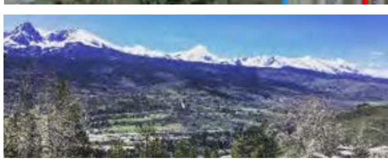
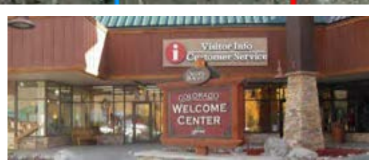
SILVERTHORNE COLORADO

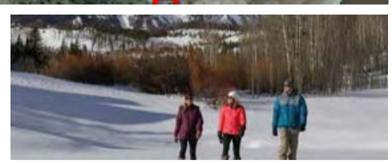
# Town of Silverthorne

# Drainage Master Plan

WRIGHT WATER ENGINEERS, INC.





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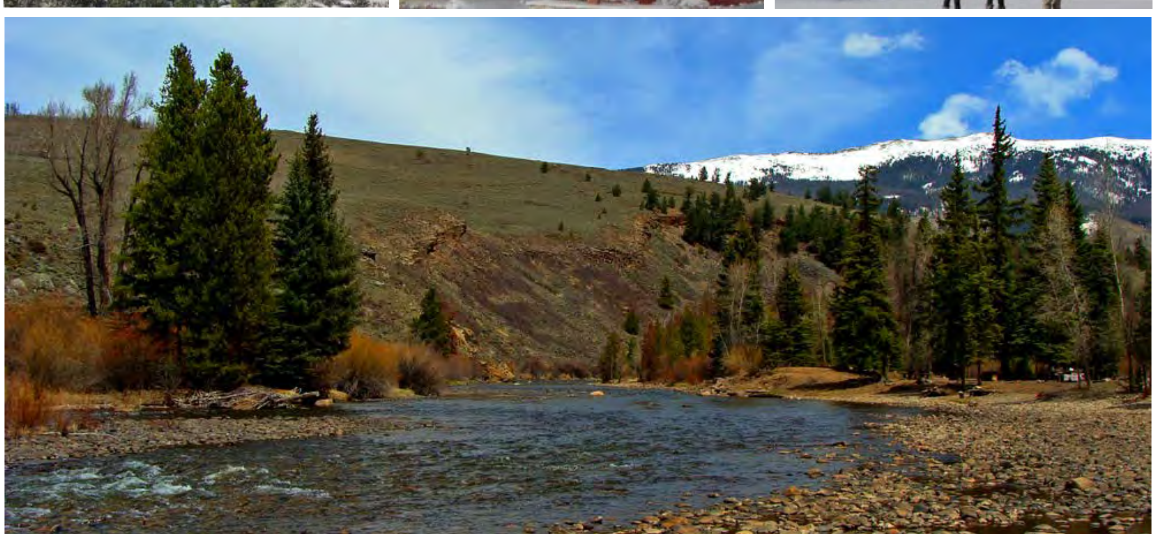

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# Town of Silverthorne

## Drainage Master Plan

July 2020

*Prepared for:*


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# EXECUTIVE SUMMARY

## ES.1 Purpose and Objectives

The Town of Silverthorne Drainage Master Plan (Plan) was updated to assess the function of the stormwater infrastructure throughout the Town of Silverthorne (Town) and identify potential multi-objective improvement projects and recommendations. The Plan provides the Town with a comprehensive analysis of the conveyance capacity and flood risk associated with major drainageways throughout the Town and includes opportunities for stormwater quality enhancement.

The Plan studied, evaluated, and documented the Town's current stormwater infrastructure through hydrologic and hydraulic modeling of storm events ranging from the 2-year to 100-year event. Wright Water Engineers, Inc. (WWE) used these analyses to understand infrastructure deficiencies, improvement opportunities, and prioritize stormwater projects. Site-specific improvement projects were prioritized and included in the Recommended Plan (Section 9 of the Plan), along with stormwater criteria and maintenance recommendations, water quality recommendations, and potential areas of future study.

## ES.2 Modeling Approach

A detailed hydrologic and hydraulic evaluation was completed for the major drainageways in Silverthorne using the U.S. Environmental Protection Agency's (EPA) Stormwater Management Model (SWMM). These analyses simultaneously accounted for rainfall and snowmelt, the two processes that generate runoff in the Study Area. Infrastructure capacity was assessed using previously surveyed information for the major drainageways throughout the Town. Ultimately, the hydrologic and hydraulic modeling identified deficiencies and opportunities for improvement.

## ES.3 Prioritized Projects

Seventeen (17) site-specific improvement projects were identified to address deficiencies found via the modeling analyses and interviews with Town staff. Identified projects were evaluated against multi-objective criteria, prioritizing projects that address multiple needs, including both stormwater capacity and flood risk issues along with water quality concerns.

## ES.4 General Recommendations and Next Steps

In addition to site-specific improvement projects, general recommendations and next steps are also included in the Plan to provide guidance in maintaining and developing future stormwater infrastructure.

The Plan emphasizes the need for routine maintenance of the drainage facilities throughout the Town. Maintenance activities would involve clearing culverts of sediment and debris, repairing or replacing damaged infrastructure, and stabilizing open channel drainageways. Routine maintenance would not only restore and maintain the design capacity of many of the Town's stormwater facilities but would also provide water quality benefits. The development of a stormwater utility fee or similar funding mechanism would help provide the necessary funding to complete routine maintenance activities and identified capital improvement projects.

In addition to routine maintenance, an update to the Town's Drainage Design Criteria will provide guidance for future development of stormwater infrastructure and help to control water quality. After review of the existing criteria, this Plan recommends that the Town further develop these standards to update design methodologies and practices such that they reflect the best available information and guidance. The Mile High Flood District's (MHFD) Urban Storm Drainage Criteria Manual (USDCM) could potentially be a good model for a criteria manual for the Town, along with updates to the recommended procedures for hydrologic analyses. The City of Aspen's Urban Runoff Management Plan is one example of a criteria manual that was developed by adapting the guidance in the USDCM to work in a mountain setting.

Continued study of hazards in Silverthorne would provide the Town with a more robust understanding of local risks. Potential areas of future study include the development of a detailed two-dimensional hydraulic model to understand the drainage flow paths of undersized major and minor drainageways, and an analysis of mud and debris flow risk throughout the Town.

# 1. INTRODUCTION

In 2019, the Town of Silverthorne (Town) authorized Wright Water Engineers, Inc. (WWE) to develop a drainage master plan for the Town. This work included the assessment of existing infrastructure through hydrologic evaluations of watershed basins and hydraulic analysis of stormwater drainage infrastructure within, and immediately adjacent to, the Town limits.

## 1.1 PURPOSE AND SCOPE

The Town of Silverthorne Drainage Master Plan (Plan) is a comprehensive planning document that evaluated the hydrology and hydraulics associated with the drainage infrastructure within the Town to identify potential deficiencies and proposed improvements, including:

- Quantification of stormwater runoff for various rainfall events (2-, 5-, 10-, 25-, 50-, and 100-year) from drainage basins in and around the Town.
- Identification of potential flood risks and stormwater infrastructure capacity issues for drainageways throughout the Town.
- Identification of potential improvements to address flood risk and infrastructure capacity issues throughout the Town.
- Identify opportunities to improve water quality of stormwater runoff.
- Evaluate the Town's stormwater funding needs and develop recommendations for a fee structure to meet those needs.

The scope of work completed as part of this Plan included the following:

- Review of existing stormwater literature and available drainage reports for the Town, including previous drainage master plans.
- Review of existing stormwater infrastructure data and determination of data gaps.
- Field verification of existing stormwater infrastructure data provided by the Town.
- Development of watershed hydrology for 2-, 5-, 10-, 25-, 50-, and 100-year return period storm events with inclusion of snowmelt events.
- Calibration of watershed hydrology with use of stream gages and local regression equations.
- Utilization of the Environmental Protection Agency (EPA) Stormwater Management Model (SWMM) to route hydrology from drainage basins through the Town's stormwater infrastructure to evaluate its existing capacity and identify areas of potential deficiencies.
- Development and prioritization of proposed improvements for identified infrastructure deficiencies.
- Identification of opportunities for stormwater quality improvements.
- Development of a maintenance program and fee structure to cover the costs of identified improvements and maintenance activities.

Hydrologic and hydraulic analyses were completed to inform recommended improvement opportunities throughout the Town. Though these peak flow estimates are useful for master planning purposes, they should not be relied upon for design. More detailed hydrology should be developed during the design and implementation of the recommended projects, including more detailed basin discretization from higher resolution topography and surveyed appurtenances. Further basin discretization and the utilization of different hydrologic methods can produce variable peak flow results.

### 1.2 MAPPING AND SURVEYS

The Town provided Geographic Information System (GIS) based aerial imagery and topography, as well as an inventory of existing stormwater infrastructure. This database contained fundamental information associated with the majority of the Town's storm drains and drainageways including: size and material (storm drains and culverts), locations of storm inlets and manholes, previous urban watershed delineations, and locations of stormwater detention facilities. Previously completed drainage studies and reports provided by the Town also included map exhibits and drawings which were used in conjunction with the GIS dataset to develop the hydrologic and hydraulic model.

Field verification of this information was completed for areas included in the hydrologic and hydraulic model; however, no additional survey data were collected as part of the development of the Plan. Though the Town's GIS information included the location of much of their stormwater infrastructure, surveyed invert elevations and storm drain slopes were not included in the dataset. Furthermore, the GIS dataset for the stormwater detention facilities did not include facility volume or outlet structure configuration.

### 1.3 DATA COLLECTION

The Town provided much of the background information required to complete this Plan. The data came in various forms such as reports and GIS databases, as well as interviews with Town staff. The information provided by the Town was verified, where possible, in the field by WWE. Below is a list of the data provided by the Town:

- Town's Topography provided as a Digital Elevation Model (DEM) with 1-meter by 1-meter grid cell resolution,
- Town's GIS storm water infrastructure inventory of storm drain, inlets, and ditches,
- Town's Drainage Design Criteria, and
- Interviews with Town staff regarding historical flooding and known existing drainage deficiencies.

## 1.4 ACKNOWLEDGEMENTS

WWE wishes to acknowledge the various Town staff as well as WWE colleagues for their support in developing this Plan.

- Tom Daugherty – Director of Public Works, Town of Silverthorne,
- Susan Pearson – Town Engineer & Floodplain Administrator, Town of Silverthorne,
- Scott Schreiber, PE – Project Manager, Wright Water Engineers,
- Dr. Andrew Earles, PE – Vice President and Peer Reviewer, Wright Water Engineers,
- Dr. Christopher Olson, PE – Senior Engineer, Wright Water Engineers,
- Drake Ludwig, PE – Project Engineer, Wright Water Engineers.

# 2. STUDY AREA

## 2.1 PROJECT AREA

Located in Summit County, Colorado, the Town is situated along the Blue River, just below the Dillon Reservoir. Incorporated in 1967, the Town continues to grow from its beginnings as a camp for workers building the Dillon Dam into a thriving community of roughly 4,000 residents. Nestled in the Blue River Valley between the peaks of the Rocky Mountain's Gore Range and Williams Fork Mountains, the Town is a popular stop for travelers along Interstate 70 (I-70) which runs through the southern end of the Town.

Downstream of the outlet from Dillon Reservoir, the Blue River meanders through the heart of the Town and is the ultimate receiving water for stormwater runoff in the area. The flow of the Blue River through the Town is heavily influenced by the operations of the Dillon Reservoir.

In addition to I-70 bisecting the southern end of Town, Colorado State Highway 9, otherwise known as the Blue River Parkway, runs predominantly along the west side of the Blue River. Much of the runoff generated from the western side of Town must cross Highway 9, via a series of existing culverts, before ultimately flowing into the Blue River.

There are a number of larger drainages that rely on the Town stormwater infrastructure to safely convey flow to the Blue River. Originating from the high elevations of the Gore Range to the west of Town are Ryan Gulch, Salt Lick Gulch, and Willow Creek. Similarly, Straight Creek is a tributary originating east of Town with its headwaters situated above the Eisenhower Tunnel in the Williams Fork Mountains. Of these tributaries, Straight Creek and Willow Creek are two of the largest with drainage areas of approximately 20 square miles and 13 square miles, respectively.

