Bibber @ 93/Crestone Annual Peak Flows

Year	Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)
1991	May 16	1.99	117
1992	August 24	1.31	39
1993	June 17	1.4	46
1994	August 11	1.04	18
1995	May 18	2.12	134
1996	May 26	0.91	12
1997	August 5	1.23	32
1998	August 3	1.5	55
1999	May 01	1.77	88
2000	July 16	1.95	112
2001	July 13	1.5	55
2002	May 24	1.14	25
2003	July 29	2.86	252[1]
2004	June 27	1.59	65
2005	April 12	1.23	32
2006	Missing	missing	missing
2007	April 24 and May 14	1.14	25
2008	August 7 and August 8	0.70	10
2009	June 02	1.02	17
2010	April 30	1.24	32
2011	June 20	0.80	10
2012	July 9, July 25 and July 26	0.7	10
2013	September 13	3.43	272-394[2]
2014	July 7	2.72	134[3]
2015	May 8	2.70	132[3]
2016	April 17	3.46	278[4]
2017	May 08	3.54	292[4]
2018	May 04	2.48	100[4]
2019	July 21	1.64	70[5]

[1] Gage record high water from the data period prior to the culvert reconstruction with weir walls

[2] 2013 saw both the culvert reconstruction work AND the record high water/channel forming event on September 13. Peak flow is given as a range representing estimates of the minimum and maximum peak flow.

[3] 2014 and May, 2015 peak flows from the period of time when the riser pipe was encased in deposition from the 2013 flow event; creating concern that the riser pipe was not hydraulically connected to the stream.

[4] Stages recorded after the August 13, 2015 work to raise the stop bolt appear unreasonably high, for both low flows and for peak flows, for years 2016-2018

[5] New gage site; 10064 at Crestone St.

Gage 320 Van Bibber @ Sports Complex

Currently the Sports Complex gage on Van Bibber lies about 0.6 miles upstream of the entrance to the 1,600 foot-long, double 14.5 by 8.5 RCB culvert that carries Van Bibber Creek beneath the Arvada Plaza downstream from Kipling Street. Its present location is upstream from a grouted concrete drop structure in a trapezoidal channel reach which was built in 2006 as part of the Arvada Van Bibber Creek Flood Protection project (Figure 6). The current location is just a short distance downstream of the original gage location, which began logging water level data on 04/02/1990. At that time, the gage was located 53 feet upstream of the apron which directed flow into triple RCB culverts that used to carry Van Bibber Creek northward under W. 58th Avenue (now the reconstructed channel keeps flows on the south side of 58th Avenue).

In 2018 it was realized that a new rating was not attached to the stage data from the gage in its new location/channel configuration. Rating development work was completed late in 2018 and the resulting stage-discharge rating was used to back-calculate flow estimates to 2006. Therefore, there are two discrete periods in the record that reflect different (but nearby) gage locations in very different physical arrangements and with separate ratings; the period from 1990-2005 and from 2007 to the present (the gage was out of service for the entirely of 2006 due to the channel realignment construction, and also produced insufficient data to determine an annual peak in 2012). The periods of record under each rating are 16 years and 12 years (omitting 2012).

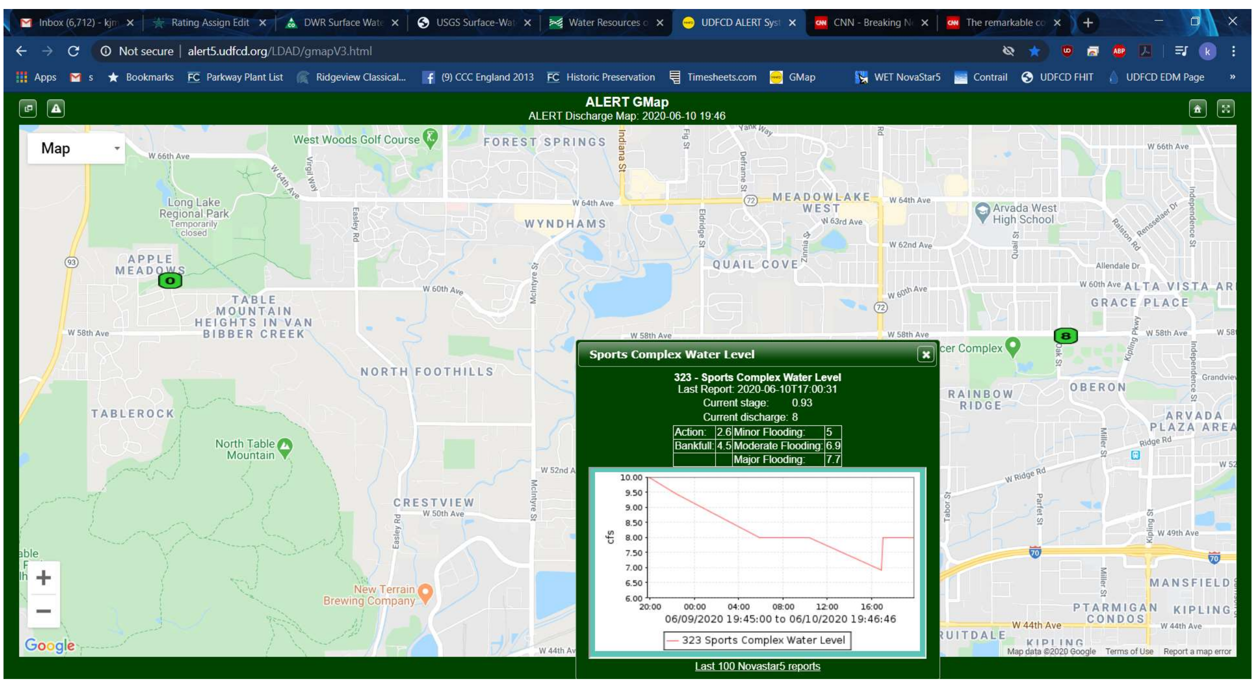


Figure 6. Location of ALERT Station 320 Van Bibber Creek @ Sports Complex from GMap.

Prior to the post-2006 rerating effort, and based on the relatively narrow range of pre-2006 annual peaks for the gage, it seemed likely that the high flows detected at this gage were very frequently or even annually truncated due to the interception of flood flows in Van Bibber Creek at canal crossings upstream of this gage. The Church Ditch, Farmer's High Line Canal, Juchem Ditch and Croke Canal may all have the potential to reroute stormwater in Arvada creeks, conveying it northward across basin boundaries and in some cases releasing it in low capacity ditch reaches. Most certainly it is expected that the Croke Canal intercepts flood flows from Van Bibber Creek upstream of the Sports Complex gage during some high flow events (particularly in 2013). The 4x4 siphon which conveys Van Bibber Creek beneath the Croke Canal is shown in Figure 7. Flows that exceed its capacity are likely diverted northward in the Croke Canal rather than continuing eastward in Van Bibber Creek

Figure 7. Van Bibber Creek is conveyed under the Croke Canal upstream of 320 Sports Complex

However, the new rating for the post-2006 period produced many annual peaks that likely do not exceed the capacity of the siphon and suggested that the pre-2006 rating may have overestimated flows in the range of the annual peaks. An analysis of the pre-2006 rating and existing survey data for the pre-2006 gage and channel configuration lent credence to this hypothesis.

The shape of the original pre-2006 rating curve includes a zone where a single rating ordinate breaks the expected shape of the curve and causes flow estimates to rapidly increase for small increases in stage;

nearly all of the pre-2006 annual peaks are estimated by this portion of the rating curve. The pre-2006 curve was based upon HW/D inlet control analysis of the triple RCB culverts assuming that would produce backwater upstream of those culverts. However, it was also recognized that a higher "lip" at the upstream end of the concrete apron leading to the culverts would control low flows, and the low flow portion of the rating was adjusted to try to recognize the apron's initial hydraulic control over backwater at the gage. Somehow this tinkering at the low end resulted in the unusual divot shape in the rating curve.

The pre-2006 rating has been adjusted using existing information about the reach's geometry and hydraulic characteristics. A survey-based rating development for the pre-2006 channel and gage configuration is impossible now, as those physical features are gone. However, information on the development of the pre-2006 rating curve included two sources of relevant data: an October 28, 1994 field survey conducted by UDFCD personnel with the express intention characterizing the concrete apron "cutoff" elevation that provided low flow control of backwater on the gage, AND a March 10, 2000 reach- and cross-section survey undertaken by HDR Engineering, Inc. under contract to UDFCD to determine NWS "E-19" style critical stages for the gage location. The former provided distances between the relevant cross sections and the downstream cross-section's approximate geometry (because this cross section is concrete lined, flow line elevations can be assumed to vary only slightly across the width of each culvert, etc.); the latter provided a hydraulic slope for the reach and the upstream cross-section's (natural) geometry in more detail. Reducing the survey data to a consistent vertical datum was not without some uncertainty, but a simple HEC-RAS backwater model representing the low flow control regime produced a curve in the critical range of the rating that exhibits a more typical shape than the original pre-2006 rating curve. The pre-2006 flows were backfilled using the revised rating, as shown in Table 2.

Table 2. Van Bibber @ Sports Complex Annual Peak Flows

Year	Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)
1990	July 8	3.60	174
1991	July 22	3.70	193[1]
1992	August 24	2.80	53
1993	June 7	2.80	53
1994	June 1	2.50	33
1995	July 15	2.80	53
1996	August 26	2.72	47
1997	August 4	2.81	54
1998	August 7	2.94	64
1999	May 20	3.34	123
2000	July 17	3.54	162
2001	May 4	3.38	131
2002	August 5	2.86	58
2003	April 19	3.25	106
2004	June 27	3.28	112
2005	August 4	2.36	25
2006	Out of service		
2007	April 24	2.19	97

2008	August 16	1.75	50
2009	July 20	2.35	118
2010	July 7	1.94	67
2011	July 7	2.42	128
2012	July 7	1.89	172
2013	September 12	2.54	146
2014	May 30	1.98	73
2015	May 8	3.28	286[2]
2016	June 28	2.08	84
2017	July 26	2.06	81
2018	September 5	2.96	221
2019	July 5	2.30	112

[1] Gage record high water from the pre-2006 data period

[2] Gage record high water from the post-2006 data period

Gage 100 Ralston Creek @ Carr Street

The Carr Street gage's period of record is from 1988 to the present, with no data for 2012 (due to redevelopment of the park including realignment of the stream channel with subsequent re-installation of the gage). Carr Street flows have been estimated by an original rating and then by a 2014 rating developed from a reach- and cross-section survey and HEC-RAS model by WET. The 2014 rating has produced a reduction in the average annual peak flow at Carr St., which has not been uncommon as new ratings have been required for established gage locations. However, in this case the annual peaks, plotted as a rating, show no discontinuity between the two ratings. For the pre-2014 period, the average annual peak at Carr Street was 1,075 cfs. For the period so far since 2014, it has been 523 cfs (the latter figure represents the average of a relatively brief period of record).

Table 3. Ralston Creek @ Carr Street Annual Peak Flows

Year	Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)
1988	August 4	24.11	870
1989	June 3	24.42	996
1990	July 8	24.80	1,249
1991	July 22	27.20	3,010[1]
1992	August 24	23.40	588
1993	June 17	23.80	756
1994	April 25	24.10	882
1995	July 15	24.90	1,315
1996	August 26	25.24	1,536
1997	August 4	24.86	1,291
1998	October 16	24.70	1,167
1999	May 20	26.40	2,300
2000	July 17	25.20	1,539
2001	July 13	24.40	1,020
2002	August 5	23.10	460
2003	April 19	23.20	490
2004	June 8	24.40	979

2005	August 4	22.60	298
2006	July 4	24.36	976
2007	April 24	23.70	677
2008	August 16	23.42	581
2009	June 26	24.22	918
2010	April 23	24.26	934
2011	May 18	24.94	1,340
2012	Missing	Missing	Missing
2013	September 13	23.71	703
2014	July 8	22.49	250
2015	May 8	24.38	941[2]
2016	June 28	23.65	582
2017	June 8	22.45	240
2018	September 5	22.44	238
2019	July 13	22.16	184

[1] Peak Flow for the 1988-2011 period (in the Original stream and gage configuration and first rating)

[2] Peak Flow for the post-2012 period (since the channel realignment and gage reinstallation)

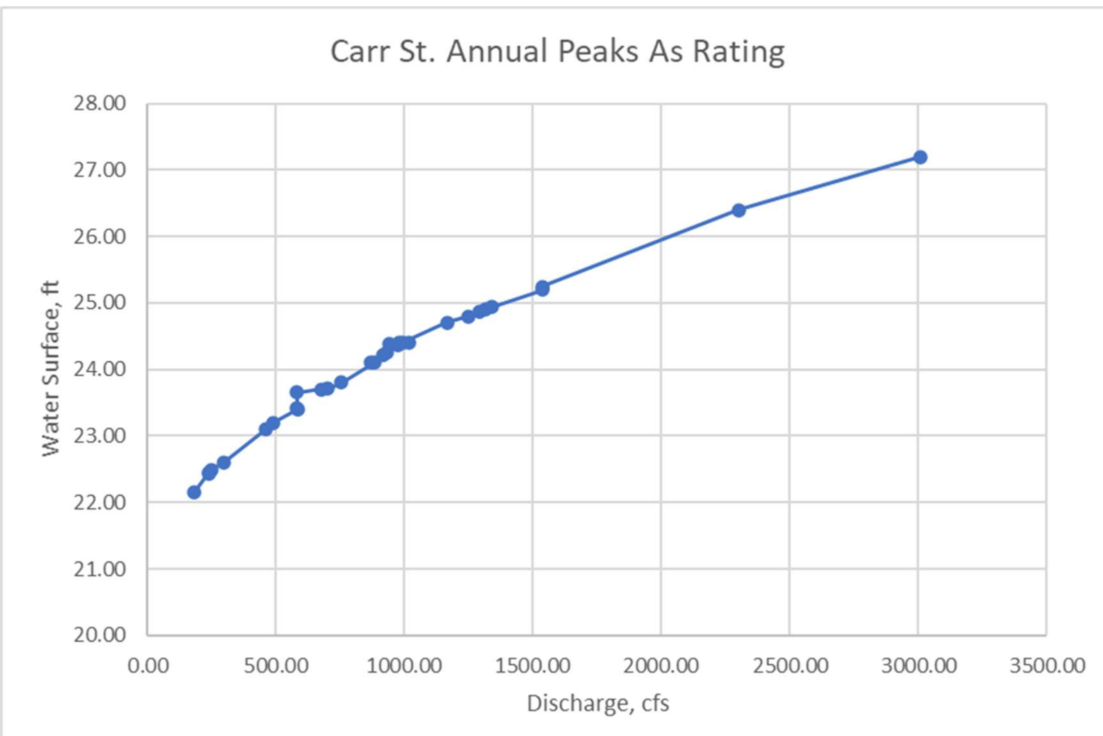


Figure 9. Ralston Creek annual peaks plotted as a rating. Differences between the pre-2014 channel reconstruction and the post-2014 channel reconstruction are not readily observable.

