***Dam Breach Study and Flood Inundation Mapping***
***For Silver Creek Dam***
***Marion County, Oregon***
**NID# OR00622**

Prepared for:

**City of Silverton**
**306 S Water St**
**Silverton, OR 97381**

Prepared by:

**WEST Consultants, Inc.**
**2601 25th Street SE, Suite 450**
**Salem, OR 97302-1286**

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# EXECUTIVE SUMMARY

This report summarizes potential flood inundation from a hypothetical dam breach of Silver Creek Dam located in the City of Silverton, Marion County, Oregon. The dam was constructed in 1974. The dam is owned and operated by the City of Silverton for municipal water supply and recreation uses. The results from this analysis will be used by the city for planning purposes and to update the flood inundation mapping in their Emergency Action Plan (EAP) for the dam. A dam breach flood from Silver Creek Dam is expected to flow north down Silver Creek through a confined valley with intermediate reaches of narrow, flat floodplains until about 1 mile downstream of the dam. At about 1 mile downstream of the dam, floodwaters would flow into the urbanized floodplain of City of Silverton on the east side of the river where it would then flow north through the city. At about 3.1 miles downstream of the dam near Pine Street, portions of the floodwaters would leave the Silver Creek system and continue to flow north to Abiqua Creek.

A Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) model of Silver Creek upstream of Silver Creek Dam was developed to estimate the inflow hydrograph to dam for the 0.2 percent Annual Chance Exceedance (ACE) and General Storm Probable Maximum Flood (PMF) events.

Silver Creek Dam is an earthen (with a clay core) embankment founded on bedrock. Dam breach parameters for the dam were estimated using the latest guidance available for an earthen embankment dam. A combined one-dimensional (1D)/two-dimensional (2D) unsteady Hydrologic Engineering Center River Analysis System (HEC-RAS) version 5.0.7 (HEC, 2010) hydraulic model of Silver Creek was developed to define the expected inundation boundaries of flooding from a potential dam breach. The entire reach upstream of the dam and the main channel downstream of the dam were defined in the model as a 1D reach, and the floodplain areas in the reach downstream of the dam were defined as 2D areas. A recent bathymetric survey of the reservoir completed by Stuntzner Engineering & Forestry LLC indicates that the reservoir has a storage volume of only 790 ac-ft, which is less than the design volume of about 1,300 ac-ft. For this study, cross sections upstream of the reservoir were modified to re-establish the volume reservoir at the spillway crest.

Three dam breach scenarios were considered to determine the maximum water surface elevations in the study reach and generate a dam breach inundation map: (1) a Sunny Day "no warning" breach, (2) a breach during a 0.2% ACE flood, and (3) a breach caused by a General Storm PMF. The PMF breach scenario produced the largest peak discharge, downstream water surface elevations, and inundation extents. In all scenarios, there would be pronounced flooding in the City of Silverton with the fast arrival times. A sensitivity analysis of the model parameters was considered for this study given their high level of uncertainty.

Dam breach inundation boundaries for the study reach were generated using HEC-RAS for the three breach scenarios. A dam breach inundation map for each scenario is provided in Appendix B. Appendix B also include maps showing damage zones based on the criteria documented in *Damage to Residential Buildings due to Flooding of New Orleans after Hurricane Katrina* (Pistrika and Jomkman, 2010). All digital files used to create these maps are provided in the DVD included in Appendix C.

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# 1 INTRODUCTION

## 1.1 Scope of Study

The objective of this study is to conduct sufficient hydrologic and hydraulic analysis to determine the potential dam breach floodplain below Silver Creek Dam located in the Cascade foothills about 2.5 miles southeast of the City of Silverton in Marion County, Oregon. The results from this analysis will be used by the city for planning purposes and to update the flood inundation mapping in their Emergency Action Plan (EAP) for the dam. The study reach extends from about 1.5 miles upstream of the dam to the Brush Creek Dr. SE Highway crossing, which is located about 5.1 miles downstream of the dam. The previous dam breach analysis (Philip Williams & Associates, PWA, 2000) indicated that a portion of the dam breach flood hydrograph would leave the Silver Creek system and flow north towards Abiqua Creek. The split flow occurs at about 3.1 miles downstream of the dam near Pine Street. The current study includes this split flow reach to about 1.5 miles downstream of the split. The scope of study includes: a site visit, hydrologic modeling, hydraulic modeling, development of inundation maps, and study documentation. Unless otherwise specified, all elevations listed within this report are referenced to the North American Vertical Datum of 1988 (NAVD88).