Small culvert crossings and storm drains make up the majority of the Town's stormwater infrastructure, conveying local flows through roadside swales before their outfall into the Blue River. Large culvert crossings, some of which convey stormwater underneath Highway 9, do exist for the larger drainages mentioned previously.

Historically, drainage through the Town Core area was conveyed to the north, parallel to Highway 9 via ditches and swales. Drainage from the west has historically been captured by the West Town Swale, concentrating a large drainage area to a highway culvert just south of the 13$^{th}$ Street intersection. The Bobo Ditch diverts water from the Blue River and conveys irrigation flow to the north along Rainbow Drive, on the east side of Town. In the absence of formal stormwater infrastructure on the east side of Town, this ditch conveys runoff from the east side of Town to the north, parallel to Highway 9.

The areas tributary to the Blue River include high gradient mountainous terrain ranging in elevation from roughly 13,350 feet above sea level along the peaks in the Gore Range to approximately 8,550 feet at the most downstream location of the Study Area. Due to the high elevation, a significant portion of the precipitation in this area includes snowfall which accumulates during the winter and melts during the warmer spring and summer months. As an average of 105 inches of snowfall per year melts, the resultant runoff is conveyed through the Town's stormwater infrastructure. Thus, both snowmelt and rainfall runoff must be simultaneously considered when evaluating stormwater infrastructure in this area.

The extents of the Study Area and major drainageways are outlined on the vicinity map, Figure 1. Map A.1 also provides an overview of the project extents.

## 2.2 LAND USE

The majority of the land use within the Town consists of medium to low density residential neighborhoods. Over the years, the Town has also become Summit County's central retail destination, including numerous commercial and high-density land uses through the Town Core along the southern end of Town. Outside of the Town limits, the Eagles Nest Wilderness area extends to the west, whereas the Town of Dillon, unincorporated Summit County, and the Ptarmigan Peak Wilderness border the Town to the east.

Vegetative cover within the Town limits predominantly consists of native shrubs and grasses, along with formally landscaped areas throughout both the residential and designated open space areas. Some stands of deciduous trees can be found in the northern limits of town. These stands turn into deciduous and evergreen forested areas outside of the Town boundary, as elevation increases. Deciduous and evergreen forests cover the mountains, climbing above the Town limits. High alpine regions of the tributary headwaters rise above timberline and exhibit rock outcroppings, seasonal snow coverage, and alpine lakes. The Blue River, Willow Creek, and Straight Creek, along with the other large tributaries, support healthy riparian vegetation along their banks and floodplain areas.

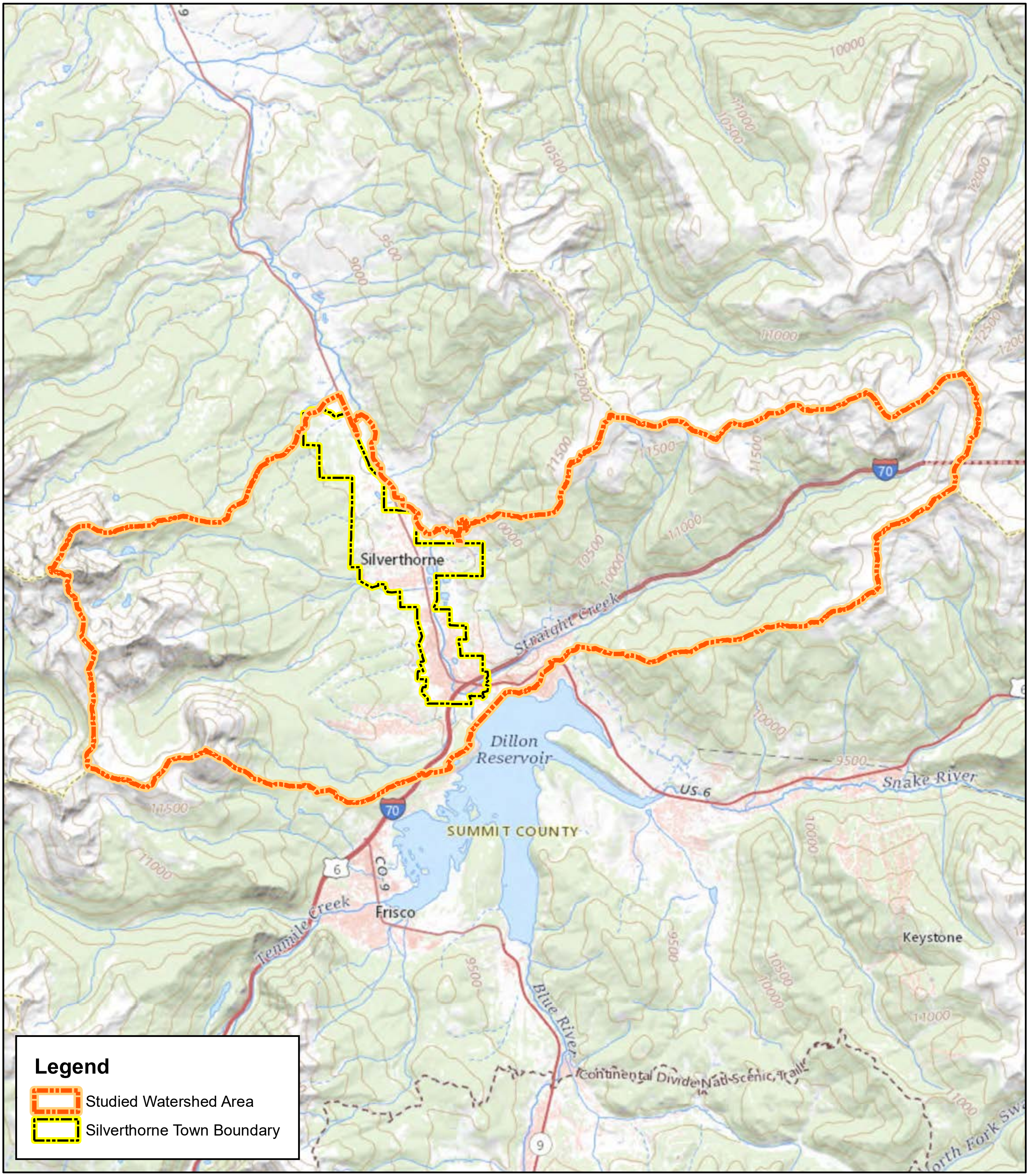
Roadways, parking lots, rooftops, and other impervious surfaces also cover the Town landscape, especially in the Town Core where many of the commercial and retail areas are located. Impervious surfaces account for roughly 16% of the area within the Town boundary. Map A.2 illustrates the dominant land uses within the Study Area.

## 2.3 FLOOD HISTORY

Based upon interviews with Town staff, there has been little documented flood history in the Town. According to a source interviewed for a 2019 article in the local newspaper, Summit Daily, flooding has been a concern for areas along Rainbow Drive, north of the Town.

The 2018 U.S. Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) report for Summit County states that major floods on streams with the County are caused by the melting of the winter snow accumulation with some influence from rainfall later in the season. This observation highlights the need for assessing the compound effects of both rainfall and snowmelt runoff processes when assessing the capacity of stormwater infrastructure in this area.

Since the construction of the Dillon Reservoir, the FIS report states that there have been no serious flood problems along the Blue River, downstream of the dam. The report also notes that floodwaters along Willow Creek have been known to breach Smith Ranch Road to the south of the creek, ponding behind Highway 9. Smith Ranch Road has recently been improved from a gravel drive to a paved collector, it is unknown if overtopping is still a concern following these improvements.

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SILVERTHORNE COLORADO

WWE WRIGHT WATER ENGINEERS, INC.

*TOWN OF SILVERTHORNE, CO*

# VICINITY MAP

SILVERTHORNE DRAINAGE MASTER PLAN

Base Map: USGS The National Map (October 2018)

0 5,400 Feet

FIGURE 1

## 2.4 KNOWN DEFICIENCIES

A need for a comprehensive understanding of the drainage patterns throughout the Town was the impetus for this Plan. Due to the age and condition of much of the Town's stormwater infrastructure, the existing drainage patterns and deficiencies through Town were not well understood.

Because there has been such minimal flood history documented within the Town, very few areas of known flood-related deficiencies have been previously identified within the Study Area. Previous study of the Smith Ranch community had highlighted potential capacity issues with the 24-inch storm drain beneath Highway 9, just south of Willow Creek. As development of this area progresses, alternatives to alleviate flood risk should be considered.

As mentioned previously, the West Town Swale currently concentrates flow from a large drainage area to a highway culvert just south of the intersection of Highway 9 and 13th Street. Not only does this drainage pattern increase the peak flows seen at this crossing, but also reduces the effective slope of the drainage infrastructure through Town, increasing the potential for sediment deposition. Creating flow paths for drainages west of the Town Core to more directly flow into the Blue River, rather than being routed to the north, will help to alleviate the hydraulic loading at the highway culvert and could reduce the potential for sediment deposition.

From field inspections and interviews with Town staff, ongoing maintenance of stormwater infrastructure was identified as a critical need. Many of the inspected culverts have been partially filled with sediment and/or damaged. The flared end sections and entrances to culverts were found to be smashed at some locations. Many of the existing culverts are made of corrugated metal and, therefore, are susceptible to abrasion from fast flowing, sediment laden stormwater.

Some of the larger culverts conveying stormwater from the west towards the Blue River are owned by Colorado Department of Transportation (CDOT) and require extensive coordination and resources to be repaired or replaced. Routine cleaning and monitoring of culverts and swales would help to maintain the infrastructure's design conveyance capacity.

There are also a number of smaller stormwater detention sites throughout the Town that require maintenance. The majority of these detention facilities have a simple weir outlet configuration, many of which are damaged or failing. Retrofitting these facilities to provide for full spectrum detention would provide water quality benefits and likely reduce peak stormwater flow rates downstream.

The Blue River through Town lost its Gold medal status in 2016. Anthropogenic alterations to stream flows, sparse aquatic invertebrate populations, low nutrient content, and degraded habitat have all been cited as reasons for the delisting. Though the Dillon Dam is responsible for the hydromodification of stream flows, the cause of the impairments is largely unknown. The health of the Blue River should be monitored, and upstream water quality control measures should be implemented to protect this fishery as it is one of the Town's central recreational attractions.

# 3. HYDROLOGIC ANALYSIS

To evaluate the existing capacity of the stormwater infrastructure in Silverthorne, a semi-distributed, event-based hydrologic model was developed in SWMM to simulate runoff produced from a range of rainfall and snowmelt events. Event-based modeling was applied to all tributaries to the Blue River within the Town. The hydrology of the Blue River is heavily influenced by the operations of the Dillon Reservoir, thus not included within the scope of the event-based hydrologic and hydraulic evaluation. However, the recently completed FEMA FIS report was included in this study by reference for hydrology, hydraulics, and flood risk/deficiency identification for the Blue River.

## 3.1 PREVIOUS STUDIES

Previous hydrologic studies performed for the Town were provided by the Town for review by WWE and are listed in Table 1 below. Most of these studies used either the Rational Method or the Natural Resource Conservation Service (NRCS) Curve Number and Unit Hydrograph methods for hydrologic analysis based on now-outdated storm distributions and rainfall depths. Though it is the primary process from which peak flows are generated in the high alpine streams of Silverthorne, snowmelt runoff had not been included in any of the previous studies.

A number of the previous plans also included design documents which were reviewed to identify flow paths and existing infrastructure.

***Table 1. Summary of Previous Studies***

| Study | Study Completed By | Year of Study |
|---|---|---|
| Silverthorne Drainage Master Plan | Wright-McLaughlin Engineers | 1976 |
| Drainage Master Plan for the Town of Silverthorne | McLaughlin Water Engineers | 1983 |
| Preliminary Drainage Report – Silver Mountain Village Planned Unit Development | David L. Kotzebue | 2001 |
| 2001 Drainage Master Plan Update | Farnsworth Group | 2001 |
| Master Drainage Plan Phase I Final Plat Submittal – South Maryland Creek Ranch | Tetra Tech | 2016 |
| Preliminary Drainage Report – Smith Ranch | Wright Water Engineers, Inc. | 2018 |
| Final Drainage Report – Alpine Lumber at Moorlag Subdivision Lots 1 and 2 | Galloway & Company, Inc. | 2019 |

### 3.2 MODELING SOFTWARE

SWMM was used to model and evaluate the Town's existing stormwater infrastructure. SWMM is an open source, public domain model. Version 5.1.013 of the SWMM model engine was used to complete this analysis. To aid in model development, PCSWMM version 7.2, a proprietary software developed by Computational Hydraulics International (CHI), was used. PCSWMM is designed to provide a more user-friendly interface to the SWMM environment, while maintaining the underlying SWMM open source engine (SWMM version 5.1.013). PCSWMM includes improved tools for model development that are not available in SWMM, while maintaining compatibility with the public software. The models developed as part of this Plan are thus compatible with the free SWMM software and also can be used in PCSWMM.