Recommendations/Concerns

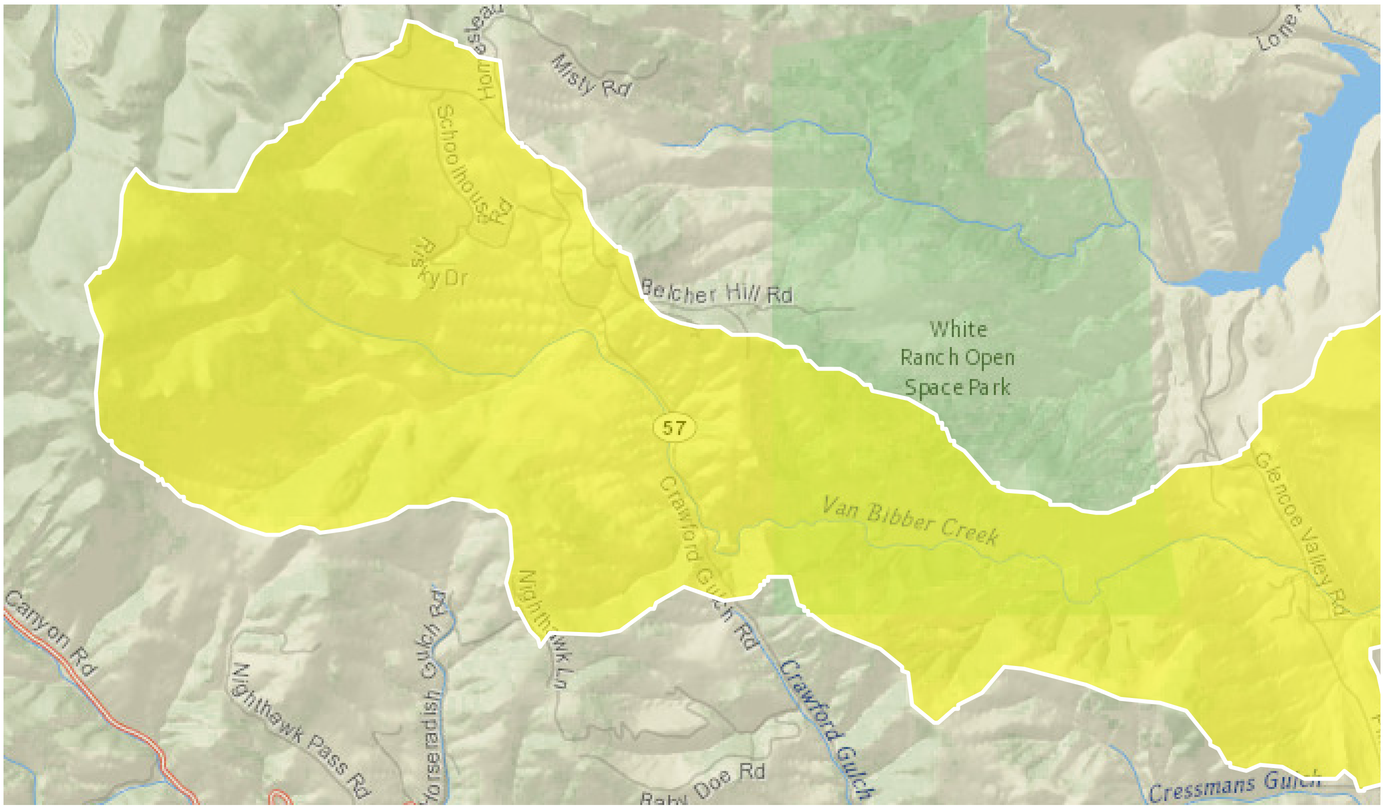
To evaluate the record at 320 Van Bibber @ Sports Complex, it might be helpful to estimate the capacity of the siphon under the Croke Canal upstream of the Sports Complex gage (and, potentially, of other canal crossings). If those structures do limit the peak flows that will be detected at Sports Complex by diverting flows northward, it would be useful to know at what flow that might occur.

At both the Ralston @ Carr Street and Van Bibber @ Sports Complex gages, catching a higher-than-baseflow discharge event with an instantaneous discharge measurement/water surface elevation survey, and using that empirical data point to assess the theoretical rating results, would be helpful.

Appendix. Streamstats Reports

StreamStats Report

Region ID: CO
Workspace ID: CO20200708183513135000
Clicked Point (Latitude, Longitude): 39.80216, -105.22469
Time: 2020-07-08 12:35:33 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	9.4	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	29	percent
PRECIP	Mean Annual Precipitation	21.68	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.14	inches
STATSCLAY	Percentage of clay soils from STATSGO	17.98	percent

Parameter Code	Parameter Description	Value	Unit
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5874	feet

Peak-Flow Statistics Parameters[55 Percent (5.22 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	9.4	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	29	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	21.68	inches	18	47

Peak-Flow Statistics Parameters[45 Percent (4.19 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	9.4	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.14	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	17.98	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5874	feet	4290	8270

Peak-Flow Statistics Flow Report[55 Percent (5.22 square miles) Mountain Region Peak Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	58	ft^3/s	49
5 Year Peak Flood	88.1	ft^3/s	44
10 Year Peak Flood	109	ft^3/s	41
25 Year Peak Flood	136	ft^3/s	40
50 Year Peak Flood	166	ft^3/s	39
100 Year Peak Flood	190	ft^3/s	36

Statistic	Value	Unit	SEp
200 Year Peak Flood	211	ft^3/s	36
500 Year Peak Flood	253	ft^3/s	33

Peak-Flow Statistics Flow Report[45 Percent (4.19 square miles) Foothills Region Peak Flow 2016 5099]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	71	ft^3/s	117
5 Year Peak Flood	186	ft^3/s	87
10 Year Peak Flood	305	ft^3/s	80
25 Year Peak Flood	507	ft^3/s	80
50 Year Peak Flood	698	ft^3/s	83
100 Year Peak Flood	937	ft^3/s	88
200 Year Peak Flood	1210	ft^3/s	94
500 Year Peak Flood	1640	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	63.8	ft^3/s
5 Year Peak Flood	132	ft^3/s
10 Year Peak Flood	196	ft^3/s
25 Year Peak Flood	302	ft^3/s
50 Year Peak Flood	403	ft^3/s
100 Year Peak Flood	523	ft^3/s
200 Year Peak Flood	657	ft^3/s
500 Year Peak Flood	871	ft^3/s

Peak-Flow Statistics Citations

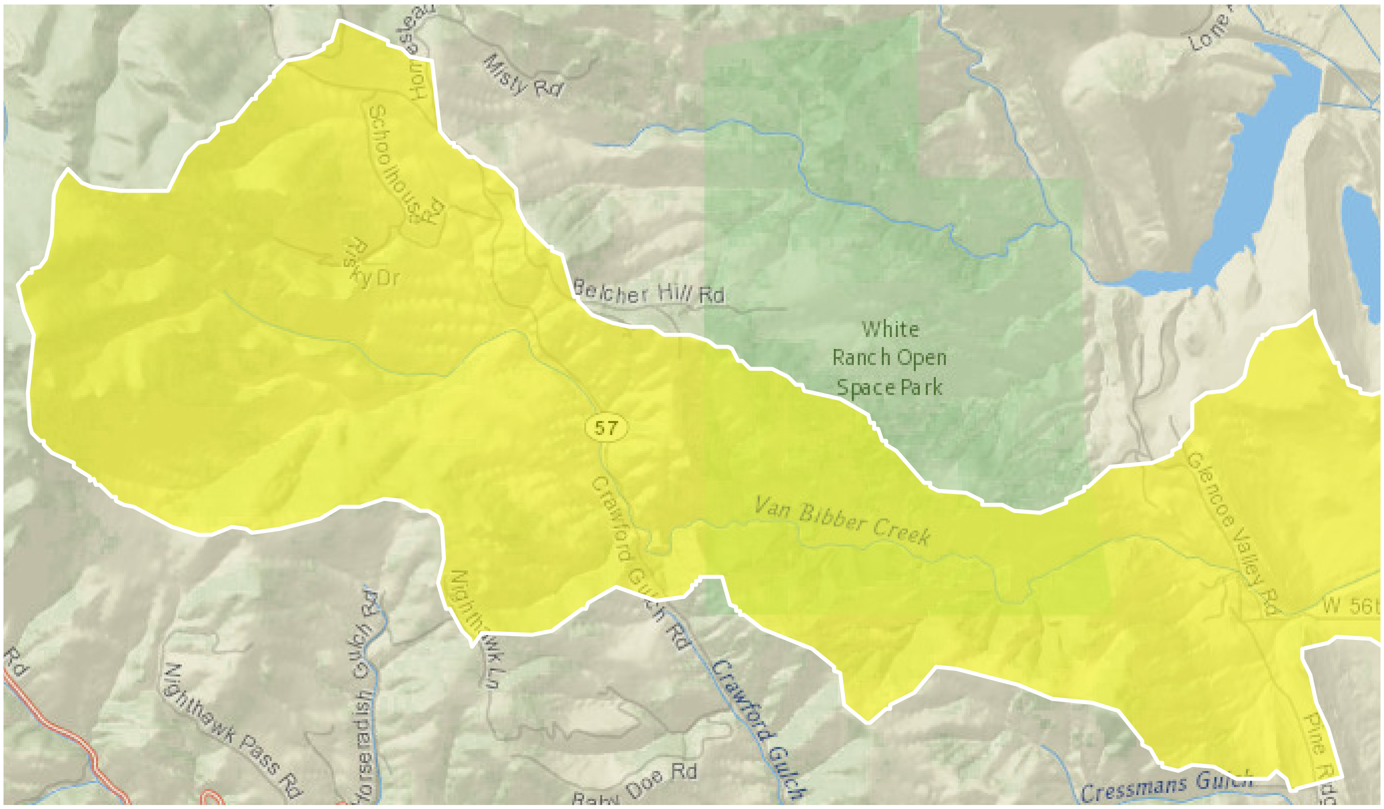
Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

StreamStats Report

Region ID: CO
Workspace ID: CO20200708191804122000
Clicked Point (Latitude, Longitude): 39.80628, -105.21295
Time: 2020-07-08 13:18:22 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	10.2	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	28	percent
PRECIP	Mean Annual Precipitation	21.46	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.15	inches
STATSCLAY	Percentage of clay soils from STATSGO	19.18	percent

Parameter Code	Parameter Description	Value	Unit
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5803	feet

Peak-Flow Statistics Parameters[51 Percent (5.22 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	10.2	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	28	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	21.46	inches	18	47

Peak-Flow Statistics Parameters[49 Percent (4.95 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	10.2	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.15	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	19.18	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5803	feet	4290	8270

Peak-Flow Statistics Flow Report[51 Percent (5.22 square miles) Mountain Region Peak Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	60.1	ft^3/s	49
5 Year Peak Flood	91.6	ft^3/s	44
10 Year Peak Flood	114	ft^3/s	41
25 Year Peak Flood	142	ft^3/s	40
50 Year Peak Flood	173	ft^3/s	39
100 Year Peak Flood	198	ft^3/s	36

Statistic	Value	Unit	SEp
200 Year Peak Flood	220	ft^3/s	36
500 Year Peak Flood	264	ft^3/s	33