# 2 WATERSHED DESCRIPTION

## 2.1 Dam and Reservoir

Silver Creek Dam is located on Silver Creek within the southeast of the Silverton City (Figure 2-1). The dam was constructed in 1974. The dam is owned and operated by the City of Silverton for municipal water supply and recreation uses. Construction drawings of the dam are provided in Appendix A.

The dam consists of a 65-foot-high, 490-foot-long earthen (with a clay core) embankment founded on bedrock, 42-inch-diameter low level reinforced concrete outlet, and a 120-foot-long concrete-lined uncontrolled spillway structure. The embankment has a minimum elevation of 416.2 ft. The minimum elevation of the spillway crest is 415.87 ft. At the top of the uncontrolled section, the design reservoir storage volume is about 1,300 acre-ft. A recent bathymetric survey of the reservoir completed by Stuntzner Engineering & Forestry LLC indicates that the reservoir has a storage volume of only 790 acre-ft. This elevation-volume relation will be referred to as the “2019 elevation-volume” relationship in this report.

The drainage basin for Silver Creek Dam is shown in Figure 2-2. It has a total area of about 44 square miles and varies in elevation from about 375 ft at the dam to about 1,028 ft at Rocky Top peak near the southern portion of the watershed. The basin is bounded on

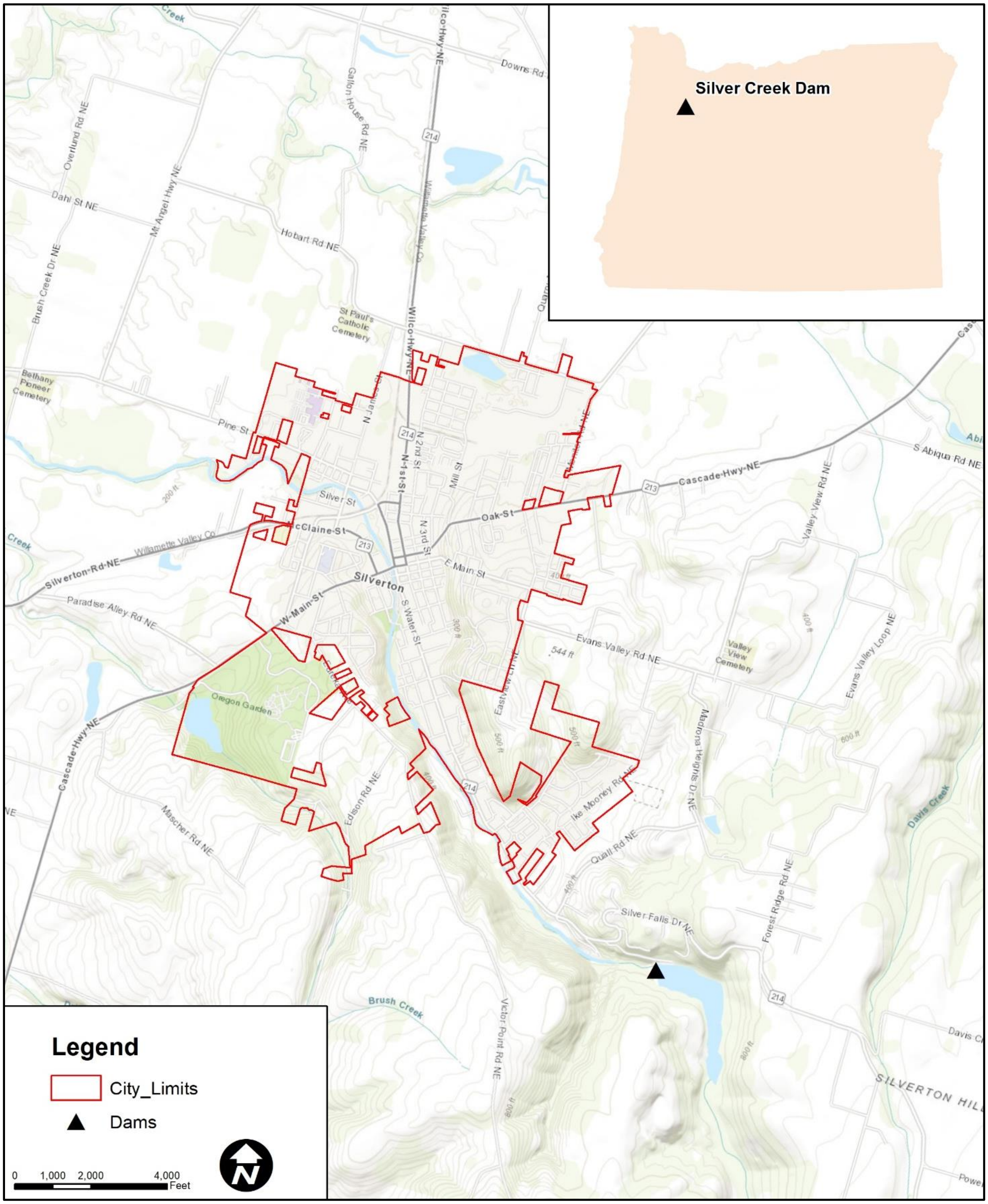


**Figure 2-1.** Location map

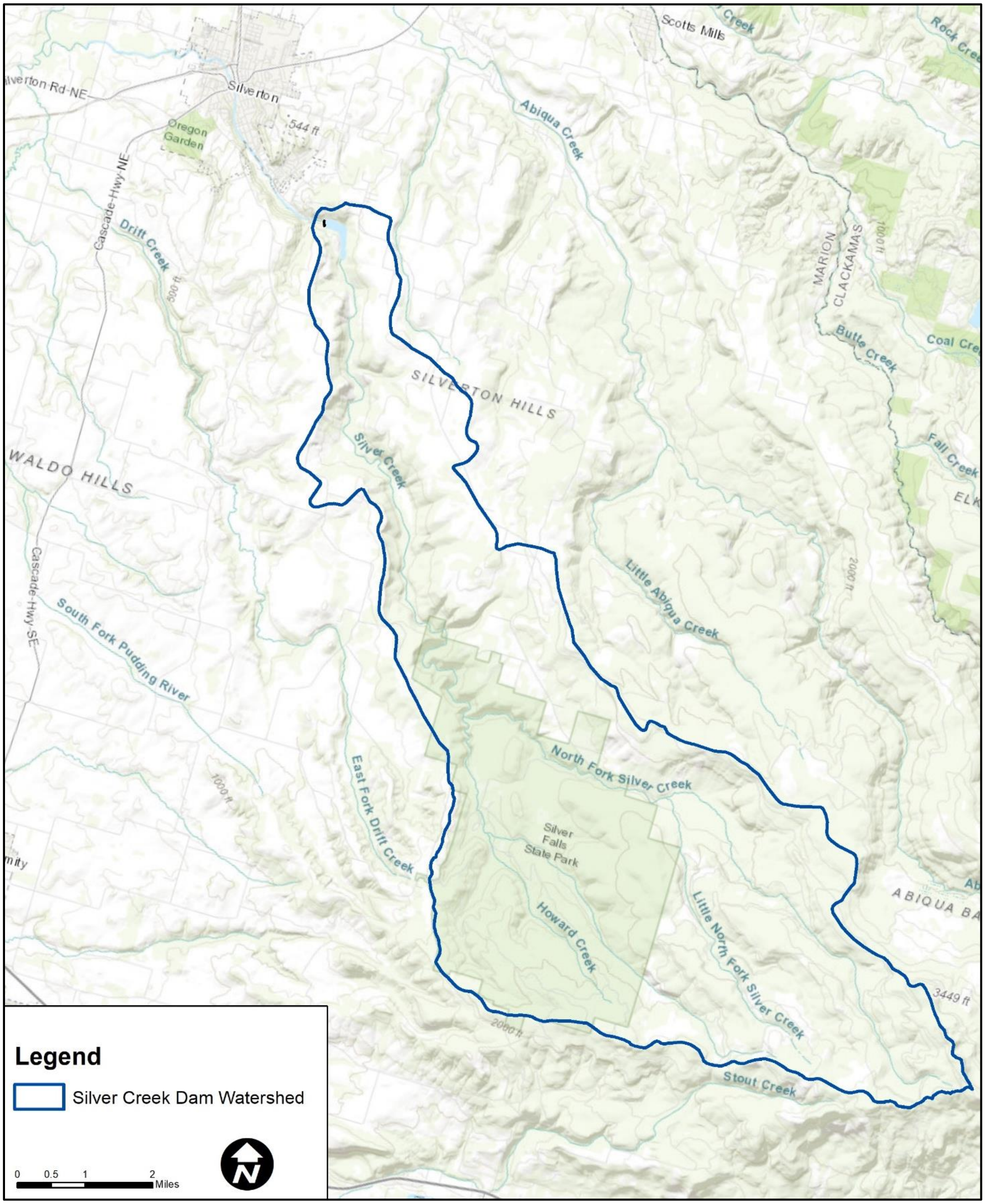


**Figure 2-2.** Silver Creek Dam drainage basin

the east by Wildcat Ridge, on the south by Big Green Mountain; and on the west by the divide between Silver Creek and Drift Creek. Land use in the basin is characterized as primarily Douglas-fir forest with some farm land near the northern portion of the watershed.