### 3.3 SUBBASIN PARAMETERIZATION

To convert rainfall into runoff, subbasin characteristics were specified in the hydrologic model to reflect existing watershed conditions. These model inputs account for processes including depression storage, infiltration, and the conversion of excess precipitation into surface runoff.

SUBBASIN DELINEATION

Utilizing the DEM provided by the Town and the automated delineation tool in PCSWMM, the Study Area was discretized into subbasins with an average area of 250 acres. The delineation was then checked and refined as necessary for the high alpine watersheds outside of the Town limits. Manual refinement and further discretization of subbasin boundaries provided the level of detail necessary to appropriately evaluate the capacity of critical stormwater infrastructure throughout the Town. Delineations from previous studies, existing infrastructure, and topographic data were used to refine the subbasin delineations in the developed areas. Subbasins were divided such that spatial variability in land cover and topography were accounted for. The hydrologic analysis includes 224 subbasins for a total drainage area of approximately 49.4 square miles (average subbasin size of 140 acres after refinements).

IMPERVIOUSNESS

SWMM partitions subbasins into pervious areas, such as lawns and open space, and impervious areas, such as roofs and roads, to differentiate between the surface runoff processes for each. The percent of imperviousness for a given subbasin is largely dependent on the land use of that area. The U.S. Geologic Survey (USGS) develops and releases a nationwide landcover and imperviousness dataset, the National Land Cover Database (NLCD), which was leveraged for parameterization of many subbasin variables. A spatial average of the impervious areas included in the 2016 NLCD was used to assign the percent imperviousness for each subbasin.

The impervious coverage dataset is illustrated on Map A.2. For areas with impervious surfaces, the dataset identifies the percentage of impervious cover. In addition to spatially averaging these percentages, areas outside of the delineation were assumed to have a 2% imperviousness value. From these values, an approximate spatial average was then estimated for each subbasin. Subbasins within the Town boundary have levels of imperviousness ranging from 2% to 79% with an area-weighted average imperviousness of 16%.

INFILTRATION

Infiltration losses associated with the pervious fraction of each basin are accounted for via Horton's infiltration model. At the onset of a storm event, rainfall infiltrates into the soil at the soil's maximum infiltration capacity; however, as precipitation continues, the ability of the soil to store additional water decreases at an exponential rate under the Horton model. As the rate of rainfall exceeds the soil's capacity for infiltration, the excess rainfall fills depression storage areas (discussed in following section) and ultimately becomes surface runoff.

Infiltration input parameters for the Horton method are dependent on soil type, specifically the NRCS Hydrologic Soil Group (HSG). Though soils from each HSG exist within the Study Area, the majority of soils fall into HSG B, with a substantial amount of HSG C soils present within the Town boundary. Soils classified as HSG C and D tend to produce larger volumes and increased rates of runoff than HSG A and B soils. The extent of each soil group is shown on Map A.3. The values recommended in the Mile High Flood District's (MHFD) Storm Drainage Criteria Manual (USDCM, 2018) were used for model inputs and are summarized in the table below. Soils with a designated HSG of A/D were conservatively assigned infiltration rates of HSG D soils. For each subbasin, a spatial average of these values was taken from the existing soil survey and included as input into the hydrologic model.

***Table 2. Horton Infiltration Parameters by Hydrologic Soil Groups (MHFD, USDCM, 2018)***

| Horton Infiltration Parameter | HSG A | HSG B | HSG C and D |
|---|---|---|---|
| Maximum Infiltration Rate (inches/hour) | 5 | 4.5 | 3.0 |
| Minimum Infiltration Rate (inches/hour) | 1.0 | 0.6 | 0.5 |
| Decay Constant (hours$^{-1}$ ) | 2.52 | 6.48 | 6.48 |

DEPRESSION STORAGE

Depression storage accounts for the fraction of rainfall that is captured by small irregularities in the topography or depressions within each subbasin such that it infiltrates or evaporates prior to becoming surface runoff. Along with infiltration inputs, SWMM requires the input of typical depression storage values by land use, referenced from the MHFD's USDCM and summarized in the table below, to calculate the effective rainfall resultant from any given storm event. Using the landcover data included in the 2016 NLCD, depression storage values were spatially averaged for each subbasin.

Surface runoff is generated when the total depth of water, rainfall minus infiltration, exceeds the depth associated with the depression storage for a given subbasin. The excess precipitation is then converted into a runoff hydrograph via a transform method described in the following section.

***Table 3. Depression Storage Values by Land Use (MHFD, USDCM, 2018)***

| Description | Depression Storage |
|---|---|
| Impervious Areas | |
| Roofs-Sloped | 0.05 inches |
| Roofs-Flat and Large Paved Areas | 0.10 inches |
| Pervious Areas | |
| Lawn Grass | 0.35 inches |
| Wooded Areas and Open Fields | 0.40 inches |

TRANSFORM METHOD

Once the precipitation exceeds the infiltration capacity of the underlying soil and fills the depression storage, it becomes surface runoff. Excess precipitation is converted into a runoff hydrograph for each subbasin based upon its unique characteristics, including its; shape, dimension, slope, and surface coverage. As discussed below, SWMM uses these characteristics to create a simplified representation of each subbasin to develop a hydrograph for the surface runoff generated from that subbasin.

SWMM converts excess precipitation into surface runoff via a nonlinear reservoir model where each subbasin is conceptualized into a rectangular surface with a uniform slope and width. Using the provided DEM, the average slope of each subbasin was calculated in PCSWMM for input into the model. The width of the conceptualized plane was also calculated in PCSWMM and taken as the quotient of the subbasin area divided by the length of overland flow. For large, undeveloped subbasins, the overland flow length was assumed to be 200 feet, after which flow becomes concentrated.

SWMM further assumes that flow across the idealized subbasin surface behaves like uniform flow within a very wide rectangular channel. Thus, Manning's equation can be used to compute the flow rate produced by each subbasin. Typical Manning's roughness values to simulate overland flow in this manner differ from those used for hydraulic routing, such as through a culvert or in a channel, due to the shallow nature of overland flow. Recommended values are published in the EPA SWMM Reference Manual. The values used in this hydrologic analysis are assigned by land use and summarized in the table below. Similar to the imperviousness and depression storage, the 2016 NLCD was used to spatially average these Manning's roughness values by land use throughout the Study Area.

***Table 4. Manning's 'n' Values for Overland Flow by Land Use (EPA SWMM Reference Manual)***

| Description | Manning's 'n' |
|---|---|
| Impervious Areas | |
| Suburban Residential Land Use | 0.055 |
| Semi-Business Land Use | 0.035 |
| Pervious Areas | |
| Parks and Lawns | 0.075 |
| Shrubs and Bushes / Timberland | 0.100 |

## 3.4 GAGE ANALYSIS

Two stream gages are currently operated on drainageways through the Study Area, on Straight Creek and the Blue River. Historical gage data is also available on Willow Creek. Each gage has been, or is currently, operated by the USGS in cooperation with Denver Water. Historical streamflow records collected by stream gages on Straight and Willow Creeks provided insight into the characteristics of the studied watersheds, including the dominant mechanism behind peak flow events, and estimated magnitude and likelihood of those events. As previously mentioned, flow in the Blue River through Silverthorne is heavily influenced by the outlet operations from the Dillon Reservoir, thus not representative of event-based runoff responses.

For Straight and Willow Creeks, a flood frequency analysis was completed to gain an understanding of the relative frequency and magnitude of flood events based upon historically observed peak flow events. Daily discharge data were used to investigate the seasonal trends in streamflow to develop a more robust understanding of the processes that drive peak flow events throughout Silverthorne. A drainage that exhibits high flow during the spring and summer months is likely to have peak flow events that are driven primarily by snowmelt runoff, whereas a daily discharge record that does not exhibit this trend is likely to have peak flow events that are primarily driven by rainfall.

FLOOD FREQUENCY ANALYSIS

WWE performed a flood frequency analysis using the streamflow data obtained from both the Straight and Willow Creek gages to compare the estimated peak flow rate from the hydrologic model to that of observed events. Because the scope of the hydrologic model did not include the Blue River, a similar comparison could not be made at that gage location.

The Straight Creek below Laskey Gulch near Dillon, CO gage (USGS09051050) is located outside of the Town limits, near the Deer Path Road crossing. The gage has a drainage area of 18.3 square miles and is at an elevation of 9,070 feet. The gage has a 33-year period of record (1987-2019) from which a flood frequency analysis was performed using the methodology outlined in the USGS Bulletin 17B (Bulletin 17B). Bulletin 17B is a guidance document that outlines a statistical method for determining flood flow frequencies from historical peak flow records. A minimum period of record of over 25-years is sufficient to make reasonable estimates of the 100-year event using this methodology. The flood frequency analysis was performed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's Statistical Software Package (HEC-SSP) version 2.2. This software includes the statistical method outlined in Bulletin 17B for determining flood flow frequency. The analysis was repeated for the Willow Creek near Dillon, CO gage (USGS09051500) located just upstream of the Highway 9 crossing. This gage has a 9-year period of record from 1942-1951, a 13.4 square mile drainage area, and is at an elevation of 8,690 feet. The period from 1942 to 1951 was a relatively wet period throughout Colorado, thus gaged peak flow rates may have been relatively high while this gage was in operation. Because the Willow Creek gage has a period of record of less than ten years, the reliability of the flood frequency estimation is less than that of a gage with a longer period of record, and flood estimates for the 10-year event or larger likely have a high degree of uncertainty. That being said, a flood frequency analysis was completed for the Willow Creek gage for comparison purposes only.

Consistent with Bulletin 17B methodology, the Single Grubbs-Beck Low Outlier Test and Weibull plotting positions were utilized to develop flood estimates for a range of events. To account for irregularities in the peak flow frequency distribution, a skew coefficient is required to develop a frequency curve that best represents the sample data. To determine which skew value provided the best representation, the test was run three different times: once using the station skew coefficient as provided by USGS, once using the regional skew coefficient as provided by the Generalized Skew Coefficients of Logarithms of Annual Maximum Streamflow chart (Regional Skew Chart) shown on Plate 1 of Bulletin 17B, and once using the weighted skew coefficient as calculated by HEC-SSP from the provided station and regional skew coefficients. The regional skew coefficient was determined to be -0.55, and a mean square error of 0.302 is provided for all regional skew coefficients on the Regional Skew Chart. The station skew coefficients provided by USGS for the Straight and Willow Creek gages are -0.306 and 0.254 with mean square error values for each being 0.181 and 0.537, respectively. For each gage, the three computed curves were compared and it was determined that station skew provided the optimal computed curve out of the three.

The resulting computed flows for Straight Creek and Willow Creek are provided in the tables below. As a final check for reasonableness of the computed flow frequency estimates, the flow rates published in the latest FEMA FIS report for Summit County are also included these tables as are flow estimates from the USGS StreamStats application which are based upon regional regression equations.