Peak-Flow Statistics Flow Report[49 Percent (4.95 square miles) Foothills Region Peak Flow 2016 5099]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	82	ft^3/s	117
5 Year Peak Flood	217	ft^3/s	87
10 Year Peak Flood	357	ft^3/s	80
25 Year Peak Flood	597	ft^3/s	80
50 Year Peak Flood	823	ft^3/s	83
100 Year Peak Flood	1110	ft^3/s	88
200 Year Peak Flood	1430	ft^3/s	94
500 Year Peak Flood	1950	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	70.8	ft^3/s
5 Year Peak Flood	153	ft^3/s
10 Year Peak Flood	232	ft^3/s
25 Year Peak Flood	364	ft^3/s
50 Year Peak Flood	490	ft^3/s
100 Year Peak Flood	641	ft^3/s
200 Year Peak Flood	811	ft^3/s
500 Year Peak Flood	1080	ft^3/s

Peak-Flow Statistics Citations

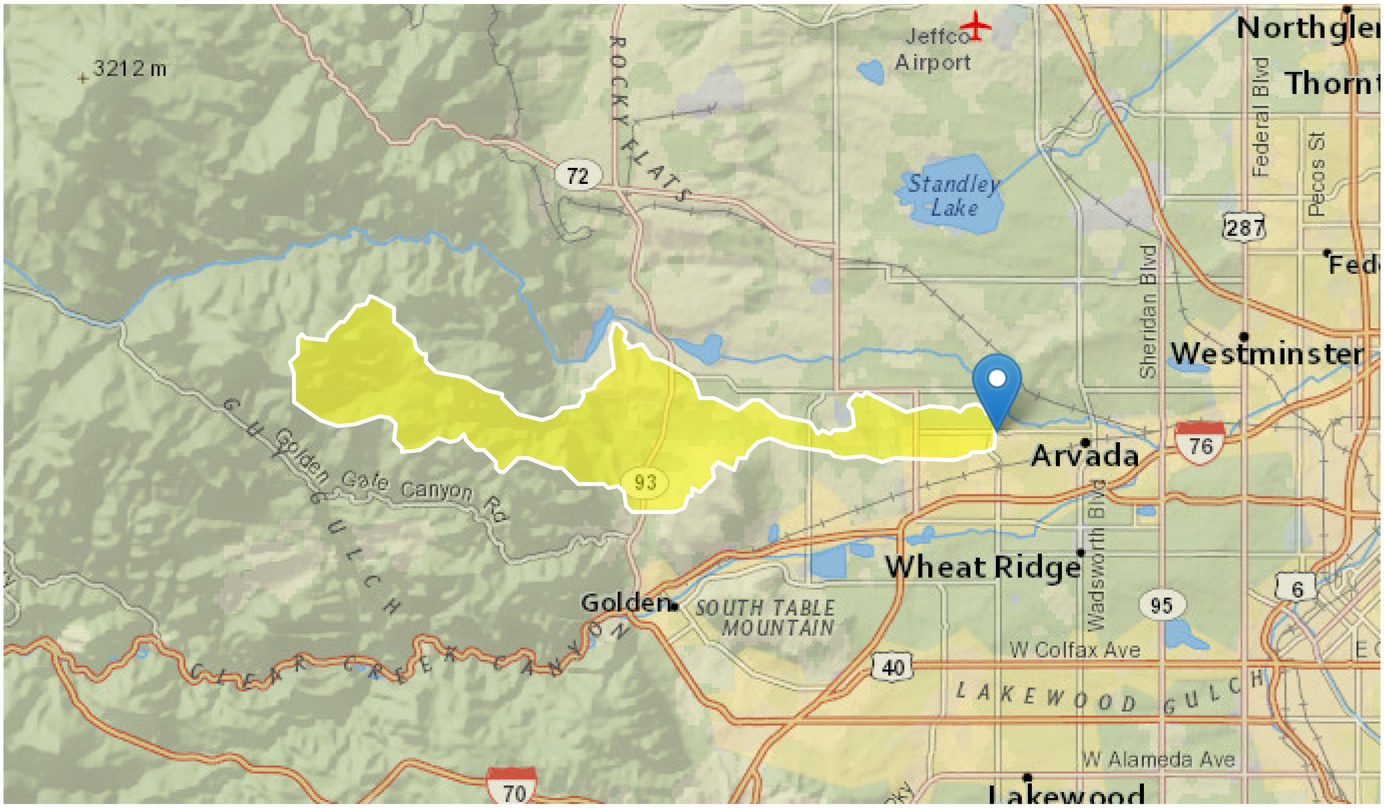
Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

StreamStats Report

Region ID: CO
Workspace ID: CO20200722172202385000
Clicked Point (Latitude, Longitude): 39.80132, -105.11039
Time: 2020-07-22 11:22:17 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	17.5	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	20	percent
PRECIP	Mean Annual Precipitation	20	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.21	inches
STATSCLAY	Percentage of clay soils from STATSGO	24.21	percent

Parameter Code	Parameter Description	Value	Unit
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5371	feet

Peak-Flow Statistics Parameters[30 Percent (5.22 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	17.5	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	20	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	20	inches	18	47

Peak-Flow Statistics Parameters[70 Percent (12.3 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	17.5	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.21	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	24.21	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5371	feet	4290	8270

Peak-Flow Statistics Flow Report[30 Percent (5.22 square miles) Mountain Region Peak Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	74.6	ft^3/s	49
5 Year Peak Flood	115	ft^3/s	44
10 Year Peak Flood	146	ft^3/s	41
25 Year Peak Flood	181	ft^3/s	40
50 Year Peak Flood	222	ft^3/s	39
100 Year Peak Flood	258	ft^3/s	36

Statistic	Value	Unit	SEp
200 Year Peak Flood	289	ft^3/s	36
500 Year Peak Flood	345	ft^3/s	33

Peak-Flow Statistics Flow Report[70 Percent (12.3 square miles) Foothills Region Peak Flow 2016 5099]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	178	ft^3/s	117
5 Year Peak Flood	494	ft^3/s	87
10 Year Peak Flood	832	ft^3/s	80
25 Year Peak Flood	1420	ft^3/s	80
50 Year Peak Flood	1980	ft^3/s	83
100 Year Peak Flood	2680	ft^3/s	88
200 Year Peak Flood	3510	ft^3/s	94
500 Year Peak Flood	4820	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	147	ft^3/s
5 Year Peak Flood	381	ft^3/s
10 Year Peak Flood	628	ft^3/s
25 Year Peak Flood	1050	ft^3/s
50 Year Peak Flood	1460	ft^3/s
100 Year Peak Flood	1960	ft^3/s
200 Year Peak Flood	2550	ft^3/s
500 Year Peak Flood	3490	ft^3/s

Peak-Flow Statistics Citations

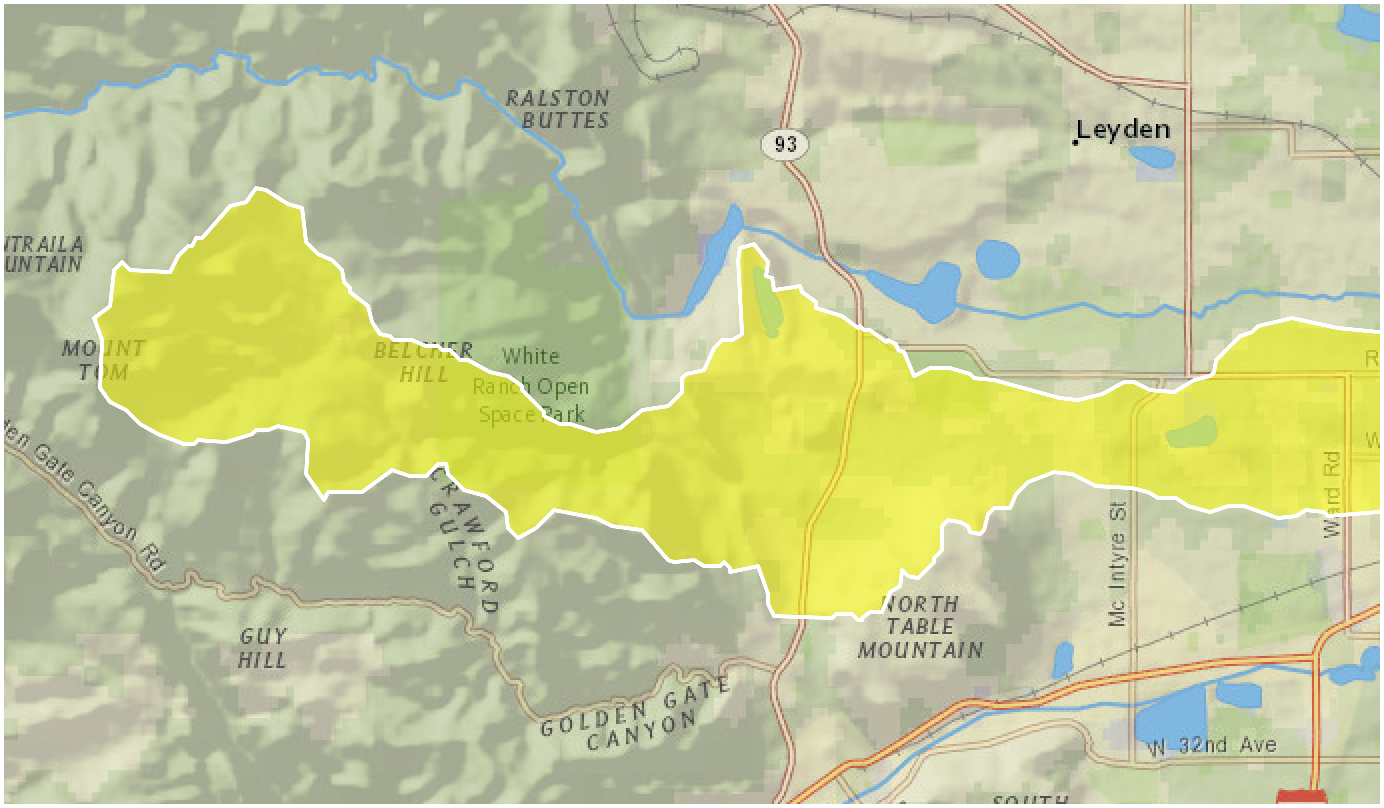
Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

Van Bibber Creek @ Mouth StreamStats Report

Region ID: CO
Workspace ID: CO20200623150343729000
Clicked Point (Latitude, Longitude): 39.80367, -105.10009
Time: 2020-06-23 09:04:00 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	20.3	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	18	percent
PRECIP	Mean Annual Precipitation	19.62	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.22	inches
STATSCLAY	Percentage of clay soils from STATSGO	25.77	percent

Parameter Code	Parameter Description	Value	Unit
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5343	feet
I24H100Y	Maximum 24-hour precipitation that occurs on average once in 100 years	4.74	inches
I24H2Y	Maximum 24-hour precipitation that occurs on average once in 2 years - Equivalent to precipitation intensity index	1.92	inches
I6H2Y	Maximum 6-hour precipitation that occurs on average once in 2 years	1.21	inches

Peak-Flow Statistics Parameters[26 Percent (5.22 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	20.3	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	18	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	19.62	inches	18	47

Peak-Flow Statistics Parameters[74 Percent (15.1 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	20.3	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.22	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	25.77	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5343	feet	4290	8270

Peak-Flow Statistics Flow Report[26 Percent (5.22 square miles) Mountain Region Peak Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	79.1	ft^3/s	49

Statistic	Value	Unit	SEp
5 Year Peak Flood	123	ft^3/s	44
10 Year Peak Flood	156	ft^3/s	41
25 Year Peak Flood	193	ft^3/s	40
50 Year Peak Flood	238	ft^3/s	39
100 Year Peak Flood	276	ft^3/s	36
200 Year Peak Flood	311	ft^3/s	36
500 Year Peak Flood	371	ft^3/s	33

Peak-Flow Statistics Flow Report[74 Percent (15.1 square miles) Foothills Region Peak Flow 2016 5099]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	209	ft^3/s	117
5 Year Peak Flood	585	ft^3/s	87
10 Year Peak Flood	989	ft^3/s	80
25 Year Peak Flood	1690	ft^3/s	80
50 Year Peak Flood	2360	ft^3/s	83
100 Year Peak Flood	3200	ft^3/s	88
200 Year Peak Flood	4200	ft^3/s	94
500 Year Peak Flood	5770	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	176	ft^3/s
5 Year Peak Flood	466	ft^3/s
10 Year Peak Flood	775	ft^3/s
25 Year Peak Flood	1300	ft^3/s
50 Year Peak Flood	1810	ft^3/s
100 Year Peak Flood	2450	ft^3/s
200 Year Peak Flood	3200	ft^3/s
500 Year Peak Flood	4380	ft^3/s