## 2.2 Stream Channel Characteristics

A breach of Silver Creek Dam would send floodwaters down Silver Creek through a confined valley with intermediate reaches of narrow, flat floodplains until about 1 mile downstream of the dam. At about 1 mile downstream of the dam, floodwaters would flow into the urbanized floodplain of City of Silverton on the east side of the creek where it would then flow north through the city. At about 3.1 miles downstream of the dam near Pine Street, portions of the floodwaters would leave the Silver Creek system and continue to flow north to Abiqua Creek. For the study reach downstream of the dam, Silver Creek is relatively steep with three distinct reaches that range in slope between 0.4 and 0.9 percent. The creek downstream of the dam is comprised of gravel, cobble, and bolder material along the bed and trees along the banks. Figure 2-3 shows typical a reach of Silver Creek downstream of the dam.

**Figure 2-3.** Typical cross section of Silver Creek below Silver Creek Dam

# 3 DATA COLLECTION

## 3.1 Geometry Data

A 3-ft seamless Digital Elevation Model (DEM) was developed using LiDAR data obtained from the Oregon Department of Geology and Mineral Industry (DOGAMI) and bathymetric data recently obtained of the reservoir by Stuntzner Engineering & Forestry LLC. The DEM is referenced to the horizontal North American Datum 1983 (NAD83) USA Contiguous Albers Equal Area Conic USGS version (Feet) and the vertical datum of NAVD88. The DEM of the study area terrain was defined in a raster format. The model geometry necessary for development of a hydraulic model was extracted using ArcGIS. Cross sections of the reach upstream of the dam were used to model the breach-induced drawdown and to route the flow properly through the reservoir.

## 3.2 Survey Data

The geometry data developed as discussed in the previous section is considered sufficient to define the potential extent of inundation of a dam breach flood event. Therefore, no additional ground survey data was collected. However, general information related to the various bridge structures was obtained during the field investigation. The information included the number of piers, pier type and dimensions, distance between the road and low chord of the bridge deck, and the distance between the road and high chord of the bridge deck.

# 4 HYDROLOGY

## 4.1 Introduction

Two hydrologic events were considered for the dam breach analysis: (1) the 0.2% annual chance event (formerly referred to as the 500-year event), and (2) the General Storm Probable Maximum Flood (PMF), which is the flood resulting from this Probable Maximum Precipitation (PMP) event. Regarding the PMP event, two types of PMP events should be considered for a dam structure: (1) General Storm PMP that represents an extreme winter storm with pronounced precipitation over a large area and long durations (3 days for areas covering up to 10,000 square miles); and (2) Local Storm PMP that represents intense localized thunderstorms during the warm season (6 hours for areas covering less than 500 square miles). Based on a detailed study of the Willamette River (USACE, 2017a), the general storm PMP within the Willamette River Basin (WRB) will produce a significantly higher runoff peak discharge and volume than for the local storm PMP event, and the critical duration for inflow was 72 hours. As a result, the

PMF based on the general storm PMP and a 72-hour duration was considered for the PMF and 0.2% annual chance of exceedance (ACE) flood event in this evaluation.

## 4.2 Model Development

Recently, WEST worked with the U.S. Army Corps of Engineers Portland District (NWP) in the development of a Hydrologic Modeling System (HEC-HMS) model of the WRB to determine the runoff hydrograph from the watershed for rainfall and snowmelt processes. HEC-HMS was developed by the USACE Hydrologic Engineering Center (HEC) and is designed to simulate the complete hydrologic processes of dendritic watershed systems. It includes traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. It also includes procedures necessary for continuous simulation and to simulate snowmelt (USACE, 2015). The HEC-HMS model is a detailed, distributed model comprised of averaged subbasin parameters. The model was calibrated to five winter flood events and four spring flood events. Information about the recent development and calibration efforts of the Willamette River HEC-HMS model is documented in *Willamette River Basin Dams, Hydrologic Model Calibration and Validation Report* (USACE, 2016) for the upper watershed and in *Lower Columbia River Stage-Frequency Study – Lower Willamette HEC-HMS Model Development and Calibration Technical Memorandum* (WEST, 2018).