***Table 5. Straight Creek Flood Frequency Summary Table***

| Return Period | Percent Chance Exceedance | 2018 FEMA FIS (cfs) | USGS StreamStats - Regional Regression Equations (cfs) | Bulletin 17B Computed Flow Estimate (cfs) |
|---|---|---|---|---|
| 2-Year | 50% | - | 170 | 130 |
| 5-Year | 20% | - | 240 | 200 |
| 10-Year | 10% | 374 | 290 | 260 |
| 25-Year | 4% | - | 340 | 320 |
| 50-Year | 2% | 478 | 410 | 370 |
| 100-Year | 1% | 522 | 450 | 420 |

***Table 6. Willow Creek Flood Frequency Summary Table***

| Return Period | Percent Chance Exceedance | 2018 FEMA FIS (cfs) | USGS StreamStats - Regional Regression Equations (cfs) | Bulletin 17B Computed Flow Estimate (cfs) [1] |
|---|---|---|---|---|
| 2-Year | 50% | - | 140 | 140 |
| 5-Year | 20% | - | 190 | 170 |
| 10-Year | 10% | 309 | 230 | 190 |
| 25-Year | 4% | - | 280 | 210 |
| 50-Year | 2% | 394 | 330 | 230 |
| 100-Year | 1% | 430 | 360 | 250 |

[1.] The limited 9-year period of record is not sufficient for reliable flood flow estimates on Willow Creek. The results of the flood frequency analysis are included for comparison purposes only.

As shown in Tables 5 and 6, the flood frequency analyses for Straight and Willow Creeks generally estimated peak flow rates to be somewhat lower than those derived from regional regression equations and those included in the FIS report. From this comparison, it was determined that the flood estimates computed via the regional regression equations were generally in-line with those computed in the flood frequency analyses with an additional factor of safety, and could be appropriate for use in model calibration.

DAILY DISCHARGE STATISTICS

Similar to the rest of the Study Area, the Straight and Willow Creek gages are located within a watershed that is influenced by runoff generated from snowmelt. Because the flood frequency analysis was developed from peak annual flow events, an understanding of the influence of snowmelt on these peak events is necessary in identifying the dominant hydrologic process behind flooding in this area. To make this determination, the daily discharge statistics were reviewed for seasonal trends. Streams that exhibit seasonally high flows in the Spring often have annual peak flows that are influenced by snowmelt runoff. Figure 2 below illustrates the strong seasonal trends recorded for the Straight Creek gage from 2010 to 2020, indicating that snowmelt runoff accounts for a significant fraction of the observed peak flows in this watershed, as well as others tributary to the Blue River.

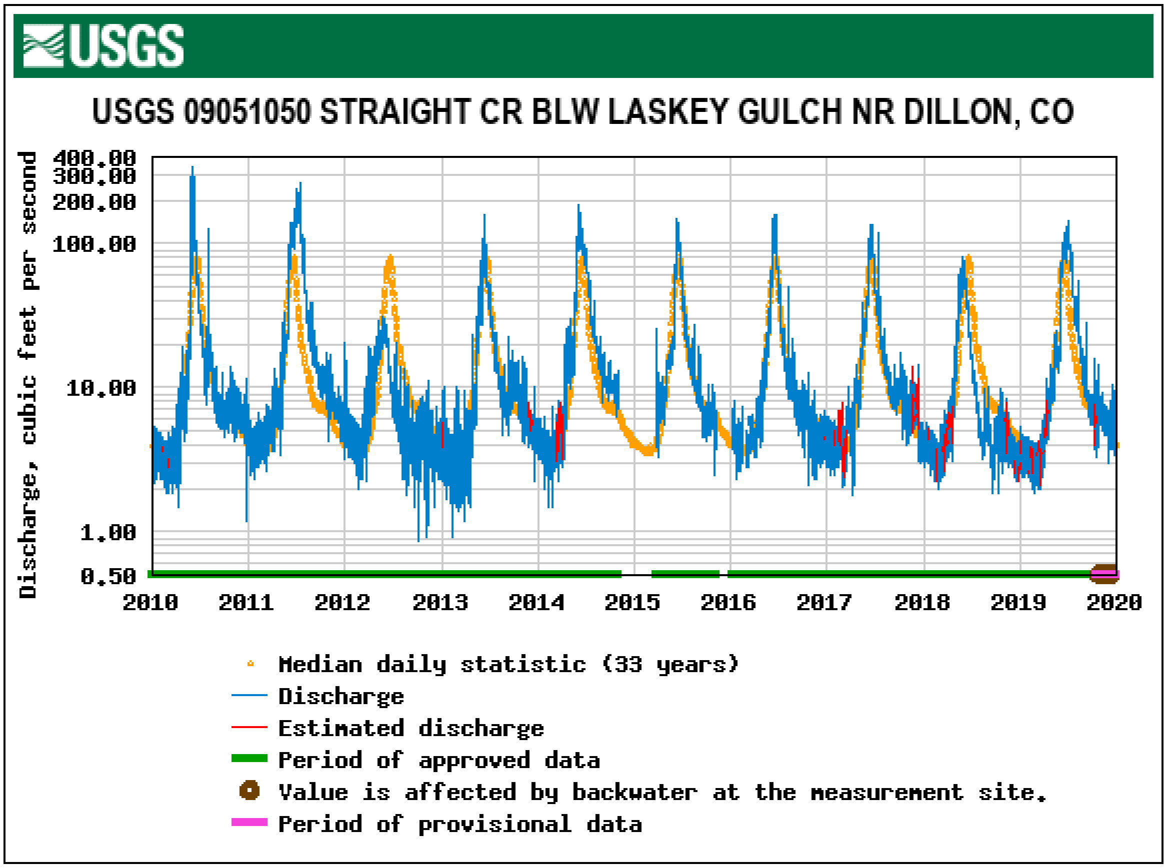


***Figure 2. Seasonal Trends in Daily Discharge Records for the Straight Creek Gage***

### 3.5 RAINFALL

The design rainfall utilized for hydrologic analysis was adapted from the U.S. National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 8 (NOAA, 2013). Six-hour point precipitation depths for the selected storm events were sampled from a central location within the Town for applicability to future studies. These precipitation depths are summarized in Table 7 below. Many of the previously completed drainage studies were developed on a previously published NOAA rainfall atlas that is no longer applicable. For example, there has been roughly a 15% decrease in 6-hour rainfall depths between NOAA Atlas 2, and the latest NOAA Atlas 14. This is due to analysis of a longer period of record associate with the NOAA Atlas 14 analysis.

***Table 7. NOAA Atlas 14, 6-Hour, Point Precipitation Depths for the Town of Silverthorne***

| Return Period | 2-Year | 5-Year | 10-Year | 25-Year | 50-Year | 100-Year |
|---|---|---|---|---|---|---|
| Precipitation Depth (inches) | 0.76 | 0.91 | 1.05 | 1.29 | 1.49 | 1.71 |

Temporal rainfall distributions developed for Climate Region 2, Western Colorado, are based on the Temporal Distributions of Heavy Precipitation analysis presented in NOAA Atlas 14 (NOAA, 2013). The distributions published in this document are based upon a probabilistic approach to analyzing temporal rainfall event data first introduced by Floyd Huff (Huff, 1967). The rainfall events analyzed for the development of storm distributions for this climate region were based on real rainfall events observed in western Colorado, thus providing the most up-to-date and region-specific data available to the Town. The 2-hour temporal storm distribution used in the Colorado Unit Hydrograph Procedure (CUHP) has been found to produce peak flow estimates that are significantly higher than gaged flow estimates for larger events in nearby communities, thus was not used in the hydrologic evaluation for this Plan.

NOAA Atlas 14 includes temporal distributions for events of varying duration, including: 6-, 12-, 24- and 96-hour storm durations. To represent the nature of the convective, shorter-duration storms that tend to produce localized flooding in Colorado, the 6-hour duration was selected for hydrologic analysis.

The analysis of regional rainfall events by NOAA produced a total of 36 representative temporal storm distributions for each of the aforementioned durations. These distributions are grouped by quartile and occurrence probabilities. First-quartile distributions include events where the greatest percentage of the total precipitation fell during the first quarter of the storm duration. Similarly, second-, third- and fourth-quartile distributions were developed from events with the greatest percentage of rainfall occurring in each of the respective quarters.

Distributions are further divided into occurrence probabilities for each quartile, ranging from 10% to 90%, in 10% increments, with the 50% distribution representing the median rainfall pattern. In this manner, NOAA Atlas 14 provides numerous temporal rainfall distributions; however, substantive guidance regarding the selection of a distribution is not included in the document. The median rainfall distribution is the least intense distribution, with intensities increasing towards the 10% and 90% distributions.

A report published by James Bonta in Applied Engineering in Agriculture, titled 'Development and Utility of Huff Curves for Disaggregating Precipitation Amounts' (Bonta, 2004), states that, "Selection of a particular percentage curve is arbitrary, and a rational basis for the selection of a particular isopleth is not available." The report does state, however, that Huff had suggested the use of the median, 50% storm distribution, within the quartile that is most frequently occurring for a given area. Still, Huff also acknowledged the utility of 10% or 90% curves for estimating runoff resultant of more intense events.

Because minimal guidance has been produced in regard to the selection of the appropriate distribution for a given region and duration, the distribution that best represented the hydrologic processes of the Study Area and produced peak flow estimates that reasonably fit the flood frequency analyses in the area, as outlined in Appendix D, was chosen for use in the hydrologic analysis.

A second quartile distribution was selected for hydrologic analysis to represent the effects of the leading edge of a rainfall event on the soil's antecedent moisture condition prior to the onset of the storm's most intense rainfall. In other words, the relatively less-intense rainfall that falls during the first quarter of the storm begins to infiltrate and fill depression storage areas. As the ground becomes saturated and depressions begin to fill, the most intense rainfall, falling during the second quartile of the event, loses less water to infiltration and depression storage, generating more surface runoff. The temporal storm distribution used for the hydrologic modeling of this study is provided in Table 8 below.

***Table 8. NOAA Atlas 14, 6-Hour, Second-Quartile 10% Temporal Distribution***

| Time (hours) | Cumulative Percentage of Total Precipitation | Time (hours) | Cumulative Percentage of Total Precipitation |
|---|---|---|---|
| 0:00 | 0.0% | 3:30 | 99.3% |
| 0:30 | 9.5% | 4:00 | 100.0% |
| 1:00 | 21.2% | 4:30 | 100.0% |
| 1:30 | 42.1% | 5:00 | 100.0% |
| 2:00 | 66.4% | 5:30 | 100.0% |
| 2:30 | 85.3% | 6:00 | 100.0% |
| 3:00 | 95.6% | | |

### 3.6 SNOWMELT

The daily discharge data for the Straight Creek gage indicate that peak flow events are often generated from both snowmelt and rainfall runoff processes. Therefore, the hydrologic analysis included both a design rainfall event as well as the application of snowmelt runoff rates for each return period.

Though the Town's Drainage Design Criteria does not currently include information regarding snowmelt runoff, the stormwater criteria for the Town of Vail does provide unit rates of snowmelt runoff by return period, summarized in Table 9. As shown in the daily discharge gage analysis, peak flow events for high alpine watersheds occur during runoff season and are often resultant of a rainfall event; therefore, modeling the compound effects of snowmelt and rainfall runoff is necessary.

***Table 9. Unit Rates of Snowmelt Runoff (Town of Vail Criteria)***

| Return Period | Unit Rate of Snowmelt Runoff (cfs/ac) |
|---|---|
| 2-Year | 0.040 |
| 5-Year | 0.048 |
| 10-Year | 0.060 |
| 25-Year | 0.067 |
| 50-Year | 0.072 |
| 100-Year | 0.080 |

An analysis of NRCS Snow Telemetry (SNOTEL) data was completed to verify the appropriateness of the inclusion of snowmelt runoff in the hydrologic analysis. Elevation and snow depth records were reviewed and compiled for SNOTEL stations within close proximity to the Study Area.

Figure 4 below illustrates the frequency of which an inch or more of snowpack was observed during the months of May and June, peak snowmelt runoff season, at each station from 2010 to 2019. Based upon this analysis, it was determined that subbasins with elevations of 9,000 feet or higher have sufficient spring snowpack such that snowmelt runoff is likely to be a contributing factor in peak runoff to the Town's stormwater infrastructure.