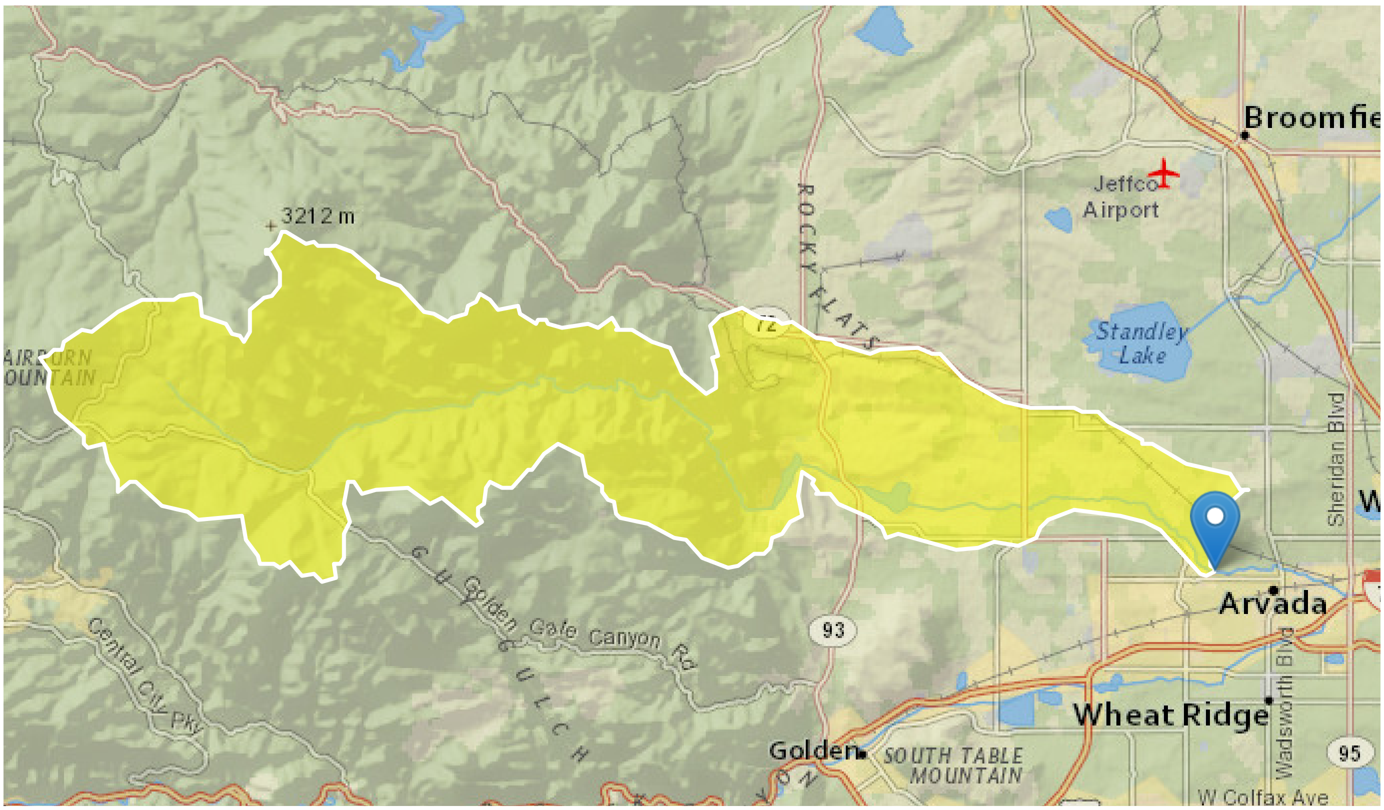
Peak-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)
Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

Ralston Cr immediately upstream of confluence with Van Bibber

Region ID: CO
Workspace ID: CO20200722171605873000
Clicked Point (Latitude, Longitude): 39.80377, -105.10026
Time: 2020-07-22 11:16:21 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	68.4	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	24	percent
PRECIP	Mean Annual Precipitation	21.13	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.09	inches

Parameter Code	Parameter Description	Value	Unit
STATSCLAY	Percentage of clay soils from STATSGO	23.44	percent
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5343	feet

Peak-Flow Statistics Parameters[55 Percent (37.9 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	68.4	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	24	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	21.13	inches	18	47

Peak-Flow Statistics Parameters[45 Percent (30.6 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	68.4	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.09	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	23.44	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5343	feet	4290	8270

Peak-Flow Statistics Flow Report[55 Percent (37.9 square miles) Mountain Region Peak Flow]

PII: Prediction Interval-Lower, Plu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	250	ft^3/s	49
5 Year Peak Flood	376	ft^3/s	44
10 Year Peak Flood	470	ft^3/s	41
25 Year Peak Flood	563	ft^3/s	40
50 Year Peak Flood	689	ft^3/s	39

Statistic	Value	Unit	SEp
100 Year Peak Flood	791	ft^3/s	36
200 Year Peak Flood	881	ft^3/s	36
500 Year Peak Flood	1040	ft^3/s	33

Peak-Flow Statistics Flow Report[45 Percent (30.6 square miles) Foothills Region Peak Flow 2016 5099]

PIl: Prediction Interval-Lower, Plu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	390	ft^3/s	117
5 Year Peak Flood	1010	ft^3/s	87
10 Year Peak Flood	1660	ft^3/s	80
25 Year Peak Flood	2770	ft^3/s	80
50 Year Peak Flood	3820	ft^3/s	83
100 Year Peak Flood	5110	ft^3/s	88
200 Year Peak Flood	6630	ft^3/s	94
500 Year Peak Flood	8970	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	313	ft^3/s
5 Year Peak Flood	660	ft^3/s
10 Year Peak Flood	1000	ft^3/s
25 Year Peak Flood	1550	ft^3/s
50 Year Peak Flood	2090	ft^3/s
100 Year Peak Flood	2720	ft^3/s
200 Year Peak Flood	3450	ft^3/s
500 Year Peak Flood	4580	ft^3/s

Peak-Flow Statistics Citations

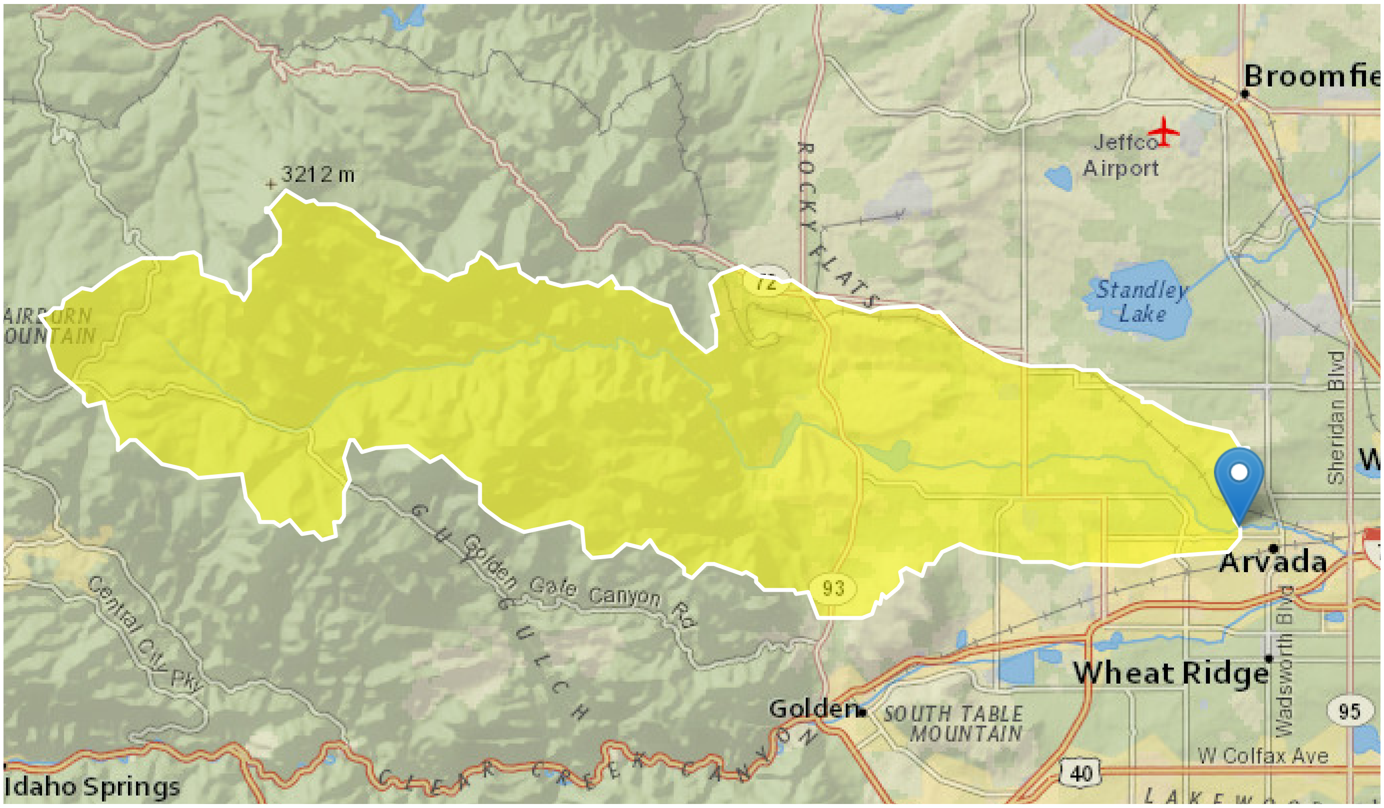
Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

StreamStats Report

Region ID: CO
Workspace ID: CO20200722173208110000
Clicked Point (Latitude, Longitude): 39.80442, -105.09167
Time: 2020-07-22 11:32:23 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	89.1	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	22	percent
PRECIP	Mean Annual Precipitation	20.77	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.12	inches
STATSCLAY	Percentage of clay soils from STATSGO	24.03	percent

Parameter Code	Parameter Description	Value	Unit
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5333	feet

Peak-Flow Statistics Parameters[48 Percent (43.1 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	89.1	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	22	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	20.77	inches	18	47

Peak-Flow Statistics Parameters[52 Percent (46.1 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	89.1	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.12	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	24.03	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5333	feet	4290	8270

Peak-Flow Statistics Flow Report[48 Percent (43.1 square miles) Mountain Region Peak Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	292	ft^3/s	49
5 Year Peak Flood	440	ft^3/s	44
10 Year Peak Flood	552	ft^3/s	41
25 Year Peak Flood	660	ft^3/s	40
50 Year Peak Flood	808	ft^3/s	39
100 Year Peak Flood	931	ft^3/s	36

Statistic	Value	Unit	SEp
200 Year Peak Flood	1040	ft^3/s	36
500 Year Peak Flood	1220	ft^3/s	33

Peak-Flow Statistics Flow Report[52 Percent (46.1 square miles) Foothills Region Peak Flow 2016 5099]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	479	ft^3/s	117
5 Year Peak Flood	1240	ft^3/s	87
10 Year Peak Flood	2050	ft^3/s	80
25 Year Peak Flood	3420	ft^3/s	80
50 Year Peak Flood	4720	ft^3/s	83
100 Year Peak Flood	6340	ft^3/s	88
200 Year Peak Flood	8220	ft^3/s	94
500 Year Peak Flood	11100	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	389	ft^3/s
5 Year Peak Flood	856	ft^3/s
10 Year Peak Flood	1320	ft^3/s
25 Year Peak Flood	2090	ft^3/s
50 Year Peak Flood	2830	ft^3/s
100 Year Peak Flood	3720	ft^3/s
200 Year Peak Flood	4750	ft^3/s
500 Year Peak Flood	6350	ft^3/s

Peak-Flow Statistics Citations

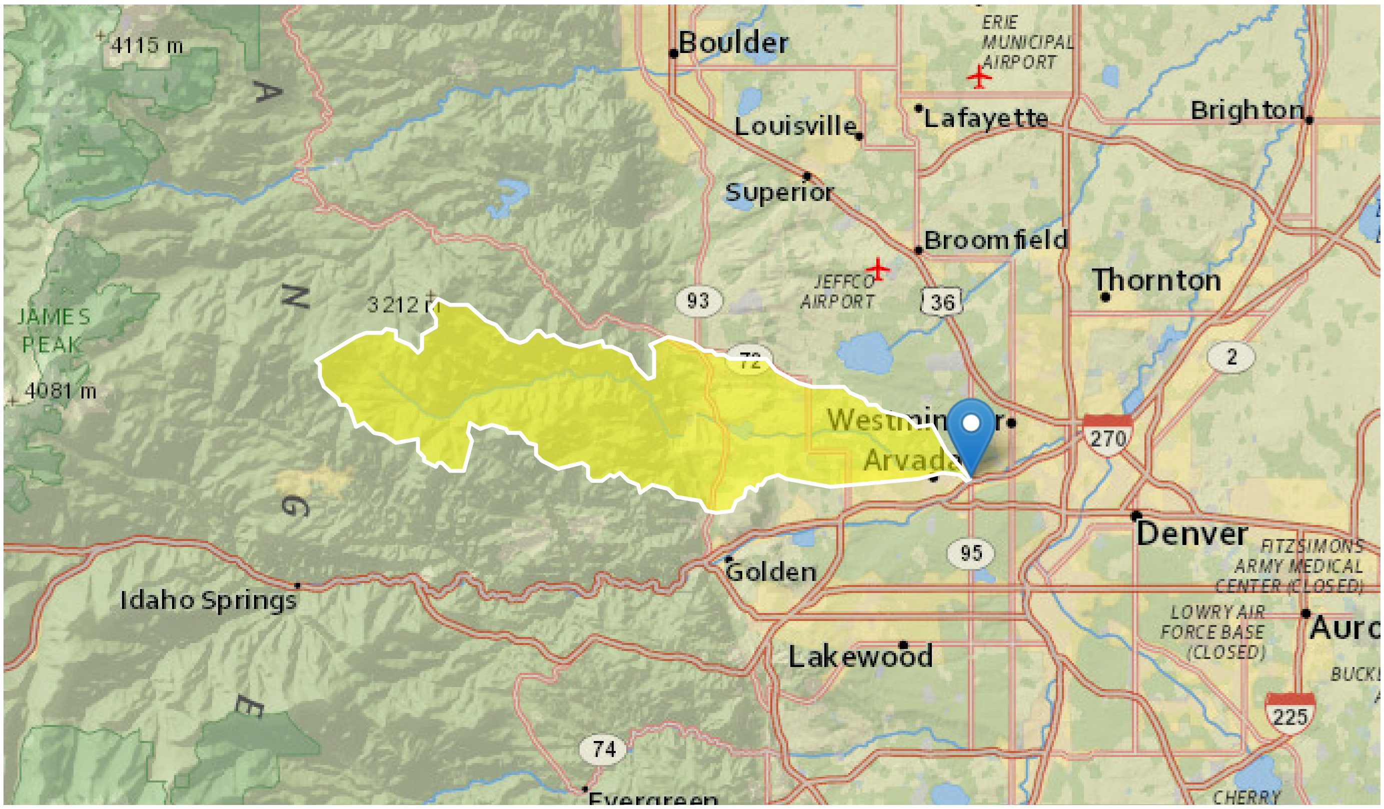
Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