The Silver Creek Dam watershed is located within one of the Pudding River subbasins in the WRB HEC-HMS model, and calibration of the Silver Creek USGS gage was considered for the evaluation of the lower WRB. As a result, the HEC-HMS model was developed from the lower WRB HEC-HMS model. Detailed information about the model parameters is provided in the following paragraphs.

### 4.2.1 Meteorologic Model - Precipitation

As part of the 2016 USACE Study, gaged hourly precipitation data within the vicinity of the WRB was gathered from the National Oceanic and Atmospheric Administration's (NOAA) National Climatic Data Center (NCDC) website (NCDC, 2016). The precipitation spatial and temporal depths for the various historic events considered in defining the WRB Dam PMF were obtained from the 2016 USACE Study and used to define the PMF for the Silver Creek Dam.

### 4.2.2 Meteorologic Model - Evapotranspiration

In the WRB, large precipitation events generally occur in the cold and wet winter and early spring season. These conditions do not allow for a significant amount of evapotranspiration to occur during large precipitation events. Therefore, evapotranspiration was assumed to be zero for the purposes of this study.

### 4.2.3 Meteorologic Model - Snowmelt

As in the 2016 USACE Study, snowmelt is estimated in the HEC-HMS model using the

Temperature Index Method and the recommended snowmelt parameters documented in *Development of Snow Model Parameters for the Willamette River Basin* (CRREL, 2016). The controlling condition for the initial Snow Water Equivalent (SWE) for the PMF of the WRB Dams was the initial SWE from the February 1996 flood event, so it was used for the Silver Creek Dam HEC-HMS model. More information about the snowmelt parameters can be found in Section 2.3.3 in the USACE 2016 Report.

For this study, the percentage within each elevation band was determined for the Silver Creek Dam watershed using ArcGIS and the WRB DEM. The majority of the watershed is within two elevation bands between 1,000 ft and 3,000 ft.

### 4.2.4 Basin Model – Soil Infiltration Loss

The soil infiltration loss rates used in the Silver Creek Dam HEC-HMS model are based on the Green and Ampt loss method, which is a physically-based approximation of Darcy’s equation. This method requires the initial soil moisture content, maximum soil moisture content, wetting front suction, hydraulic conductivity, and percent impervious area for each subbasin.

The subbasin average maximum soil moisture content, hydraulic conductivity, and wetting front suction values were developed for each subbasin using tables based on soil texture classification and the same procedure discussed in Section 2.3.4 of the USACE 2016 report. Since there is significant uncertainty in parameter extrapolations, further adjustments were made as part of the model calibration efforts.

### 4.2.5 Basin Model – Surface Loss

Total surfaces losses in HEC-HMS include the sum of interception and depression storage. In the 2016 USACE report, the interception was modeled using the canopy method in HEC-HMS, and depression storage was modeled using the surface method in HEC-HMS. The total canopy and depression storage were estimated using the same procedures presented in Section 2.3.5 of the USACE 2016 report. The average weighted interception and depression storage values were determined to be at 100%, reflecting no influences on the results, for all of the winter storm calibration considered for the 2016 USACE study. The calibration events considered for this study are either the same as or similar to the winter storm calibration events for the 2016 USACE study, so both of these storage components were assumed to be at 100%.

### 4.2.6 Basin Model – Clark Unit Hydrograph Parameters

The Lower WRB HEC-HMS model uses the Clark Unit Hydrograph Method for transformation of excess precipitation to runoff. The Clark Unit Hydrograph Method is based on the relationship of the cumulative area of the watershed contributing runoff with time. The ordinates of the time-area curve are converted to a volume of runoff for each unit of excess rainfall and interpolated to a given time step. The resulting hydrograph is then routed through a linear reservoir to simulate the storage effects of the basin and account for attenuation of the hydrograph flood peak.

Two parameters are required for the Clark Unit Hydrograph Method: (1) time of concentration for the basin, $T_C$; and (2) basin storage coefficient, R. In the Clark Unit-Hydrograph Method, $T_C$ is the time from the end of effective precipitation to the inflection point of the recession limb of the runoff hydrograph. The inflection point on the runoff hydrograph corresponds to the time when overland flow to the channel network ceases;beyond that time the measured runoff results from drainage of channel storage. Therefore, Clark's $T_C$ is the travel time required for the last drop of effective precipitation at the hydraulically most distant point in the watershed to reach the basin outlet. For this study, the initial $T_C$ was estimated to be equivalent to the time of travel for runoff from the point in the subbasin farthest from the subbasin outlet. This parameter was estimated using the same methodology applied for the USACE 2016 study outlined in the *Technical Release 55 (TR-55): Urban Hydrology for Small Watersheds* (NRCS, 1986), and documented in Section 2.3.6 of the USACE 2016 report.