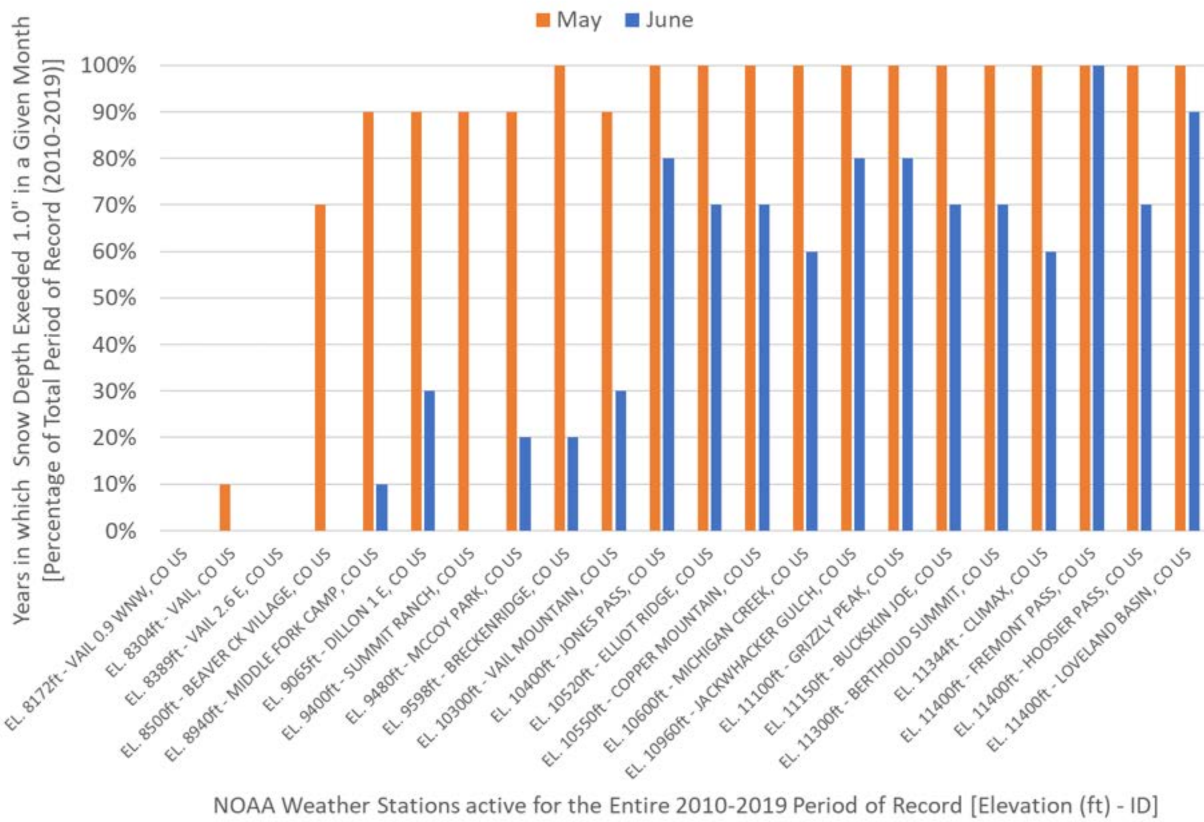


***Figure 3. Observed Snow vs. Elevation During Runoff Season***

The hydrologic analysis completed as part of this Plan accounts for both rainfall and snowmelt runoff processes simultaneously through the application of the published unit rates of snowmelt runoff and an adjustment to subbasin imperviousness for areas at or above 9,000 feet. A 50% increase to the calculated imperviousness was made for all subbasins above this elevation to account for the loss of infiltration capacity due to partial snow coverage. The rainfall-runoff model thereby accounts for snow coverage as well as the snowmelt processes that are present during the rain-on-snow events that drive peak flows through Town.

## 4. HYDRAULIC ANALYSIS

Using the stormwater infrastructure dataset provided by the Town, critical channel and culvert elements were included in the SWMM model to route the modeled hydrographs through the existing stormwater infrastructure in Silverthorne. Routing the computed hydrographs through these elements provides insight into potential capacity deficiencies within the Study Area. Because detailed stage-storage and outlet configuration data were not readily available for the Town's detention facilities, they were not included in the model. Map C provides a detailed routing schematic outlining the studied subbasins and routing elements included in the model for the Town.

### 4.1 ROUTING METHOD

Hydrographs generated by upstream subbasins are routed through the studied infrastructure elements via the kinematic wave method. This method was chosen since it is generally best suited for steep channel reaches and is the industry standard for master planning analyses where backwater effects and pressurized flow are not a significant consideration. The kinematic wave method solves the continuity and simplified momentum equations in each routing element, making it a numerically stable routing method. From prior experience, the dynamic wave solution generally indicates that drainage elements had more capacity than was calculated with the kinematic wave solution, producing results that were ultimately less conservative.

To account for uncertainties and provide a level of conservatism, the kinematic wave was confirmed to be the most appropriate routing method. Furthermore, the USDCM and other municipalities recommend the use of kinematic wave routing for all master plan studies. Kinematic wave routing parameters include the following channel reach characteristics: channel length, slope, dimension, and roughness estimates.

### 4.2 PARAMETERIZATION OF ROUTING ELEMENTS

The Town's GIS-based stormwater data set and previous drainage studies were utilized to identify the location, extents, and characteristics of critical drainage paths and culvert crossings to be analyzed for the evaluation of their existing capacity.

The slope and length of each routing element, culvert or channel, were parameterized in a GIS environment. The length of the digitized conveyance element was measured and extracted for model input. Similarly, the DEM elevations at the up- and downstream limits of each element were sampled to determine the slope of said drainage structure. It is assumed that this method provides an appropriate approximation of the channel slope for master planning purposes; however, the slopes of closed conduit infrastructure will need to be verified during the design phase of the recommended improvements.

For capacity computations, it was also assumed that all storm drains and culvert crossings were clean and free of sediment and debris. As identified in the field, a number of culverts are partially or fully clogged with sediment or debris. However, measuring and modeling reduced culvert capacity due to sedimentation was outside of the scope of this study. A robust maintenance plan for all of the Town's stormwater infrastructure is critical to keeping these drainageways functioning, upholding the assumptions made in this analysis.

For closed conduit flow, the culvert size or dimension was referenced from the infrastructure data set provided by the Town, where available. This information was cross-referenced with previous drainage studies to verify infrastructure location, extent, material, and size. At locations where these datasets did not include culvert size and/or material information, the data was collected via field reconnaissance.

For swales and open drainageways, a representative cross section was sampled from the DEM for model input. Manning's roughness values were then assigned for the channel as well as the left- and right overbank areas for each drainage element. The table below provides a summary of typical roughness values used in the evaluation. Channel velocities were also checked for reasonableness, and adjustments were made to the Manning's roughness values for open channel elements with velocities in excess of eight feet per second. Similar to the modeled culverts, open channel flow paths through Town were verified in the field to the maximum extent practical.

***Table 10. Manning's 'n' Values for Closed Conduit and Open Channel Flow (EPA SWMM Reference Manual)***

| Description | Manning's 'n' |
|---|---|
| Closed conduit | |
| Corrugated Inner Walls (CMP / HDPE) | 0.024 |
| Smooth Inner Walls (RCP / RCBC) | 0.013 |
| Open Channel | |
| Natural Stream / Drainageway / Roadside Swale | 0.050 |
| Roadway / Curb and Gutter | 0.020 |
| Light Vegetation | 0.070 |
| Medium to Dense Vegetation | 0.100 |

# 5. MODEL VERIFICATION AND CALIBRATION

A comprehensive comparison of this hydrologic modeling approach was completed for a nearby watershed tributary to Gore Creek through the Town of Vail. As the Study Area for this Plan and the Vail watershed both include headwaters originating in the Gore Range, and are within the same hydrologic region, it was determined that this comparison was also applicable to the Town.

Using the same modeling approach and input parameters outlined in the sections above, the Vail model was run to compare the computed runoff against the observed, gaged runoff, for a gaged precipitation event. This analysis verified that the hydrologic approach outlined in this Plan produced reasonable results for application in the identification of infrastructure deficiencies. More information regarding these analyses can be found in Appendix D of this Plan.

## 5.1 AREAL REDUCTION FOR SNOWMELT RUNOFF

A comparison between the modeled event runoff, accounting for both rainfall and snowmelt, and the computed flood frequency flow rates at the Straight and Willow Creek gages was completed for a range of return intervals. The comparison generally indicated that the model flow rates were higher than those computed in the flood frequency estimate, summarized in Section 3.4 of this Plan.

The computed flow estimates were higher due to the large drainage areas contributing to each gaged location, relative to the smaller basins upstream of most infrastructure through Town. To account for the areal reduction effect of these two larger basins, the unit rates of snowmelt runoff were reduced for the subbasins contributing to Straight and Willow Creeks. Unit rates of snowmelt runoff were reduced such that the modeled peak estimate, with both rainfall and snowmelt runoff, matched the flood flow estimates calculated from the regional regression equations. The updated unit rates of snowmelt runoff for subbasins in the Straight Creek and Willow Creek watersheds are included in the table below. The unit rate of snowmelt runoff was unmodified elsewhere as other drainage areas are relatively smaller and do not justify an areal reduction factor.

***Table 11. Unit Rates of Snowmelt Runoff with Areal Reduction Factor***

| Return Period | Unit Rate of Snowmelt Runoff (cfs/ac) | |
|---|---|---|
| | Straight Creek | Willow Creek |
| 2-Year | 0.004 | 0.009 |
| 5-Year | 0.008 | 0.015 |
| 10-Year | 0.01 | 0.018 |
| 25-Year | 0.011 | 0.021 |
| 50-Year | 0.014 | 0.025 |
| 100-Year | 0.014 | 0.026 |

# 6. IDENTIFICATION OF INFRASTRUCTURE DEFICIENCIES

Capacity deficiencies were identified via the SWMM analysis and review of the latest FEMA FIS. Storm drain systems, major roadway crossings, and open channel conveyance elements were all evaluated. Model results are illustrated on Maps D.1 and D.2, and areas of identified deficiencies are highlighted on Map D.3.

## 6.1 STORM DRAINS AND MAJOR CROSSINGS

Capacity issues associated with closed conduit drainage infrastructure and major roadway crossings were identified through the hydrologic and hydraulic analyses of the 2- to the 100-year storm events.

The Town's Drainage Design Criteria currently requires that stormwater conveyance facilities be designed to the 2-year event, except when approaching major arterial streets, in which case they are required to be designed to the 5-year event. The criteria also list a number of drainages that should be designed for 100-year capacity:

- The Blue River Parkway (Colorado Highway 9 and US Highway 6),
- West Town Swale,
- Rainbow Drive Swale north of Palmers Drive (Bobo Ditch),
- Willow Creek,
- Straight Creek,
- Ryan Gulch and Salt Lick Gulch, including overflow paths,
- Drainage routes identified in the Eagles Nest Master Drainage Plan, and
- All storm water detention sites.

For comparison purposes, storm drain criteria published for municipalities similar to the Town were also reviewed and included below. Further consideration of the storm drain sizing criteria is recommended prior to updating the Town's criteria.

- City of Aspen
  - Local: 5-year capacity for minor drainage systems and 100-year capacity for major drainage systems
  - Collector/Arterial: 10-year capacity for minor drainage systems and 100-year capacity for major drainage systems (including street conveyance)
- City of Durango
  - All Roads: Minimum of 10-year capacity, potentially more if certain criteria are met. Encroachment and cross-street flow requirements vary by street classification. Some street classifications require 100-year conveyance without overtopping.

Results from the hydrologic and hydraulic model were evaluated against the existing Town's Drainage Design Criteria to understand where deficiencies in the Town's storm drain infrastructure exist. Structures that do not currently meet existing criteria are highlighted on Map D.1. The table below also summarizes these findings by storm drain designation.

***Table 12. Summary of Existing Storm Drain Capacity Evaluation***

| Storm Drain Designation | No. of Structures Analyzed | No. of Deficient Structures | Percentage of Structures Meeting Existing Criteria (%) |
|---|---|---|---|
| General Conveyance | 46 | 6 | 87% |
| Adjacent to Arterial Streets | 10 | 1 | 90% |
| The Blue River Parkway | 14 | 4 | 71% |
| West Town Swale | 3 | 3 | 0% |
| Bobo Ditch | 3 | 3 | 0% |
| Willow Creek[1] | 1 | 0 | 100% |
| Straight Creek[1] | 1 | 0 | 100% |
| Ryan and Salt Lick Gulch | 3 | 3 | 0% |

[1.] Willow and Straight Creek crossings of the Blue River Parkway are included in the Blue River Parkway Designation

The event at which each storm drain or culvert no longer has capacity to convey the peak flow was also summarized in the results shown on Map D.1. Field investigations revealed that many of these structures were not clear of sediment or debris, thus highlighting the need for routine maintenance to provide adequate conveyance and reduce flood risk.