StreamStats Report

Region ID: CO
Workspace ID: CO20200722173846279000
Clicked Point (Latitude, Longitude): 39.79752, -105.05466
Time: 2020-07-22 11:39:01 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	91.5	square miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	22	percent
PRECIP	Mean Annual Precipitation	20.67	inches
I6H100Y	6-hour precipitation that is expected to occur on average once in 100 years	3.13	inches
STATSCLAY	Percentage of clay soils from STATSGO	24.3	percent

Parameter Code	Parameter Description	Value	Unit
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	5253	feet

Peak-Flow Statistics Parameters[47 Percent (43.1 square miles) Mountain Region Peak Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	91.5	square miles	1	1060
BSLDEM10M	Mean Basin Slope from 10m DEM	22	percent	7.6	60.2
PRECIP	Mean Annual Precipitation	20.67	inches	18	47

Peak-Flow Statistics Parameters[53 Percent (48.5 square miles) Foothills Region Peak Flow 2016 5099]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	91.5	square miles	0.6	2850
I6H100Y	6 Hour 100 Year Precipitation	3.13	inches	2.38	4.89
STATSCLAY	STATSGO Percentage of Clay Soils	24.3	percent	9.87	37.5
OUTLETELEV	Elevation of Gage	5253	feet	4290	8270

Peak-Flow Statistics Flow Report[47 Percent (43.1 square miles) Mountain Region Peak Flow]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	295	ft^3/s	49
5 Year Peak Flood	445	ft^3/s	44
10 Year Peak Flood	559	ft^3/s	41
25 Year Peak Flood	668	ft^3/s	40
50 Year Peak Flood	819	ft^3/s	39
100 Year Peak Flood	943	ft^3/s	36

Statistic	Value	Unit	SEp
200 Year Peak Flood	1050	ft^3/s	36
500 Year Peak Flood	1240	ft^3/s	33

Peak-Flow Statistics Flow Report[53 Percent (48.5 square miles) Foothills Region Peak Flow 2016 5099]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SEp
2 Year Peak Flood	515	ft^3/s	117
5 Year Peak Flood	1350	ft^3/s	87
10 Year Peak Flood	2230	ft^3/s	80
25 Year Peak Flood	3740	ft^3/s	80
50 Year Peak Flood	5160	ft^3/s	83
100 Year Peak Flood	6940	ft^3/s	88
200 Year Peak Flood	9020	ft^3/s	94
500 Year Peak Flood	12300	ft^3/s	104

Peak-Flow Statistics Flow Report[Area-Averaged]

Statistic	Value	Unit
2 Year Peak Flood	412	ft^3/s
5 Year Peak Flood	924	ft^3/s
10 Year Peak Flood	1440	ft^3/s
25 Year Peak Flood	2290	ft^3/s
50 Year Peak Flood	3120	ft^3/s
100 Year Peak Flood	4120	ft^3/s
200 Year Peak Flood	5270	ft^3/s
500 Year Peak Flood	7070	ft^3/s

Peak-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

Kohn, M.S., Stevens, M.R., Harden, T.M., Godaire, J.E., Klinger, R.E., and Mommandi, A.,2016, Paleoflood investigations to improve peak-streamflow regional-regression equations for natural streamflow in eastern Colorado, 2015: U.S. Geological Survey Scientific Investigations Report 2016–5099, 58 p. (http://dx.doi.org/10.3133/sir20165099)

Application Version: 4.3.11

Attachment E. WET Gage Report for Lena Gulch

MEMORANDUM

To: Kevin Stewart, MHFD
From: Water & Earth Technologies, Inc. Kate Malers and Blair Hanna
Date: 8/6/2020
Subject: Lena Gulch Annual Peak Discharge Analysis

Summary:

The Lena Gulch watershed has four stage and discharge gages (upstream to downstream)

ALERT 1043 Lena @ U.S. Highway 6

USGS 06719560 LENA GULCH AT LAKEWOOD, CO

1023 – Lena Gulch at Nolte Pond Water Level

1003 – Lena Gulch at Maple Grove Reservoir Water Level

ALERT 1043 Lena @ U.S. Highway 6. Drainage Area (StreamStats) 3.54 mi^2. See location shown in Figure 1. Stage data available 9/4/1985 11:38:44 AM to the present. Discharge data available 3/11/1995 12:58:26 PM – present. MHFD annual peaks spreadsheet data were verified, with some changes noted. Annual peaks are shown in Table 1. Peaks are noted in ( ) if differences from the MHFD spreadsheet were found, most due to rounding. All flow estimates are products of the same rating. The gage is located in the steep chute upstream of a culvert entrance and typically measures flows that are critical or supercritical, so the rating is "flashy" and small increases in stage create large increases in discharge.

USGS 06719560 LENA GULCH AT LAKEWOOD, CO, Drainage Area 9.06 mi^2 (8.8 mi^2 StreamStats). Gage location upstream of Maple Grove Reservoir (Figure 1). Crest Stage Indicators (CSI) peak flow data are only available for 1973-2013 (1974, 1981-1986, 2008 missing). Continuous monitoring began April 2013-present. Discharge data available from 4/1/2013-present, stage and discharge pairs only available 4/1/2020-present. Data quality flags on continuous data only indicate estimated data only at some **low** discharge values (<36cfs). Field measurements have been performed at the gage since 1986 through a wide range of flows. Annual peaks are shown in Table 2.

ALERT 1023 Lena @ Nolte Pond. Drainage Area (StreamStats) 9.45 mi^2. See location shown in Figure 1. Stage and discharge data available from 4/1/1986 to the present. Annual peaks are not available for 1989, 1994 and 2014, due to limited data during these years. MHFD annual peak values were verified through cleaning currently available valid data. Annual peaks are shown in Table 3. Peaks are noted in ( ) if differences from the MHFD spreadsheet were found, most due to rounding. Different peak values were found for 1990, 1995 and 2017. All flow estimates are products of the same rating.

ALERT 1003 Maple Grove Reservoir. Drainage Area 10.5 mi^2 (StreamStats slightly downstream of dam). See location shown on Figure 1. Stage and discharge data available from 9/25/1986 to the present. Annual peaks available for 1987-present. In general, these flows are lower than other measurement locations in the watershed due to reservoir storage and attenuation. Only data describing discharges over 100 cfs were QA/QC'd and verified by WET. Annual peaks from MHFD annual peaks spreadsheet are shown in Table 4. An original rating was rendered obsolete for high discharges in December 2004 when the Fabridams were removed and replaced with steel crest gates; the rating was updated in 2011 to reflect the 2004 physical changes and in 2012 to reflect a change in Maple Grove's Gate Operations

Plan. However, both rating updates were focused on high flow discharge estimates that have not been experienced at the gage in the period of record. Also, emergency spillway gate or Fabridam lowering have never occurred.

A comparison plot of annual peaks at these four locations through time is presented in Figure 7.

USGS StreamStats analysis for watershed locations is presented at the end of this document.

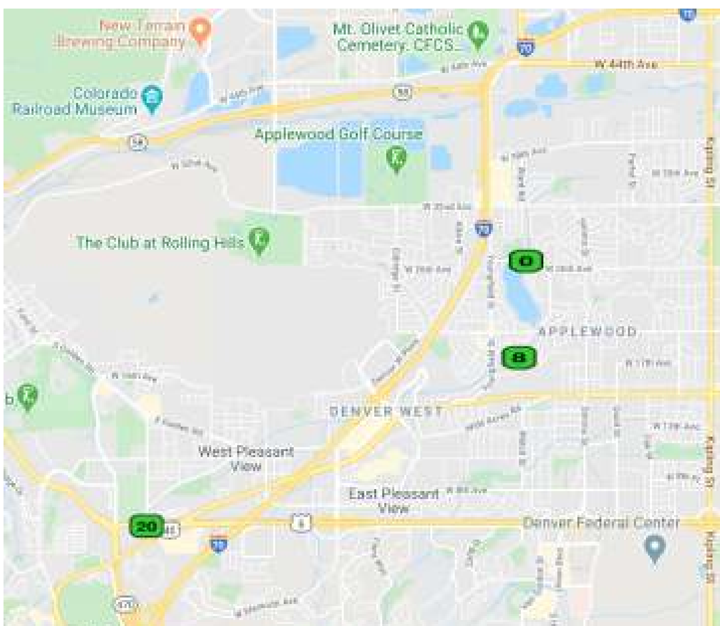


Figure 1. Location of USGS 06719560 (above) and ALERT Stations 1000, 1020, 1040 (North to South) from GMAP.

Gage Locations

Lena at Highway 6 ALERT gage measures stage at the entrance of a large concrete culvert downstream of a large concrete urban floodway with varying slope and width in the culvert approach section (Figure 2). The steep channel slope flattens abruptly at the entrance to the culvert. Two drainage pipes contribute flow at the entrance also. The PT riser pipe is therefore in a poor location for accurately measuring discharge. The PT may be measuring stage in turbulent water, possibly laterally non-uniform and possibly in a hydraulic jump that could form if the flow transitions from supercritical in the steep channel to subcritical temporarily inside the culvert (before a drop downstream). The riser would be better sited slightly upstream along the wall (although privately owned), and a rating of critical flow in a rectangular concrete channel used (although the pipe flow contribution just upstream of the culvert would not be accounted for).

Figure 2 ALERT 1043 Lena @ U.S. Highway 6.

The USGS gage in Lena gulch should win an award for functional design and aesthetics! (Figure 3). The riser and old crest stage indicator are in a pool upstream of a small hydraulic control in a straight, engineered channel. This gage is well designed to accurately measure low to moderate flows. At higher flows, a downstream drop may act as a second downstream control. This sub-urban stream channel in the vicinity of the gage is rip rap lined with a confined overbank of landscape timbers. At high flows the vegetation may interfere and there is a possibility of loss of containment to the adjacent street near the gage. Overall, a very good gaging station location.

Figure 3. USGS 06719560 LENA GULCH AT LAKEWOOD, CO

The Nolte Pond ALERT gage is another urban oasis in Lena Gulch. The riser pipe measures stage in a small concrete lined pond (Figure 4). The pond has a clear outlet that probably allows for a reasonably accurate hydraulic rating. **However**, the outlet structure was designed to allow for multiple water levels. A short metal wall that fits in a groove in the concrete can be removed. This flashboard structure appears to allow for a change of 1-1.5ft of stage in the pond by the homeowner. Unless the rating accounts (seasonally?) for the installation of the wall, the discharge could be slightly overpredicted. The NovaStar database has two ratings available (with and without flashboards). These ratings are slightly different only below the 1.2 ft of change accounting for the flashboard, with discharge differences up to 40cfs at stages < 1.2 ft. The “without flashboard” rating was applied to all the stage data in the database, there the discharges are slightly overpredicted for time when the flashboards were actually in place.

The PT pond is downstream of another pond, which has a powered gate structure (Figure 5). The homeowners could drain this upstream pond prior to a large rainfall event to allow for some additional storm runoff storage. It should be noted that adjacent to this pond a home with a large patio and pool exists, which are notably lower than the pond level. It would seem that loss off containment of Lena Gulch flow would be common. Even without active operation, this upstream pond would attenuate peak flows only slightly at the Nolte Pond gage site. Overall, a well-designed gaging station but the movement of the flashboard throughout the year is a drawback to the rating accuracy, at low flows particularly.

Figure 4. ALERT 1023 Lena @ Nolte Pond, looking downstream, looking upstream at the control and looking at the control and short metal wall (flashboard) lying near the lawn that can be removed.

Figure 5. Small pond upstream of Nolte Pond.

The flow is estimated using the reservoir water level at the ALERT 1003 Maple Grove Reservoir station (Figure 6). For existing conditions, there are two radial gates set at two different levels (6’ and 10’ heights above the concrete spillway crest) and lowered if required to change or release reservoir storage, as defined by the gate operations plan for flood control. As with Nolte Pond, an accurate stage-discharge rating can be developed for the hydraulic structure when water is flowing over the gate(s). If the hydraulic rating took the real time gate positions into account, the discharge could be more accurate. The station does not have the gate position feedback, so the gate positions are NOT included in the NovaStar rating. However, the gate operations schedule is fairly rigid and gate positions are pre-defined except during an emergency, and these gates have not been lowered to date, so the rating is as accurate as possible without real-time gate feedback.