The Clark storage coefficient is an index of the storage of excess precipitation in the watershed as it drains to the outlet. The parameter can be estimated via calibration or through regional relationships. Generally, the $R/T_C$ and $R/(R+T_C)$ ratios are fairly constant over time for a watershed and regional watersheds tend to have similar $R/T_C$ and $R/(R+T_C)$ ratios. The USACE 2016 study indicated that the R/(R+Tc) ratio should be 0.719. This ratio was refined during model calibration.

### 4.2.7 Basin Model – Baseflow

The baseflow considered in the HEC-HMS model was estimated using the Exponential Recession Method, which was selected for ease of adjustment during calibration. This method requires information related to the initial baseflow type (discharge or discharge per drainage area), initial discharge, recession constant, threshold type (discharge or ratio to peak), and discharge or ratio to peak value. The initial conditions can be defined either as a discharge or discharge per drainage area. The discharge option was used. The initial discharge is dependent on the conditions prior to the simulation period. The recession constant defines the decay of flow once the flow threshold value is met. The baseflow threshold can be defined as a ratio of peak flow or a specific discharge. The Ratio of Peak Method was used in the model.

Initial baseflow estimates were based on the recommended unit baseflow and recession parameters defined in the USACE 2016 study. The recommended unit baseflow is the average value from the winter calibration events, and the recession parameters are the minimum value from the winter calibration events.

### 4.2.8 Basin Model – Channel Routing

The Willamette HEC-HMS model uses the Muskingum-Cunge routing method. This method uses physically-based reach characteristics to compute flow attenuation and timing through the reach. The Muskingum-Cunge routing method relies on the conservation of mass and diffusion form of the momentum equation, which models non-uniform flow and hydrograph attenuation within the reach. The Muskingum-Cunge routing method does not account for backwater and it is generally used for reaches with slopes less than 10 feet per mile. The solution to Muskingum-Cunge routing also loses accuracy for rapidly rising hydrographs. Furthermore, a single, uniform, simplified cross section is used for each reach, which may not account for complexities in channel and floodplain geometry.

Data required for Muskingum-Cunge routing includes a representative simplified cross section for the entire reach, average channel bed slope, averaged Manning's roughness values, and reach length. The HEC-HMS model for this study is comprised of three types of routing reaches: (1) new reaches, (2) existing reaches that have been revised, or (3) existing reaches with no revisions. For the new cross sections, the geometric data including slope and 8-point cross section geometry were extracted from the best available LiDAR data in the Oregon LiDAR Consortium (Watershed Concepts, 2009).

### 4.2.9 Basin Model – Reservoir Routing

The Silver Creek Dam reservoir is the only reservoir included in the HEC-HMS model. The reservoir is reflected as a storage with the spillway outflow defined as an outflow rating curve and the 2019 elevation-volume relationship. The outflow rating curve was computed using the weir equation with a weir coefficient of 3.21 as determined in Table 5-7 in Handbook of Hydraulics (Brater and King, 1976).

## 4.3 Model Calibration

As previously stated, the WRB HEC-HMS model was calibrated to five winter flood events and four spring flood events, and final model parameters were recommended based on validation of the model to additional winter and spring events. The recommended parameters for the subbasin where the Silver Creek Dam watershed is located were used to define the initial parameters for Silver Creek Dam HEC-HMS model. The HEC-HMS model was calibrated to ensure that the unit discharge from the model matches the unit discharge derived from the available discharge-frequency information provided for USGS gage near the located within Region 2A of the 2005 USGS Study (USGS, 2005). The results of the calibration are summarized in Table 4-1. The calibration effort indicates that the model slightly over-predicts the unit discharge for the 1% ACE flood and slightly under-predicts the unit discharge for the 0.2% ACE flood. The initial and calibrated model parameters are summarized in
Table **4-2**.