In addition to the hydrologic and hydraulic evaluation completed as part of this study, the 2018 FEMA FIS for the Blue River indicated that none of the existing crossings of the Blue River, within the Town boundary, overtop in the 100-year event. It is recommended that any new or replacement crossings provide adequate capacity plus a minimum of 18-inches of freeboard for the 100-year event.

## 6.2 OPEN CHANNEL INFRASTRUCTURE

Open channel stormwater infrastructure (swales, curb and gutter, etc.) were evaluated for capacity and flood risk using a similar methodology to that of the storm drains and major crossings. As discussed in Section 4.2 of this Plan, representative sections were sampled from the DEM for input into the model at each open channel conveyance element, allowing for a detailed analysis of the hydraulic capacity of each drainage element.

Due to the limited resolution of the DEM and sensitivity of results to channel geometry, the capacity of the Town's open channel drainageways were assessed based upon the maximum flow depth computed for each conveyance element. This will allow the Town to easily evaluate concerns in the field. Approximate maximum flow depths at the 100-year event were grouped into four categories: 1) less than six inches, 2) between six and twelve inches, 3) between twelve and twenty-four inches, and 4) greater than twenty-four inches. Locations at which the maximum flow depth exceeded that of the drainageway capacity were identified as areas for potential drainageway improvements. The reported depths should be used, in conjunction with field verification, as a preliminary assessment of channel conveyance capacity during the design phases of the identified improvement projects. There are instances where a culvert crossing might have the adequate conveyance capacity, but the open channel conveyance elements upgradient do not. Therefore, verification of the flow reaching the culvert must be evaluated for site-specific studies. Similarly, the analysis of backwater flooding impacts from deficient storm drain systems was outside of this scope of work; however, it should be considered during the implementation of this Plan.

Over 65 miles of open channel drainageways were analyzed, of which, the following percentages fell into each of the four depth categories for the 100-year event:

- 23% of analyzed open channel length were found to have maximum flow depths less than 6 inches in the 100-year event.
- 32% of analyzed open channel length were found to have maximum flow depths ranging from 6 to 12 inches in the 100-year event.
- 28% of analyzed open channel length were found to have maximum flow depths ranging from 12 to 24 inches in the 100-year event.
- 17% of analyzed open channel length were found to have maximum flow depths exceeding 24 inches in the 100-year event.

In addition to the modeled depths in each open channel element, the following Town Criteria were used to assess roadside swales and drainage infrastructure:

***Table 13. Maximum Permissible Depth and Encroachment for Curbs and Gutters (Town Criteria)***

| Type of Road | Minor Storm (2- to 10-Year) | Major Storm (100-Year) |
|---|---|---|
| | Depth of Flow (inches) | |
| All Types | 30 inches | 36 inches |
| Local | Flow may spread to crown of street | Street right-of-way |
| Collector | Flow spread must leave at least one lane free of water | Flow shall remain in street right of way |
| Arterial | Flow spread must leave at least one lane free of water in each direction | Flow shall remain in street right of way, maximum depth at crown = 6 inches |

The model results indicated minor street flooding adjacent to the West Town Swale, along Warren and Brian Avenues. More pronounced capacity issues exist on the eastern side of Town, tributary to, and including, the Bobo Ditch drainageway. Flooding and roadway overtopping issues were found along Tanglewood Lane, Clayton Road, as well as Palmers and Rainbow Drives.

For the Blue River, the effective FEMA floodplain mapping show a number of structures closely abutting or within the regulatory 100-year floodplain limits as shown on Map B. To minimize flood risk and maintain the natural function and benefits of the riverine system, it is recommended that future development maintain a buffer between the development and the drainageway as required by the Town's Waterbody, Wetland and Riparian Protection Regulations and avoid reductions in flood conveyance capacity (e.g. encroachments into the floodway). Floodplain impacts and mitigation strategies should be considered during the redevelopment of areas already within the regulatory floodplain.

### 6.3 WATER QUALITY

Renowned for its outdoor amenities and recreation opportunities, the Town's economy relies heavily on the health and quality of its open spaces, and the rich ecological value they provide, especially the Blue River. A popular corridor amongst anglers, the Blue River through the Town is largely fed through the outlet of the Dillon Reservoir; however, local stormwater runoff can also have impacts on the water quality of this popular amenity.

The Town has historically worked to mitigate adverse water quality impacts from stormwater runoff through the installation of detention basins. Based upon the latest GIS data provided by the Town and shown on Map C, there are currently twenty-four (24) detention basins throughout Silverthorne, ranging in their size and condition. Many of these detention basins have a triangular weir outlet configuration, held in place by a wooden retaining wall structure. While this configuration likely provides some attenuation of peak stream flows, it likely does not provide the same water quality and flow attenuation benefits of a formalized, full-spectrum outlet structure.

The Town is located in a high alpine environment that sees a considerable amount of snow throughout the winter period as well as a substantially greater number of visitors during those times. With large amounts of snow and visitors, the transportation corridors throughout the Town are heavily maintained with salt and sand. There are many locations where roads run directly adjacent to, and often cross over, the Blue River. The application of salt and sand can be detrimental to a waterway, as well as the appurtenances that carry stormwater to receiving water. These transportation corridors can also contribute other typical chemical pollutants found along the roadways.

While the Blue River itself is not directly a part of this analysis, it should be noted that stormwater within the Town and the Blue River are intrinsically connected. Recently, the Blue River between Silverthorne and Green Mountain, was degraded from its Gold Medal status. Gold Medal status is a designation to a certain river, creek or stream, that specifies a certain number of trout per mile. The removal of this designation to the Blue River and reduction of number of fish per mile is an indicator that the water quality through this area needs to be evaluated and improved.

**6.4 MAINTENANCE**

Field investigations found that many of the stormwater culverts and appurtenances have not been maintained. These structures were partially or fully filled with sediment and debris. Others were found to be bent or partially damaged, compromising their ability to convey their design flow. It was evident that many of the bottoms of the culverts were rusted out or were showing early signs of failing. At many locations, the ends of driveway culverts were crushed, and adequate embankment or end sections had not been installed. Current maintenance practices were thus found to be inadequate for maintaining the design level of service.

Many of the swales throughout Town are in desperate need of maintenance as they are either largely overgrown with vegetation or substantially filled with sediment. The lack of maintenance on these swales causes water to overtop drainage ditches and flow outside typical drainage corridors causing localized ponding. The lack of maintenance on these swales also reduces opportunities for water to infiltrate as its being conveyed towards the Blue River, effecting overall water quality.

It was also found that many of the stormwater detention basins throughout Town are severely neglected. Many of these are overgrown and not even recognizable as stormwater detention basins. Some of the outlet structures associated with these detention basins are very rudimentary consisting of a simple v-notch outlet. These types of outlets provide very little benefit for water quality and do not meet full spectrum detention requirements as developed in the USDCM. Many of these outlets have also failed and are not currently providing detention benefits. It would be beneficial to identify ownership of these ponds and to put owners on notice of the need to provide maintenance.

The maintenance of the roadway corridors can also greatly affect the ability of the Town to convey and treat stormwater. As discussed previously, large amounts of traction sand are placed on the roadway during the winter months. As the snow and ice on the road melts, it carries this traction sand towards the drainage appurtenances.

## 7. CROSSINGS ALONG THE BLUE RIVER PARKWAY

The State highways that bisect the Town include I-70 and the Blue River Parkway, otherwise known as Highway 9 (north of I-70) and Highway 6 (south of I-70). The Blue River Parkway generally runs parallel to the Blue River, thus requiring stormwater crossings to safely convey runoff to its ultimate receiving waters. As the major arterial roadway through Town, analyzing and taking an inventory of these existing crossings is an important element in understanding the flood risk along the highway and identifying potential improvement opportunities. A total of fifteen (15) crossings were analyzed along the Blue River Parkway, of which four (4) did not have capacity to convey the 100-year event. The table below summarizes the findings from each crossing. Map D.2 also provides a summary of this information along with crossing locations and approximate contributing areas.

It is understood that these crossings are within CDOT's right-of-way and therefore owned and maintained by CDOT. Therefore, any work on proposed culverts, including maintenance, would need to be coordinated with CDOT.

***Table 14. Summary of Blue River Parkway Crossings***

| Crossing Location | Shape and Size | Drainage Area [acres] | 100-Year Flow [cfs] | Crossing Capacity [cfs] | Percent Full in 100-Year Event [%] |
|---|---|---|---|---|---|
| At Straight Creek | Box: 6' x 9' | 12,700 | 580 | 1,500 | 39% |
| South of 3rd Street | Pipe: 2.5' Dia. | 60 | 7.9 | 26 | 30% |
| South of 6th Street | Pipe: 3.5' Dia. | 26 | 14 | 130 | 11% |
| At Annie Road | Pipe: 3.0' Dia. | 34 | 19 | 65 | 29% |
| At 11th Street | Pipe: 2.0' Dia. | 17 | 10 | 13 | 77% |
| South of 13th Street | 2x Pipe: 3.5' Dia. | 1,150 | 120 | 110 | 100% |
| South of Smith Ranch Road | Pipe: 2.0' Dia. | 348 | 42 | 11 | 100% |
| At Willow Creek | 2x Box: 5' x 5' | 8,540 | 360 | 1,100 | 33% |
| North of Willowbrook Road | Pipe: 2.0' Dia. | 38 | 10 | 26 | 38% |
| South of Golden Eagle Road | Pipe: 2.0' Dia. | 254 | 27 | 23 | 100% |
| North of Golden Eagle Road | Pipe: 2.0' Dia. | 7.4 | 1.4 | 17 | 8% |
| Near Blue River WWTP | Pipe: 2.0' Dia. | 283 | 34 | 48 | 71% |
| Across from Spur Circle | Pipe: 2.0' Dia. | 33 | 1.4 | 30 | 5% |
| Across from Saddle Circle | Pipe: 3.0' Dia. | 605 | 63 | 98 | 64% |
| South of Rancher's Road | Pipe: 2.0' Dia. | 216 | 20 | 14 | 100% |

Included in the table above is the percentage of full flow capacity at which the culvert flows during a 100-year event. This information was included to identify potential opportunities to utilize existing crossings with adequate capacity to alleviate the hydraulic loading on existing, undersized crossings, without the need of installing a new crossing beneath the highway. The reported capacity of each crossing is based on a normal depth analysis and does not include backwater and tailwater effects, or sedimentation. Because these considerations were not accounted for, the actual conveyance capacity may be less than those included in Table 14 and should verified during the implementation of this Plan.

The crossing south of 13th Street conveys stormwater from the West Town Swale to the Blue River and is slightly undersized for the 100-year event. The calculated capacity of the crossing is 110 cfs, where the 100-year event was found to be 120 cfs. Historically, the West Town Swale conveyed stormwater runoff from the southern end of Town, near Warren Avenue, north through the Town Core, and ultimately to 13th Street. Projects identified in Section 8 of this Plan outline potential opportunities to reroute flood flows such that they flow more directly to the Blue River and utilize the additional capacity of existing culverts. Specifically, projects PR050, PR060, and PR070 include rerouting flood flows from the West Town Swale to existing culverts south of 3rd Street, south of 6th Street, and at Annie Road, respectively.

The existing storm drain system downstream of the Smith Ranch development is also undersized. With a computed capacity of 11 cfs and a 100-year flow rate of 42 cfs, opportunities to alleviate flood risk at this location were also identified in the following list of site-specific projects. It was determined that increasing the capacity of the crossing through replacement of the existing pipe is not a feasible alternative. Therefore, two alternatives for routing flows away from this crossing are proposed as projects PR080 and PR090 of this Plan. Plan project PR080 would involve rerouting flows from Smith Ranch to the Highway 9 culvert south of 13th Street, once its capacity has been alleviated through the construction of any combination of PR050, PR060, and/or PR070. Alternatively, Plan project PR090 would include a full spectrum detention basin upstream of the undersized crossing to attenuate peak flow rates from entering the downstream storm infrastructure. The basin would be configured such that an overflow would route runoff to the north, utilizing the additional capacity of the Willow Creek culvert.