Prior to 2012, a different rating accounted for the two Fabridams that were in place, using a similar operation schedule. The height of the dams could change during an emergency, although they were never lowered.

The discharges at this station will be much different from the other stations along Lena Gulch because of attenuation of the flood peak due to the reservoir and change in storage if water is not flowing over the dam prior to an event. In summary, the ALERT gage at Maple Grove reservoir may not always accurately portray the flow below the reservoir (not accounting for real time gate positions if gates were moved in an emergency and not accounting for some dam seepage) nor does this station accurately portray the overall flow in Lena Gulch watershed (not accounting for change in storage over time).

Figure 6 ALERT 1003 Maple Grove Reservoir.

Data QA/QC Method

Despite the gaging station measurement accuracy issues indicated above, the existing available data were analyzed. WET plotted all available ALERT stage and rated discharge data. A constant threshold was used to remove obvious erroneous data from analysis (e.g., discharges above 10,000 cfs ignored assuming radio decoding problem or instrumentation testing). Annual peaks were calculated with Excel pivot tables. Individual erroneous points were identified and removed from analysis by scanning stage and flow data to find problems. All data are retained in the NovaStar database for reference during the analysis, erroneous data points are just removed from consideration. Annual peaks were compared against the MHFD Annual Peaks spreadsheet. Some rounded values for stage and/or discharge were originally presented in this Peaks spreadsheet.

WET removed bad ALERT data from consideration, assuming signatures of bad data are:

- Instantaneous large increase in stage, followed eventually by an instantaneous large decrease in stage. This usually occurs as a block of data but sometimes occurs as a single point. This is assumed to be radio/decoding error or calibration/testing of PT. Hard to tell if bad data for change in stage of e.g.,~1 ft in a few minutes (resulting in change in discharge of 300-500cfs). These small “waves” could be real during runoff events.
- ALERT data points sent every minute for hours and slowly increasing. This is a failing PT. Hard to determine bounds of bad data.
- Very high values of stage near startup in March/April. This is PT testing data.

Within a dataset if years of data are of much lower quality (e.g., incomplete data during flood season, very few days of data, abundant noted erroneous data points) these years are marked as “insufficient good data to determine peak.” For some years, a maximum peak can be found within a section of clean data but if the dataset does not include measurements throughout June-September or at least July and August, it is unlikely the annual “maximum” is a trustworthy annual peak that can be used for comparison with other years. For these reasons, a maximum value was not even presented.

Table 1. Annual Peak Flows ALERT 1043 Lena @ U.S. Highway 6, Data from current MHFD annual peak spreadsheet in ( ) if differences were found, mostly due to rounding.

Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)	Comment
1985	--	--	Insufficient good data to determine peak
1986	--	--	Insufficient good data to determine peak
1987	--	--	Insufficient good data to determine peak
1988	--	--	Insufficient good data to determine peak
1989	--	--	Insufficient good data to determine peak
1990	--	--	Insufficient good data to determine peak
1991	--	--	Insufficient good data to determine peak
1992	--	--	Insufficient good data to determine peak
1993	--	--	Insufficient good data to determine peak
1994	--	--	Insufficient good data to determine peak
1995	--	--	Insufficient good data to determine peak
1996	--	--	Insufficient good data to determine peak
1997	--	--	Insufficient good data to determine peak
1998	--	--	Insufficient good data to determine peak
9/21/1999	19.87	28	
9/20/2000	19.95 (19.90)	42 (41)	
7/10/2001	20.44 (20.40)	288 (285)	
8/27/2002	20.20	158 (160)	
8/29/2003 (7/29/2003)	20.28 (21.01)	201 (598)	New Peak, 598 in likely erroneous data
6/8/2004	20.86 (20.90)	**514 (520)**	**Historic Peak**
5/29/2005	19.90	33	
7/7/2006	20.00	55	
5/29/2007	--	--	Insufficient good data to determine peak
2008	--	--	Insufficient good data to determine peak
2009	--	--	Insufficient good data to determine peak
2010	--	--	Insufficient good data to determine peak
2011	--	--	Insufficient good data to determine peak
2012	--	--	Insufficient good data to determine peak
2013	--	--	Insufficient good data to determine peak
5/24/2014	20.58	365	
5/8/2015	20.12	115	
8/30/2016	20.73	447	
7/26/2017	19.98	47	
7/3/2018	--	--	Insufficient good data to determine peak
7/20/2019	20.84	505	

Table 2. Annual Peak Flows data from USGS 06719560 LENA GULCH AT LAKEWOOD, CO Data are USGS documented annual peaks from crest stage gage data for all years except 2013-present.

Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)
10/12/1973[2]	10.35	17
1974 data missing		
7/20/1975	14.41	641
9/7/1976	11.84	238
7/5/1977	14.2	615
5/17/1978	12.22	110
1979, date unknown	11.8	76
5/5/1980	14.17	450[3]
1981 – 1986 data missing		
1987, date unknown	13.78	587
8/4/1988	11.96	223
6/3/1989	12.12	215
7/8/1990	12.16	222
6/1/1991	13.59	514
8/24/1992	11.69	227
6/17/1993	11.12	115
8/10/1994	11.58	201
5/17/1995	12.75	327
5/26/1996	11.78	166
7/27/1997	12.64	305
7/25/1998	12.39	260
6/25/1999	12.06	206
8/17/2000	12.41	263
7/8/2001	12.43	267
5/24/2002	10.91	62
8/30/2003	11.9	183
6/27/2004[4]	13.13	407[4]
8/4/2005	12.14	218
8/13/2006	11.09	79
5/23/2007	12.85	347
2008 data missing		
6/14/2009	12.04	219
6/12/2010	11.56	147
9/14/2011	11.82	190
7/8/2012	11.99	214
9/9/2013	13.62	546[1]
5/24/2014	13.24	455[1]
5/8/2015	11.85	302[1]
8/30/2016	12.11	392[1]
5/18/2017	11.2	134[1]
7/15/2018	11.88	312[1]
7/20/2019	14.38	**693[1] (historic peak)**

[1] Continuous monitoring data, USGS documented annual peaks verified in 5-minute data.

[2] 1973 incomplete year of data. USGS station description: "Crest-stage gage first established on Alkire Street, approx. October 1973."

[3] Outlier not following rating, possibly in error. Hard to tell if stage or discharge are erroneous.

[4] 2004 Annual peak at other basin gages was found to be on 6/8/2004. The USGS provides only one data point per year, so no data are available for 6/8/2004 for comparison.

Table 3. Annual Peak flows ALERT 1023 Lena @ Nolte Pond, Data from current MHFD annual peak spreadsheet in ( ) if differences were found, mostly due to rounding.

Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)	Comment
8/22/1986	46.90, (46.80)	73, (66)	MHFD spreadsheet peak stages rounded
6/8/1987	50.00, (49.90)	701, (672)	MHFD spreadsheet peak stages rounded
8/4/1988	47.95, (47.90)	210, (194)	MHFD spreadsheet peak stages rounded
1989	--	--	Insufficient good data to determine peak
7/8/1990 (10/16/1990)	47.62, (49.10)	165, (439)	New peak. Old peak in bad data, all data 10/1-end ignored. Possible storm data in Oct, but too much bad data to trust the peak
6/1/1991	49.50	555	
8/24/1992	47.50	150	
9/17/1993	47.40	138	
1994	--	--	Insufficient good data to determine peak
7/19/1995 (4/16/1995)	47.50	150	Different peak date found (this stage and discharge did not occur on 4/16)
9/18/1996	47.67	172	
7/27/1997	47.75	182	
7/30/1998	47.30	125	9/18-end of year ignored
8/31/1999	47.03	86	
8/17/2000	47.17, (47.20)	109	MHFD spreadsheet peak stages rounded
7/10/2001	47.45, (47.40)	143	MHFD spreadsheet peak stages truncated
5/24/2002	47.14, (47.10)	105	MHFD spreadsheet peak stages rounded
8/30/2003	47.55	156, (159)	
6/8/2004	50.44 (50.40)	**865 (860)**	**Historical Peak**
8/4/2005	47.63 (47.60)	166 (167)	
6/3/2006 (6/24/2006)	46.68 (46.50)	58 (45)	New annual peak
5/29/2007	47.44 (47.40)	142	
9/11/2008	46.78	65	
6/14/2009 (10/30/2009)	47.35, (57.58)	131, (5,179)	New annual peak, original peak in bad data
4/22/2010	47.16	107	
7/13/2011	47.40	137	
7/17/2012	49.89	668	
9/9/2013	48.71	363	
2014	--	--	no viable data 2014; station upgraded/rebuilt 10/24/14
5/8/2015	46.94	76	
8/30/2016	47.16	108	
5/18/2017	46.73	61	
5/3/2018	46.91	74	
7/20/2019	48.63	345	

Table 4. Annual Peak flows ALERT 1003 Lena @ Maple Grove, Data from current MHFD annual peak spreadsheet are shown. Data describing peaks above 100 cfs were QA/QC 'ed and verified by WET.

Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)	Comment
6/9/1987	27.27	33	
5/20/1988	27.39	47	
6/3/1989	26.47	7	
8/17/1990	25.96	0	Zero is rated discharge at stage presented
6/1/1991	27.28	33	
8/24/1992	26.98	24	
6/18/1993	26.74	23	
6/4/1994	26.29	0	Zero is rated discharge at stage presented
5/17/1995	27.37	45	
9/19/1996	27.20	25	
7/28/1997	27.20	27	
7/30/1998	26.70	23	
4/29/1999	27.09	24	
7/17/2000	26.90	24	
5/5/2001	27.10	21	
6/25/2002	25.40	0	Zero is rated discharge at stage presented
8/31/2003	26.64	19	
6/8/2004	28.24[1]	190	
6/11/2005	27.00	25	
7/9/2006	26.30	0	Zero is rated discharge at stage presented
5/29/2007	27.50	58	
8/16/2008	26.43	5	
6/14/2009	26.78	17	
4/23/2010	26.79	23	
7/13/2011	26.60	7	
7/10/2012	25.92	0	Zero is rated discharge at stage presented
9/12/2013	28.01	**280**	**Historic Peak**
7/30/2014	27.24	132	
5/9/2015	26.84	74	
4/19/2016	26.45	29	
5/18/2017	26.71	57	
6/19/2018	26.14	6	
7/22/2019	26.61	45	

[1] Stage higher than 2013 historic peak, two different ratings in place pre-2012 and post-2012

A comparison of peaks across gages is shown graphically (Figure 7) and in tabular form (Table 5) below.

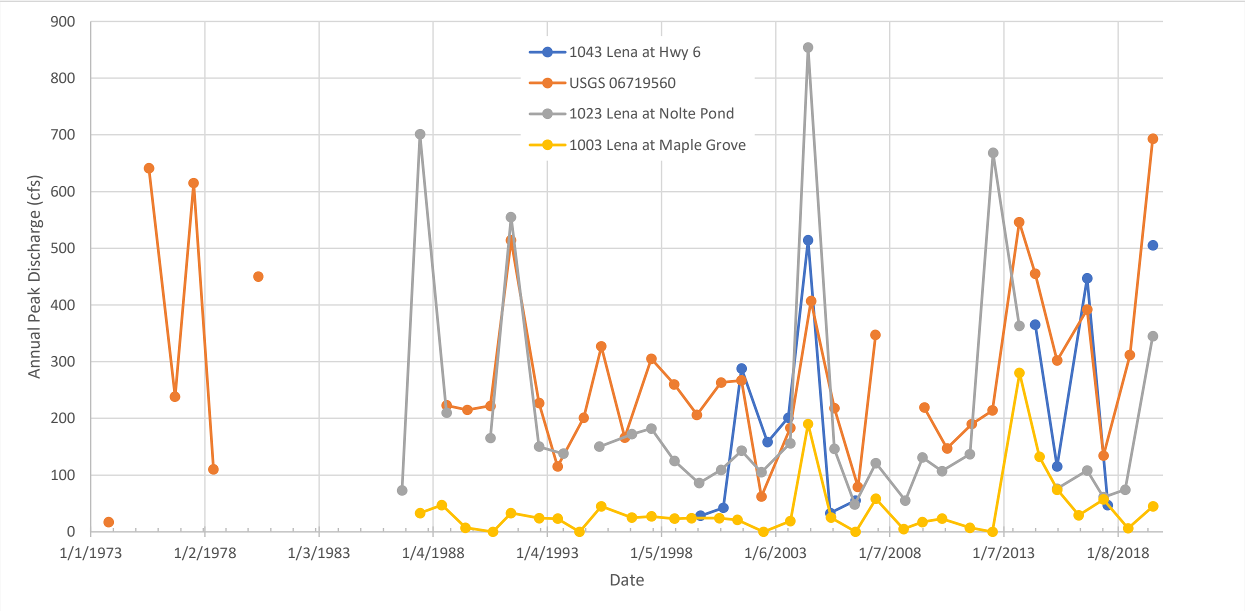


Figure 7. Annual Peaks for The Four Lena Gulch Gages.