**Table 4-1.** Summary of Calibration Results for Silver Creek Dam

| Flood Event | Unit Discharge per 2005 USGS Study(cfs/mi$^2$) | Simulated Unit Discharge (cfs/mi$^2$) |
|---|---|---|
| 1% ACE | 159 | 170 |
| 0.2% ACE | 206 | 203 |

**Table 4-2.** Summary of Calibration Model Parameters

| Model Parameter | Initial Model Parameter | Calibrated Model Parameter |
|---|---|---|
| Green & Ampt Content | 0.421 | 0.421 |
| Green & Ampt Content | 0.421 | 0.421 |
| Green & Ampt Suction (in) | 7.61 | 7.61 |
| Green & Ampt Conductivity (in/hr) | 0.028 | 0.025 |
| Percent Impervious (%) | 0.2 | 0.2 |
| Clark Unity Hydrograph Tc (hr) | 4.5 | 4.5 |
| Clark Unity Hydrograph Storage Coefficient (hr) | 11.4 | 11.4 |
| Baseflow Initial Discharge (cfs) | 7.5 | 8.0 |
| Baseflow Recession Constant | 0.85 | 0.85 |
| Baseflow Ratio to Peak | 0.1 | 0.01 |

## 4.4 General Storm PMP and PMF

The PMF flood inflow hydrograph to Silver Creek Dam was estimated using the calibrated HEC-HMS model, spatial and temporal distribution associated with historic events that have occurred within the WRB, and the 72-hour PMP precipitation depths recently determined for the entire WRB (USACE, 2017a).

As part of the PMF study for the WRB dams, an extreme storm study was completed on the WRB (USACE, 2016a). This study involved the identification of extreme historic flood events that have occurred within the WRB, collection of precipitation data for these events, and an evaluation of the top six precipitation events to define spatial and temporal distribution over the WRB (USACE 2017b) for those events. The top six measured precipitation events over the WRB are December 1964, February 1982, February 1986, February 1996, January 1974, and January 1997. The controlling condition for the Silver Creek Dam is the February 1996 event.

The 72-hour PMP precipitation depth for the Silver Creek Dam watershed was determined to be 29.0 inches using the PMP raster data sets developed for the WRB (USACE, 2017a) and the extraction tool developed as part of the study. The PMF flood inflow hydrograph to Silver Creek Dam is provided in Figure 4-1, which also shows the PMF hydrograph from the previous study completed by Philip Williams & Associates (PWA) in 2000. The peak discharge for the PMF event is about 22,300 cfs.