The existing two-foot diameter culvert beneath Highway 9, just south of Golden Eagle Road, was found to have a capacity of 23 cfs and a computed 100-year flow of 27 cfs. Plan project PR100 recommends replacing the crossing with a culvert with greater capacity to meet the 100-year requirement; however, there may also be an opportunity to use adjacent topography to formalize a full spectrum detention basin to attenuate the peak flow rate and provide water quality benefits upstream of the crossing.

Similar to the crossing south of Golden Eagle Road, the culvert south of Rancher's Road does not meet the 100-year storm criteria. The culvert capacity at this location was found to be 14 cfs where the computed 100-year flow is 20 cfs. Once again, the replacement of this crossing with a larger culvert or upstream detention would likely alleviate the flood risk in this location. Plan projects PR130 and PR131 would work to address this deficiency and are discussed in more detail in Section 8 below.

## 8. PROJECT IDENTIFICATION AND PRIORITIZATION

To address the areas of deficiencies recognized through the technical analyses, a list of identified projects was developed. These projects were evaluated against one another to gain an understanding of their priority. Map E illustrates the location of each project and potential site improvements to address the identified deficiencies. It is important to note that these identified deficiencies are recognized for current conditions. Improvements or changes in the watershed may address deficient areas, or conversely cause another issue to arise.

### 8.1 COST ESTIMATES

Cost estimates have been developed for projects which were assigned a high priority via the prioritization process outlined in Section 8.3. The MHFD's UD-MP Cost spreadsheet (version 2.2) was utilized to estimate the capital improvement costs, including engineering and administrative costs, for the identified high priority projects. A range was assigned to each estimate commensurate with that of the master-planning level of understanding of each project. Specifically, the Association for the Advancement of Cost Engineering (AACE) Class 4 estimate level was applied for study purposes. The high range of this estimate level (+50%) was assigned prior to the application of a contingency within the UD-MP Cost spreadsheet. This estimate range accounts for unknown factors, such as underground utilities and Right of Way acquisition that are difficult to predict at the master-planning level. Cost estimate ranges have been included in Section 8.2, Map E, and Appendix A for high priority projects. Supporting documentation for these estimates has been included in Appendix C.

### 8.2 IDENTIFIED PROJECTS

In addition to the other general recommendations included in Section 9, this Plan has identified seventeen (17) site-specific projects to improve the stormwater infrastructure through the Town. These projects range from infrastructure capacity improvements to full-spectrum detention basins. A brief summary description of each identified site-specific project is provided below. The location and approximate extents of each is also included on the Recommended Plan map, Map E. Cost estimates and priority have been assigned to the identified projects via the methodology presented in Sections 8.1 and 8.3 of the Plan, respectively.

***PR010:*** *Increase the capacity of the storm drain south of Highway 6*

The existing storm drain collecting stormwater from the incoming system behind Office Max and routed along Highway 6 to Straight Creek is currently damaged. The existing 36-inch diameter pipe collects runoff from upstream residential areas as well as the Dillon Ridge Marketplace, totaling a drainage area of roughly 150 acres. Provided it were clean and undamaged, the current storm drain would have a 50-year capacity of 81 cfs. The 100-year peak runoff at the Office Max inlet is approximately 87 cfs. Improvements of the storm drain would include the replacement from the Office Max inlet to the outfall into Straight Creek, including inlet improvements. Increasing the pipe size to provide 100-year capacity would also minimize the risk of local flooding and reduce maintenance needs. The estimated cost of improvements, including administration, design, and construction is estimated to be within the range of \$390,000 to \$580,000. This project was found to have a high priority.

***PR020:*** *Increase the culvert capacity beneath South Adams Avenue*
Salt Lick Gulch currently flows on the south side of I70 and is routed beneath the interstate via a culvert west of South Adams Avenue. Upstream channel conditions, however, split flow before it enters the culvert, routing water further downstream, ultimately crossing South Adams Avenue. Moreover, the culvert beneath I70 only has a 10-year capacity, with the overflow being routed to the same crossing location of South Adams Avenue. The current 18-inch culvert beneath South Adams Avenue is shown to have a capacity of 11 cfs, less than that of the 2-year event. With a 2- and 100-year peak flow event of 14 cfs and 96 cfs, respectively, increasing the culvert size at this location and providing up- and downstream conveyance improvements would reduce local flood risk and maintenance needs. This project was found to have a moderate priority.

***PR030:*** *Water Quality Facility along the Blue River beneath I70*
Runoff generated along the I70 corridor is currently untreated prior to its outfall into the Blue River. Construction of water quality facilities to treat I70 runoff from lanes on either side of the River would improve the water quality in the Blue River through the Town. This project was found to have a moderate priority.

***PR040:*** *Increase the capacity of the culverts along Salt Lick Gulch beneath South Adams Avenue*
Downstream of the I70 culvert crossing, the two ellipse culverts along Salt Lick Gulch, beneath South Adams Avenue, do not provide adequate capacity. With a capacity of only 82 cfs, the crossing sees a 2- and 100-year peak flow rate of 180 cfs and 340 cfs, respectively. Replacing these culverts would reduce flood risk at the intersection of South Adams Avenue and Wildernest Road. As the existing culverts appear to be recently installed with a steep drop inlet configuration, the original design calculations should be reviewed in light of the hydrology published within this Plan. A detailed evaluation of the inlet hydraulics was outside of the scope of this Plan, however, there are potential hydraulic efficiencies gained with the drop inlet configuration, the adequacy of which should be evaluated with the latest hydrology prior to the implementation of this project. Proposed improvements would include the replacement of the existing culverts. This project was found to have a low priority.

***PR050:*** *Formalize an alternate flow path to route drainage to the northeast along 3rd Street*
Rerouting the drainage pattern of the roadside swale along the northeast side of Brian Avenue to flow to the northeast along 3rd Street, and ultimately into the Highway 9 culvert south of the 3rd Street intersection, would alleviate some of the flooding issues realized further downstream, along the West Town Swale. Proposed improvements would include the installation of a driveway culvert and construction of a roadside swale along 3rd Street into the Highway 9 culvert. Currently, the invert of the culvert at Highway 9 is too high for these improvements to function properly, however this

project should be considered if the Highway 9 culvert is replaced and lowered in the future. This project was found to have a low priority.

***PR060:*** *Create an alternate flow path to route drainage to the northeast along 5th Street*
Rerouting the drainage pattern of the roadside swale along Brian Avenue to flow to the northeast along 5th Street, and ultimately into the Highway 9 culvert south of the 6th Street intersection, would alleviate some of the flooding issues found along the West Town Swale. These improvements could divert a 2- and 100-year peak flow rate of 14 cfs and 31 cfs, respectively, from the West Town Swale to the Highway 9 culvert. The downstream culvert of Highway 9, just south of 6th Street, has more than adequate capacity to handle the increased flow at this location. The crossing is currently only 11% full in a 100-year event with a capacity of approximately 130 cfs. Proposed improvements would include the installation of three culverts and the construction of roadside swales. The estimated cost of improvements, including administration, design, and construction is estimated to be within the range of $170,000 to $260,000. This project was found to have a high priority.

***PR061:*** *Full Spectrum Detention within Highway 9 Right of Way north of 5th Street to accompany rerouted flow path of Project PR060*
Developing a full spectrum detention basin to detain the rerouted flows from PR060 would reduce the additional peak flow loading of the Highway 9 culvert, south of the 6th Street intersection, and improve water quality prior to its outfall into the Blue River. Proposed improvements would include the grading and revegetation of the detention basin and the installation of a full-spectrum outlet structure within the Highway 9 Right of Way, between 5th and 6th Street. The estimated cost of improvements, including administration, design, and construction is estimated to be within the range of $200,000 to $300,000. This project was found to have a high priority.

***PR070:*** *Create a split flow path to route flood flows into existing storm system along Annie Road*
The installation of a split flow structure west of Adams Avenue, along the West Town Swale, to reroute a portion of the flood flows into the existing storm drain system along Annie Road, would alleviate some of the flooding issues found along the West Town Swale. Provided upstream improvements outlined in this Plan are not installed, diverting flow away from the West Town Swale, the anticipated peak flow rates into the flow split would be approximately 53 cfs in the 2-year and 110 cfs in the 100-year event. The capacity of the existing storm infrastructure should be evaluated to determine the feasibility and impact of the split flow prior to implementation. Replacement of this infrastructure may be necessary to accommodate the increased flow rates from the split. Proposed improvements would include the construction of a split flow structure (such as an overflow weir) and the subsequent downstream infrastructure to tie-into the existing system. The downstream culvert across Highway 9, south of Annie Road, currently has capacity for an additional 46 cfs, with it running at about 29% full in the 100-year event, provided a full flow capacity of roughly 65 cfs. The estimated cost of improvements, including administration, design, and construction is estimated to be within the range of $300,000 to $450,000. This project was found to have a high priority.

***PR071:*** *Full Spectrum Detention within Highway 9 Right of Way north of 5th Street to accompany rerouted flow path of Project PR060*

Developing a full spectrum detention basin to detain the split flows from PR070 would the improve the water quality of this drainage prior to its outfall into the Blue River. Proposed improvements would include the grading and revegetation of the detention basin and the installation of a full-spectrum outlet structure within Annie Road lot, just east of the intersection of Highway 9 and Annie Road. This project was found to have a moderate priority.

***PR080:*** *Create an alternate flow path to route Smith Ranch drainage to culvert beneath Highway 9 at 13th Street*

Routing the Smith Ranch drainage to the south, ultimately into the Highway 9 culvert south of 13th Street, would reduce local flood risk along and upstream of Highway 9. This culvert currently has a 50-year capacity of 110 cfs, therefore this project must be completed after the PR070 improvements alleviate the peak flow rates seen at Highway 9 crossing. Peak flow rates to be rerouted from the Smith Ranch drainage are approximately 20 cfs and 42 cfs in the 2- and 100-year events, respectively. Proposed improvements would include increased culvert and swale capacity from north of Ruby Ranch Road to South of 13th Street. Alternatively, this drainage path could be piped to the inlet of the Highway 9 culvert, just south of 13th Street. The estimated cost of improvements, including administration, design, and construction is estimated to be within the range of $440,000 to $660,000. This project was found to have a high priority.

***PR090:*** *Full Spectrum Detention downstream of Smith Ranch and upstream of Highway 9 crossing*

In lieu of PR080, developing a full spectrum detention basin to detain flows from Smith Ranch would reduce the additional peak flow tributary to the Highway 9 culvert and improve water quality. To reduce hydraulic loading on the existing, undersized storm drain beneath Highway 9, this basin should be configured such that overflow is routed to the north, into the Willow Creek drainageway. Having a capacity of roughly 1,100 cfs, the Willow Creek culvert beneath Highway 9 currently runs at roughly 33% of full capacity in the 100-year event and could therefore handle the additional flow from the detention facility. Peak flow rates to be routed through the facility, from the Smith Ranch drainage, are approximately 20 cfs and 42 cfs in the 2- and 100-year events, respectively. Proposed improvements would include the grading and revegetation of the detention basin and the installation of a full-spectrum outlet structure, including a culvert beneath Smith Ranch Road, into Willow Creek. The estimated cost of improvements, including administration, design, and construction is estimated to be within the range of $400,000 to $610,000. This project was found to have a high priority.

***PR100:*** *Increase culvert capacity beneath Highway 9 south of Golden Eagle Road*

Increasing the capacity of the existing culvert crossing of Highway 9, south of Golden Eagle Road, would reduce local flood risk along and upstream of Highway 9. The highway 9 culvert currently has a 25-year capacity of 23 cfs, with an incoming 100-year peak flow rate of 27 cfs. A more detailed hydraulic analyses should be completed to determine the extent of the flood risk in this area.

Currently, there is a depression near the inlet of the culvert which may provide inadvertent flow attenuation. Formalizing a full spectrum detention facility at this location may also help to address potential flood risk and alleviate the need for increasing the culvert capacity. Proposed improvements would include the replacement of the existing culvert. This project was found to have a low priority.