Table 5. Measured Annual Peaks when Annual Peak Days Aligned Across Gages

Date of Matched Annual peak	ALERT 1043	USGS 06719560	ALERT 1023	ALERT 1003
6/8/1987			701	33
8/4/1988		223	210	
7/8/1990		222	165	
6/1/1991		514	555	27
8/24/1992		227	150	24
6/17/1993		115		23
5/17/1995		327		45
9/18/1996			172	25
7/27/1997		305	182	24
8/17/2000		263	109	
7/10/2001	288		143	
5/24/2002		62	105	
8/30/2003		183	156	19
6/8/2004	514		854	190
8/4/2005		218	146	
5/29/2007			121	58
6/14/2009			131	17
4/22/2010			107	23
7/13/2011			137	7
9/9/2013		546	363	
5/24/2014	365	455		
5/8/2015	115	302	76	57
8/30/2016	447	392	108	
7/20/2019	505	693	345	

The peak stage and discharge data were plotted as stage-discharge ratings. At most gages, the ratings have changed over time.

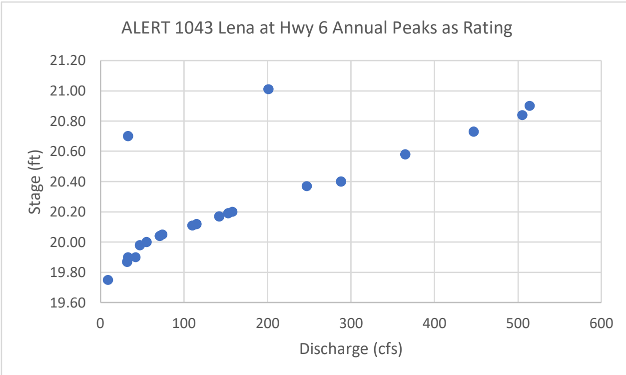
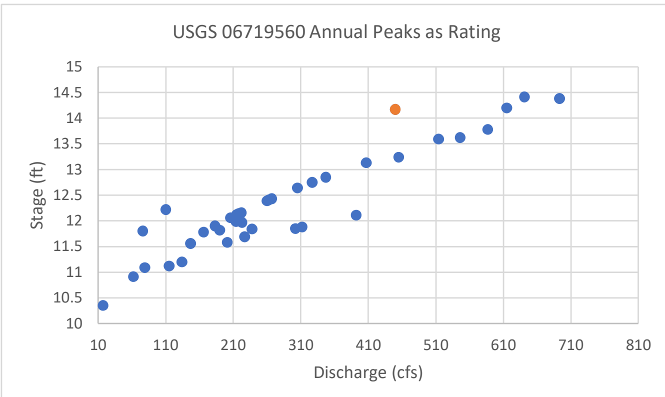


Figure 8. ALERT 1043 peaks as rating. Points from 2003 and 2005 appear as outliers



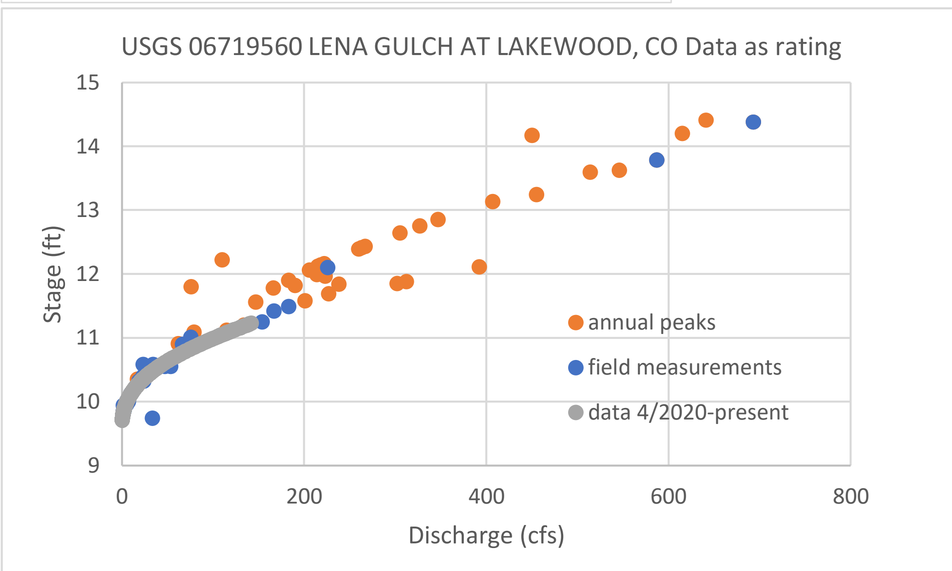


Figure 9. USGS 06719560 peaks as rating. Data point in orange is 1980 in top plot. Field discharge measurements and recent data are overlaid.

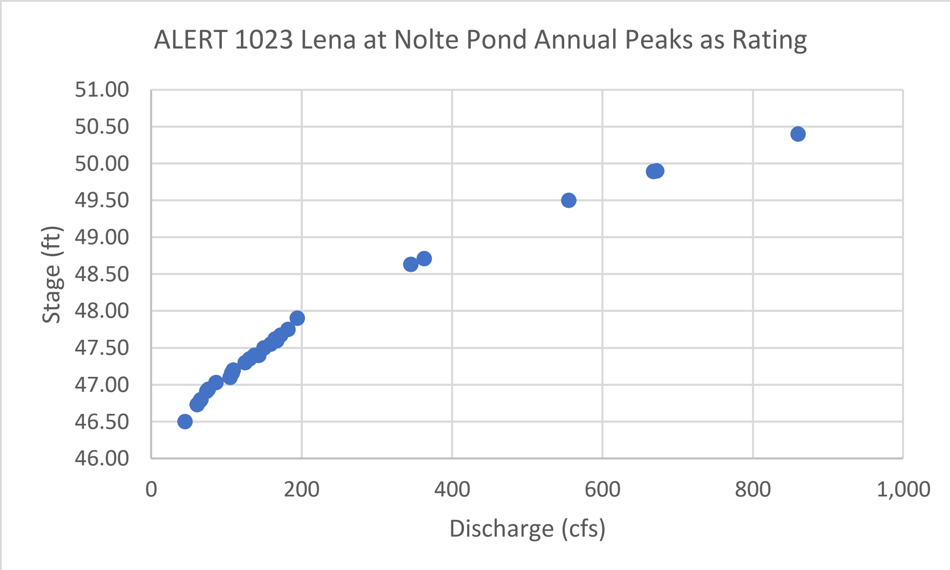


Figure 10. ALERT 1023 peaks as rating. One rating (without flashboards) used throughout time. Influence of seasonal flashboard installation or removal is not in rating.

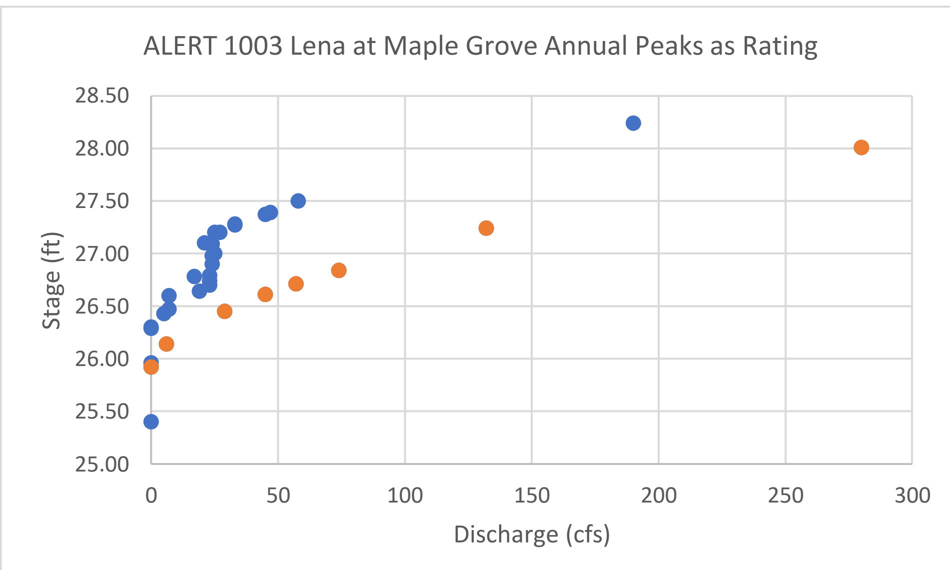


Figure 11. ALERT 1003 peaks as rating. Two ratings in place over time, points in orange are 2012-present, Blue points are the Fabridam rating.

Table 6. USGS StreamStats Results.

		StreamStats Peak Flows				
Gage Location	**Drainage Area (mi^2)**	**25-yr**	**50-yr**	**100-yr**	**200-yr**	**500-yr**
ALERT 1040 Lena at US Hwy 6	3.54	372	521	713	935	1290
USGS 06719560 Lena at Lakewood	8.8 (9.06 USGS gage info)	1010	1430	1950	2580	3570
ALERT 1023 Lena at Nolte Pond	9.45	1120	1580	2160	2850	3960
ALERT 1003 Maple Grove	10.5	1280	1810	2480	3270	4550

Model values in red do NOT account for Nolte Pond and Maple Grove reservoir storage and flow attenuation.

Summary/Recommendations

The available stage and discharge data were analyzed. Based on potential discharge characterization errors at ALERT 1040 Lena at US Hwy 6 (hydraulic issues at riser), ALERT 1023 Lena at Nolte Pond (not accounting for elevation of removable flashboard over time) and ALERT 1003 Maple Grove (not accounting for gate positions over time or reservoir storage), the USGS 06719560 Lena at Lakewood probably provides the most accurate data describing discharge in Lena Gulch.

The Hwy 6 (ALERT 1040) and Nolte Pond (ALERT 1023) datasets had significant errors limiting the amount of valid data for comparison across years.

Peak flows should be cross correlated with rainfall events.

Attachment F. WET Gage Report for Little Dry Creek

MEMORANDUM

To: Kevin Stewart, MHFD
From: Water & Earth Technologies, Inc. Kate Malers and Blair Hanna
Date: 8/6/2020
Subject: Little Dry Creek Watershed Annual Peak Discharge Analysis

Summary:

Three flow measurement points exist in the Little Dry Creek watershed, all near the watershed outlet (Figure 1). The USGS gage 06719840 provides the most complete data available.

USGS 06719840 LITTLE DRY CREEK AT WESTMINSTER, CO, DRAINAGE AREA- 10.4 mi^2. Crest Gage peak flow data are only available for 1982-2012. Continuous monitoring began May 2013-present (instrumentation shown in Figure 2). Discharge data available from 5/23/2013-present, stage and discharge pairs only available 2/5/2020-present. Data quality flags only indicate estimated data only at some low stage values. USGS peak flows match maximum of annual flows continuous data (when continuous data exist 2013-present).

USGS 06719845 LITTLE DRY CRK BLW FEDERAL BLVD AT WESTMINSTER, CO, DRAINAGE AREA- 12.7 mi^2. Continuous discharge data from 3/26/2019-present, stage and discharge pairs only available from 2/5/2020-present. Station located ~1 mile downstream from 06719840 (instrumentation shown in Figure 3). Westminster Station Park 0.3 mi upstream has a new pond that may capture flow or attenuate peak. Park was updated and pond added between 10/2014 and 10/2015, pond completed by 2/2016 (based on Google Earth imagery). Bridges between the USGS stations and the wooded area at this gage site may also attenuate peak flows measured at the two USGS gages in the future.

MHFD ALERT/ALERT2 1310 Little Dry at 64th. Drainage area unknown because of upstream diversion. Stage and discharge data available 3/6/2002-present. Station located ~0.65 mi downstream from USGS 06719845 on a split flow channel. Station measures inflow to a large detention reservoir and appears to measure only a portion of total flow when high flows are diverted from Little Dry creek to the detention basin (see Figure 4). Dates of some peak stage correspond with USGS peak days, but discharges are much lower.

USGS StreamStats analysis for the watershed is presented at the end of this document for comparison.