***PR101:*** *Full Spectrum Detention at southeast corner of Highway 9 and Bald Eagle Road*
Developing a full spectrum detention basin on the southeast corner of Highway 9 and Bald Eagle Road would reduce the hydraulic loading of the undersized downstream storm drain and provide water quality benefits prior to discharging into the downstream pond. Peak flow rates into the basin are estimated to be 12 cfs and 28 cfs in the 2- and 100-year event, respectively. Proposed improvements would include the grading revegetation of the detention basin and the installation of a full-spectrum outlet structure. This project was found to have a moderate priority.

***PR110:*** *Increase culvert capacity beneath Golden Eagle Road*
Increasing the capacity of the existing culvert crossing of Golden Eagle Road, would reduce the risk of floodwaters overtopping the roadway. The current culvert has an approximate capacity of 11 cfs, with 12 cfs and 28 cfs tributary to the crossing in the 2- and 100-year event, respectively. These peak flow rates assume the pond south of Golden Eagle Road is full and does not provide flow attenuation. Proposed improvements would include the replacement of the existing culvert. This project was found to have a low priority.

***PR120:*** *Increase culvert capacity beneath Arnica Lane*
Increasing the capacity of the existing culvert crossing of Arnica Lane, would reduce the risk of floodwaters overtopping the roadway. The current culvert has an approximate capacity of 15 cfs, with 13 cfs and 27 cfs tributary to the crossing in the 2- and 100-year event, respectively. Proposed improvements would include the replacement of the existing culvert. This project was found to have a low priority.

***PR130:*** *Increase culvert capacity beneath Highway 9 south of Rancher's Road*
Increasing the capacity of the existing culvert crossing of Highway 9, south of Rancher's Road, would prevent excess flood flows along Highway 9. The current culvert has an approximate capacity of 14 cfs, with 12cfs and 20 cfs tributary to the crossing in the 2- and 100-year event, respectively. Proposed improvements would include the replacement of the existing culvert. This project was found to have a low priority.

***PR131:*** *Full Spectrum Detention at southwest corner of Highway 9 and Rancher's Road to be coordinated with Project PR130*
Developing a full spectrum detention basin to detain flows upstream of the existing culvert crossing of Highway 9, south of Rancher's Road, would reduce the peak flow loading of the Highway 9 culvert and improve water quality. This project should be coordinated with PR130 as the installation of a detention basin may attenuate peak flows such that the existing culvert has capacity for the

detained 100-year flow, thus alleviating the need for replacement. Peak flow rates into the basin are estimated to be 14 cfs and 20 cfs in the 2- and 100-year event, respectively. Proposed improvements would include the grading and revegetation of the detention basin and the installation of a full-spectrum outlet structure. This project was found to have a moderate priority.

## 8.3 PROJECT PRIORITIZATION

The evaluation of each of the site-specific projects against the below criteria produced six (6) high priority projects, five (5) moderate priority projects, and six (6) low priority projects. A summary of this evaluation is included in Table 15 below. Appendix A also provides this information without abbreviation.

EVALUATION CRITERIA

The site-specific projects outlined in Section 8.1 were qualitatively evaluated against seven (7) evaluation criteria to develop a priority for each. These criteria compare the benefits of each project such that high-benefit, feasible projects are identified as high-priority. Moderate and low priority projects still provide benefit to the stormwater infrastructure throughout Silverthorne and should not be discounted due to their ranking. Criteria on which site-specific projects were evaluated includes:

A. Impacts to public health and safety,
B. Impacts to critical infrastructure,
C. Frequency of problem,
D. Potential damage,
E. Water quality / environmental impacts,
F. Multi-objective, and
G. Feasibility.

***Table 15. Project Prioritization***

| Project | Evaluation Criteria (Section 7.2) | | | | | | | Priority |
|---|---|---|---|---|---|---|---|---|
| | A | B | C | D | E | F | G | |
| *PR010*: Increase the capacity of the storm drain south of Highway 6 | M | M | H | H | L | L | M | High |
| *PR020*: Increase culvert capacity beneath South Adams Avenue | M | L | H | M | L | L | H | Moderate |
| *PR030*: Water Quality Facility along the Blue River beneath I70 | M | L | H | L | H | L | M | Moderate |
| *PR040*: Increase the capacity of the culverts along Salt Lick Gulch beneath S. Adams Avenue | L | L | H | M | L | L | L | Low |
| *PR050*: Formalize an alternate flow path to route drainage to the northeast along 3rd Street | L | M | M | M | L | L | L | Low |
| *PR060*: Create an alternate flow path to route drainage to the northeast along 5th Street | M | M | H | M | L | M | M | High |
| *PR061*: Full Spectrum Detention within Hwy 9 R.O.W. north of 5th Street accompanying PR060 | L | L | H | L | H | M | H | High |
| *PR070*: Create a split flow path to route flood flows into existing storm system along Annie Rd. | M | M | H | H | L | L | M | High |
| *PR071*: Full Spectrum Detention east of Annie Rd. and Hwy 9 to accompany rerouted PR070 | L | L | H | L | H | M | M | Moderate |
| *PR080*: Create an alternate flow path to route Smith Ranch to culvert at Hwy 9 and 13th Street | M | M | H | M | L | M | M | High |
| *PR090*: Full Spectrum Detention downstream of Smith Ranch and upstream of Hwy 9 crossing | M | M | H | M | H | H | M | High |
| *PR100*: Increase culvert capacity beneath Highway 9 south of Golden Eagle Road | M | M | L | M | L | L | L | Low |
| *PR101*: Full Spectrum Detention at southeast corner of Hwy 9 and Bald Eagle Road | L | L | H | M | M | M | M | Moderate |
| *PR110*: Increase culvert capacity beneath Golden Eagle Road | L | L | M | L | L | L | L | Low |
| *PR120*: Increase culvert capacity beneath Arnica Lane | L | L | M | L | L | L | M | Low |
| *PR130*: Increase culvert capacity beneath Hwy 9 south of Rancher's Road | M | L | M | M | L | L | L | Low |
| *PR131*: Full Spectrum Detention at SW corner of Hwy 9 and Rancher's Rd to accompany PR130 | M | L | M | L | H | M | M | Moderate |

Note: Table uses the lettering convention for each evaluation criteria outlined in Section 8.3 and abbreviates the 'High,' 'Moderate,' and 'Low' rankings with the letters 'H,' 'M,' and 'L,' respectively.

# 9. RECOMMENDED PLAN

The recommended Plan outlined below addresses identified deficiencies in the Town's stormwater infrastructure through; prioritized site-specific improvement projects, stormwater criteria and maintenance recommendations, including the development of a stormwater utility fee, and water quality recommendations. Potential areas of future study have also been identified based upon local risks and conditions.

## 9.1 PRIORITIZED IMPROVEMENT PROJECTS

Site-specific stormwater improvement projects detailed in Section 8 of this Plan, and prioritized in Table 15, should be considered for implementation by the Town. The site-specific project list was developed to address stormwater deficiencies found through the hydrologic and hydraulic analyses, as well as known areas of concern. The location and approximate extent of each project is illustrated on Map E. In addition to the prioritized projects outlined in this Plan, the Town should consider improvements to more efficiently route stormwater runoff generated on the east side of the Bobo Ditch to the Blue River when the opportunity presents itself.

## 9.2 UPDATES TO EXISTING DRAINAGE DESIGN CRITERIA

This Plan recommends that the Town's Drainage Design Criteria be updated using the USDCM as a guiding document. Further developing these criteria will improve the minimum standards and technical guidance for future development within the Town. It is recommended that these criteria include updated hydrologic evaluation techniques, drainage system design requirements, best practices for site design, criteria for the selection of structural stormwater controls, as well as construction and maintenance information. An update to the Drainage Design Criteria will help to maintain the serviceability of the Town's stormwater infrastructure while including requirements for stormwater quality.

Concurrent with this Project, WWE has reviewed and provided guidance to the Town on updates to the manual. The hydrologic and hydraulic modeling's guidelines from this Plan, as well as USDCM, are recommended to be incorporated. Establishment of water quality requirements is highly recommended, as well as maintenance protocols.

## 9.3 STORMWATER MAINTENANCE PLAN

A stormwater maintenance plan can help the Town monitor and preserve the integrity and function of existing stormwater appurtenances and help to improve stormwater quality. Components of a successful maintenance plan would include: a GIS-based monitoring database including all stormwater infrastructure and maintenance activities, identification of maintenance responsibility, funding, and frequency, as well as an implementation strategy for capital improvement projects outlined in this Plan or identified through routine maintenance activities. This maintenance plan can be built upon the existing stormwater GIS database to have an all-inclusive holistic understanding of the Town's stormwater infrastructure.

Additional maintenance activities could be undertaken to sweep traction sand up prior to entering the waterways. These sweeping operations are also useful for cleaning up trash and general housekeeping practices throughout the Town. Maintenance could include cleanup of leaks and spills from vehicles that take place along the transportation corridors.

### 9.4 WATER QUALITY RECOMMENDATIONS

Retrofitting the existing weir outlet structures to provide full-spectrum detention, where feasible, and requiring full spectrum detention facilities for all future development will help protect the Blue River and its tributaries from adverse water quality impacts from stormwater runoff. It is recommended that these facilities are designed to the standards outlined in the USDCM. Not only should new development be required to provide full spectrum detention facilities but should utilize Low-Impact-Development strategies to the maximum extent possible, disconnecting large impervious areas.

Where feasible, snow storage locations should be upstream of water quality facilities to capture traction sand and deicer before it is conveyed into the Blue River via snowmelt. Maintaining critical riparian vegetation and habitat along all major drainageways, including the Blue River, will also help protect water quality.

Currently, non-profit groups as well as municipalities, including the Town, are involved in a Stream Management Plan to investigate the water quality of the Blue River and develop a plan to address the water quality concerns. It is recommended that stormwater quality improvements are coordinated with this ongoing effort. The substantial amount of old and new development along the Blue River as well as the minimal stormwater quality treatment throughout the Town could be affecting the fishery of the Blue River as well as the overall watershed health.

### 9.5 STORMWATER UTILITY FEE

In order to maintain stormwater infrastructure, the Town needs a way to fund the maintenance identified in this Plan. The Town also needs a way to fund these site-specific stormwater improvement projects detailed in Section 7.1. The Town currently does not have a stormwater utility fee to help fund either the maintenance or the site-specific projects. It is our understanding that all work associated with stormwater infrastructure is currently paid for through the Town's general fund. It is WWE's recommendation that maintenance activities be paid for via a utility fee, and site-specific project be paid through capital improvement funding, since one site-specific project could utilize the entire maintenance budget. As part of this overarching project, the Town has also enlisted WWE to evaluate options for a stormwater utility fee. A memorandum outlining potential fee structures has been developed in coordination with the Town.

Included in the memorandum is an evaluation of similar communities' utility fee structure, as well as typical stormwater maintenance and capital improvement infrastructure costs. The memorandum outlines the many different ways stormwater utility fees can be generated such as: fee per parcel acre, flat fee per classification, fixed rate for all properties, percent runoff, amount impervious, etc. The memorandum also outlines typical stormwater infrastructure routine maintenance costs that could be expected for the Town.

## 9.6 POTENTIAL AREAS OF FUTURE STUDY

This Plan recognizes the benefit of future studies to complement the findings outlined in this report. Areas of potential future study include: cost estimation of stormwater improvement projects within the Town, a detailed two-dimensional hydraulic model to assess irregular flow paths and provide an approximation of inundation extents at various return intervals, a detailed assessment of mud and debris flow risk from the high alpine headwaters tributary to the Blue River (potentially including effects of wildfire), and an evaluation of the stormwater quality entering the Blue River.

## 9.7 DIGITAL FILES

Digital Google Earth and GIS shapefiles were produced and provided to the Town as part of this Plan. These shapefiles outline the spatial extents, attributes and results of the modeled subbasins, storm drains and open channel conveyances.

## 9.8 DISCLAIMER

This information included in this Plan is intended for planning purposes only. Discrete hydrologic and hydraulic evaluations should be completed for all design projects throughout the Town of Silverthorne. Existing infrastructure should be surveyed in the field and the assumptions made in this Plan should be verified during the implementation of future projects. All designs should be conducted in coordination and communication with the Town of Silverthorne.

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