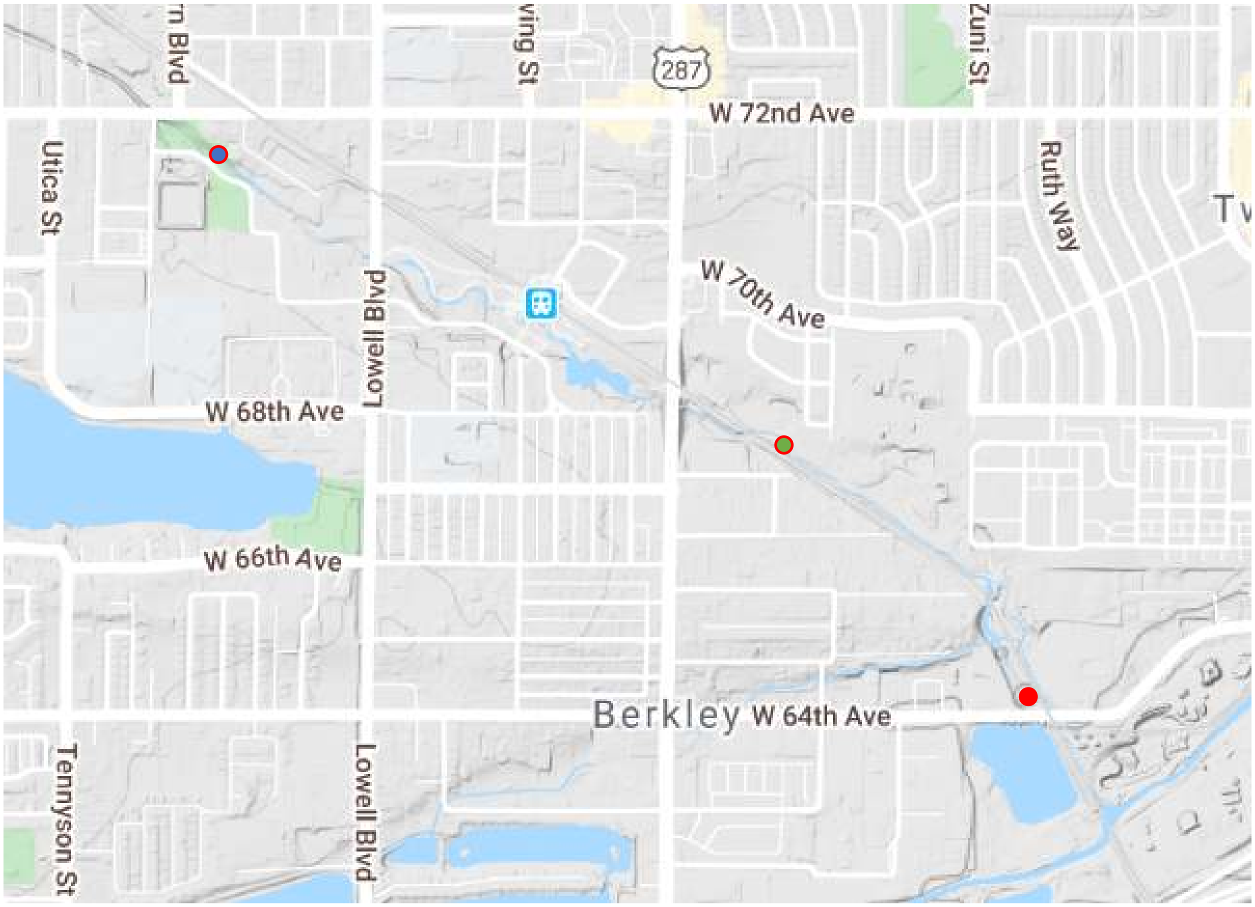


Figure 1. Little Dry Creek Gage Locations upstream to downstream near confluence with Clear Creek (lower right). Blue/red circle USGS LITTLE DRY CREEK AT WESTMINSTER, CO, green/red circle USGS LITTLE DRY CRK BLW FEDERAL BLVD AT WESTMINSTER, CO and red circle ALERT 1310 Little Dry at 64th.

Figure 2. USGS LITTLE DRY CREEK AT WESTMINSTER, CO. In open straight engineered channel with park and bike path overbank.

Figure 3. USGS LITTLE DRY CRK BLW FEDERAL BLVD AT WESTMINSTER, CO. In wooded area by engineered channel and bike path.


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Figure 4. Flood Diversion Structure (circled in yellow on first photo above) 0.5 mi upstream of ALERT Station 1310 (station location in red in first photo above). Culvert/debris dam structure (circled in orange on second photo above) is designed only allow low flows to pass and to divert peak flows through energy dissipation channel on west to detention basin below. Low flows are conveyed to the eastern channel through covered ~2-3 ft diameter culvert.

Table 1. Little Dry Creek Annual Peak Flows (data from USGS 06719840 LITTLE DRY CREEK AT WESTMINSTER unless otherwise noted)

Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)
1982, unknown date	15.52[2]	954[2]
1983, unknown date	13.01[2]	420[2]
1984, unknown date	15.52[2]	954[2]
1885, unknown date	11.35	464[2]
1986, unknown date	10.92	368[2]
1987, unknown date	11.98	762[2]
6/10/1988	9[2]	255[2]
6/3/1989	12.14	659
5/29/1990	11.58	518
6/1/1991	13.09	**1280 (historical peak)**
8/24/1992	11.41	347
6/17/1993	11.92	560
4/25/1994	11.78	494
5/17/1995	12.32	759
9/18/1996	12.07	632
7/31/1997	12.77	1040
1998 data missing		
8/09/1999	11.58	431
8/17/2000	12.1	646
8/01/2001	11.88	547
7/06/2002	10.55	191
4/19/2003	10.66	207
8/18/2004	11.32	347
8/04/2005	10.8	230
8/13/2006	11.06	278
8/24/2007	11.1	286
2008 data missing gage discontinued 9/30/2007, re-established 4/2009		
7/20/2009	12.16	676
7/12/2010	11.79	463
7/08/2011	12.59	893
7/07/2012	11.21	270
6/28/2013	11.97	600
5/22/2014	11.52	444
5/08/2015	12.15	672
6/28/2016	12.17	680
5/26/2017	11.95	544
8/18/2018	11.67	446
7/05/2019 20:10 7/05/2019 20:40[1]	12.43 NA[1]	741 805[1]

[1] 2019 data also available for USGS 06719845 LITTLE DRY CRK BLW FEDERAL BLVD AT WESTMINSTER

[2] These points follow a different stage-discharge rating compared to the rest of peaks

Table 2. Little Dry Creek Annual Peak Flows at ALERT 1310 Little Dry at 64th. MHFD Annual peak spreadsheet data in ( ) if differences found. Differences were mostly due to rounding, although the peak spreadsheet was missing discharges for some years.

Date of Peak	Peak Stage (ft)	Peak Discharge (cfs)	Comment
5/24/2002	1.11	38 (53)	New peak discharge at original date/time
8/7/2003	1.03	31	No data in Annual Peak Spreadsheet
7/23/2004	2.12 (2.10)	193 (0)	Slightly different peak stage at original peak date/time, discharge value included
7/28/2005	1.3	57	No data in Annual Peak Spreadsheet
8/13/2006	1.28	55 (70)	New peak discharge at original date/time
7/20/2007 (6/29/2007)	1.54 (3.00)	88 (0)	New peak. Erroneous datapoints during previous peak day
6/30/2008	2.27	223	No data in Annual Peak Spreadsheet
6/14/2009	2.99	398 (0)	Peak discharge value included
4/24/2010	2.48	269 (153)	New peak discharge at original date/time
7/8/2011	3.10	428 (0)	**Historical Peak**, discharge value included
4/11/2012	1.72	117 (0)	Peak discharge value included
6/28/2013	1.99	169 (138)	New peak discharge at original date/time
7/30/2014	1.72	116	
6/4/2015	2.28	225	Negative stage data July-end
6/28/2016	2.29	227	Negative stage data June, July-end
5/26/2017	1.56	90	Frequent negative stage values
7/2/2018	1.56	90	Frequent negative stage values
7/5/2019	2.35	239	Frequent constant low stage values

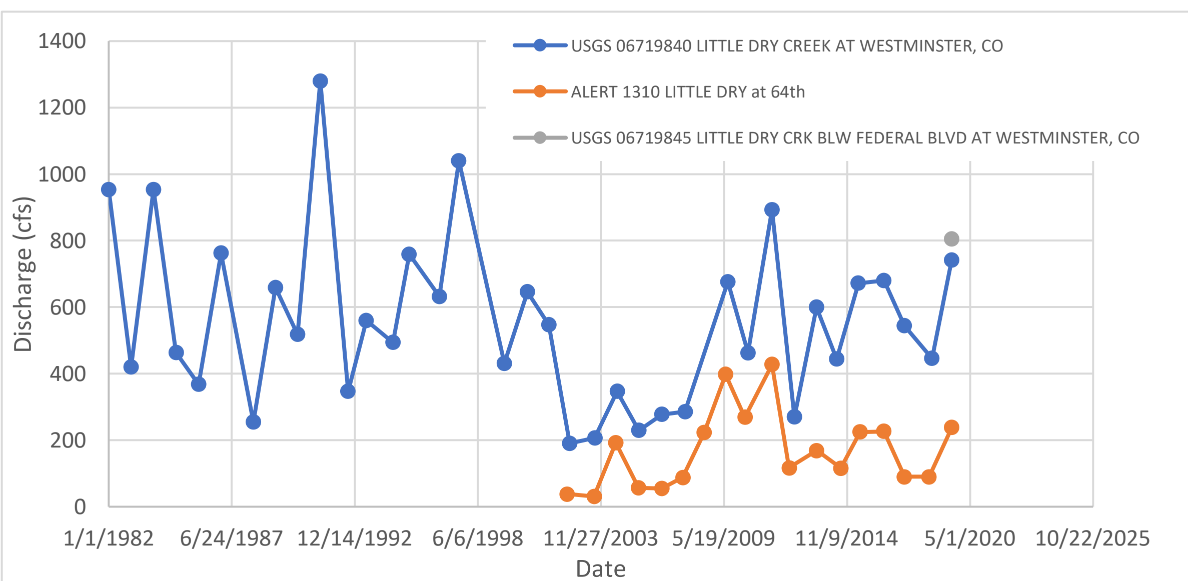

Figure 5. Comparison of Annual Peaks from Watershed Gages

USGS 06719840 stage-discharge peak data demonstrate some scatter indicating rating changes/shifts over time (Figure 6). Based on information from Greg Smith USGS, "early ratings were due to a different gage location upstream of a culvert (possibly upstream of 72nd) at a different datum. There is some vague information saying there was construction (maybe the channelization of Little Dry Creek) so the gage was moved downstream to its current location. There is about a 2-foot change in stage between the two different locations." The USGS believes the peak flow data are correct and the datum change is responsible for the differences in stage.

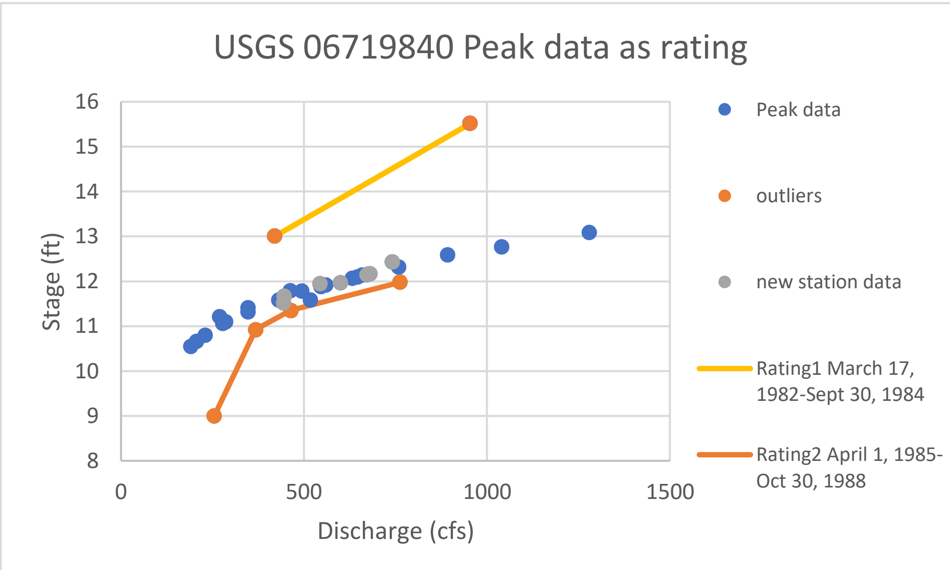

Figure 6. Stage-Discharge paired data for annual peaks

Annual peak data for ALERT 1310 displayed as stage-discharge rating demonstrate that all data follow the same rating relationship (Figure 7).

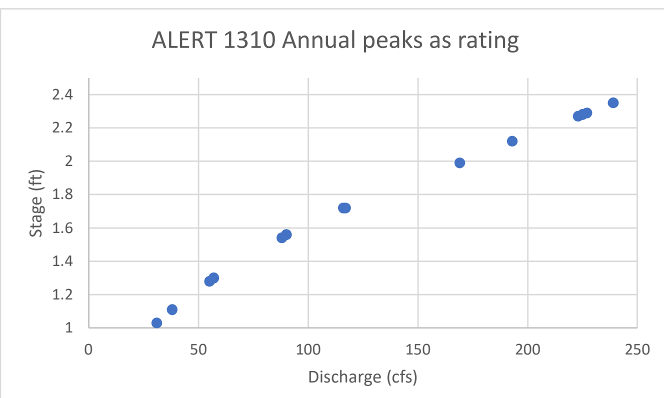

Figure 7. Stage-Discharge paired data for annual peaks for ALERT 1310

StreamStats Output

Streamstats peak flow analysis was performed for comparison with measured historical peaks at the two USGS gage locations and the mouth of Little Dry Creek (Table 3). Measured peaks are less than the 25-year frequency.

Table 3. StreamStats Peak Flow Analysis Output

		StreamStats Peak Flows						
Location	**Drainage Area (mi^2)**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**200-yr**	**500-yr**
USGS 06719840 LITTLE DRY CREEK AT WESTMINSTER, CO	10.5	637	1120	1990	2840	3920	5240	7370
USGS 06719845 LITTLE DRY CRK BLW FEDERAL BLVD AT WESTMINSTER, CO	12.4	721	1270	2250	3220	4450	5940	8370
Mouth of Little Dry Creek at Clear Creek	13.1	767	1350	2400	3430	4750	6350	8950

Recommendations

The dataset from USGS 06719840 LITTLE DRY CREEK AT WESTMINSTER, CO is the most complete hydrologic measurement of the flows in the Little Dry Creek near the outlet. The ALERT 1310 data are not comparable with these flows.

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