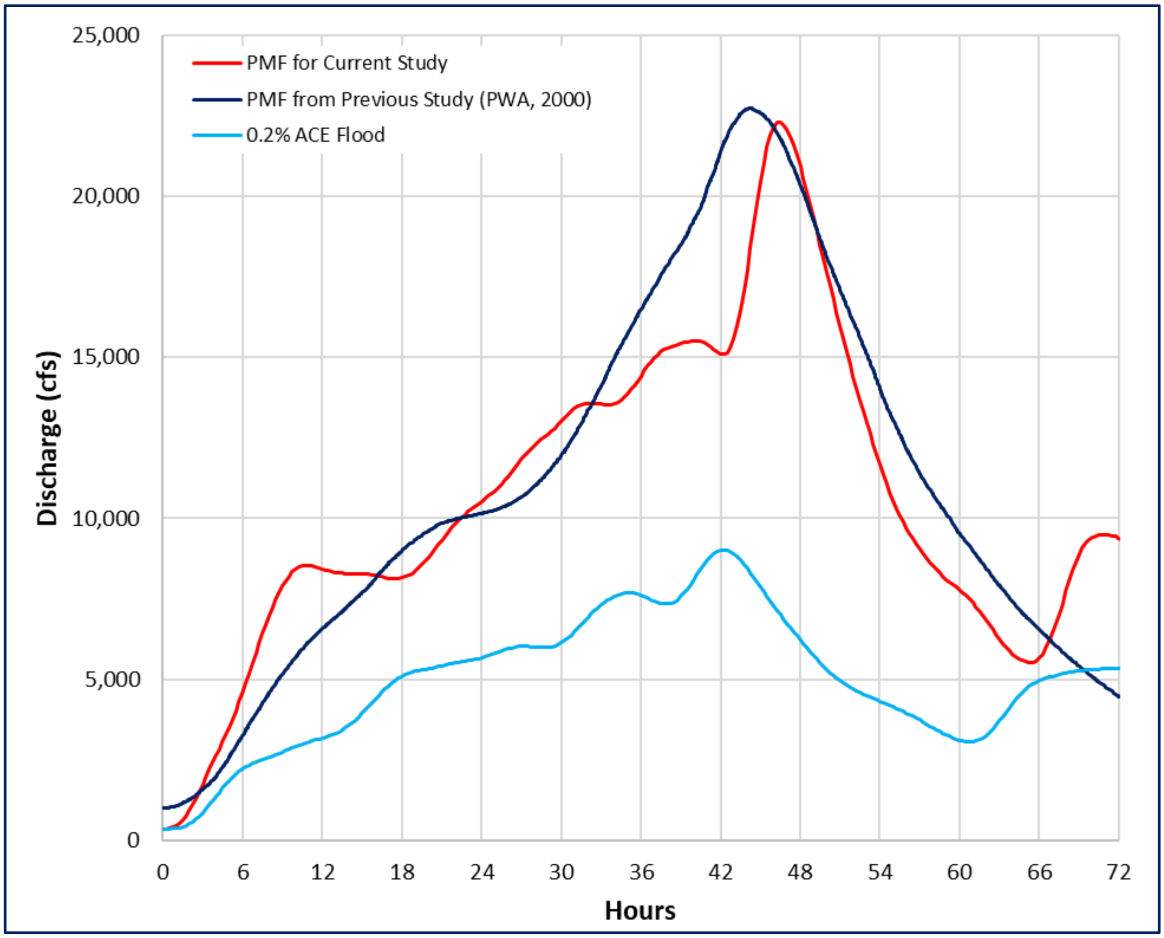


**Figure 4-1.** Inflow hydrograph to Silver Creek Dam for the 0.2% ACE and PMF events

## 4.5 0.2% Annual Chance Flood Event

The 0.2% ACE flood inflow hydrograph to Silver Creek Dam was estimated using the calibrated HEC-HMS model, spatial and temporal distribution associated with the February 1996 flood event (See Section 4.4), and the 72-hour 0.2% ACE precipitation depth. The precipitation depth-frequency relationship for the 24-hour duration was obtained from the raster data developed as part of the 2008 regional precipitation frequency study completed for the Oregon Department of Transportation (Schaefer et al, 2008). The extreme storm analysis completed by WEST in cooperation with USACE Portland District (USACE, 2016) indicated that the ratio between the 72-hour and 24-hour precipitation depths is about 1.63 for extreme storm events. This ratio was used to define the 72-hour precipitation depth-frequency depth of 14.2 inches. The 0.2% ACE flood inflow hydrograph to Silver Creek Dam is provided in Figure 4-1. The peak discharge for the 2% ACE flood event is about 9,000 cfs.

# 5 HYDRAULICS

## 5.1 Introduction

The Hydrologic Engineering Center River Analysis System (HEC-RAS) version 5.0.7 (HEC, 2010) was used for the breach analysis of Silver Creek Dam. HEC-RAS is a software package capable of computing one-dimensional (1D) steady and two-dimensional (2D) unsteady flow river hydraulics. It has the capability of modeling dam breach events under a wide range of scenarios.

A combined 1D/2D HEC-RAS model was developed for Silver Creek with a 1D reach being used for the entire reach upstream of the dam and for the main channel downstream of the dam and 2D areas for the floodplain areas in the reach downstream of the dam. Geometric features for the model were extracted from a Geographical Information System (GIS) using HEC-GeoRAS and ArcGIS. Three dam breach scenarios were analyzed: (1) a "Sunny Day" breach, (2) a breach during the 0.2% ACE flood, and (3) a breach during the general storm PMF flood. The objective of this modeling effort is to evaluate the impact of a dam breach on property downstream of the dam.

## 5.2 Development of the HEC-RAS model

HEC-GeoRAS was used to develop the HEC-RAS model. The study reach extends from about 1.5 miles upstream of the dam to the Brush Creek Drive SE Highway crossing, which is located about 5.1 miles downstream of the dam. The previous dam breach analysis (Philip Williams & Associates, PWA, 2000) indicated that a portion of the dam breach flood hydrograph would leave the Silver Creek system and flow north towards Abiqua Creek. The split flow occurs at about 3.1 miles downstream of the dam near Pine Street. The current study includes this split flow reach to about 1.5 miles downstream of the split.

### 5.2.1 Survey Data

As addressed above, no additional ground survey data was collected as part of this study. Existing LiDAR data was determined to be sufficient to define the extent of inundation associated with a potential dam breach.

### 5.2.2 Streams

Silver Creek downstream and upstream of the dam was represented as a single river reach in the HEC-RAS model. A plan-view of the 1D and 2D model reaches is shown in Figure 5-1.

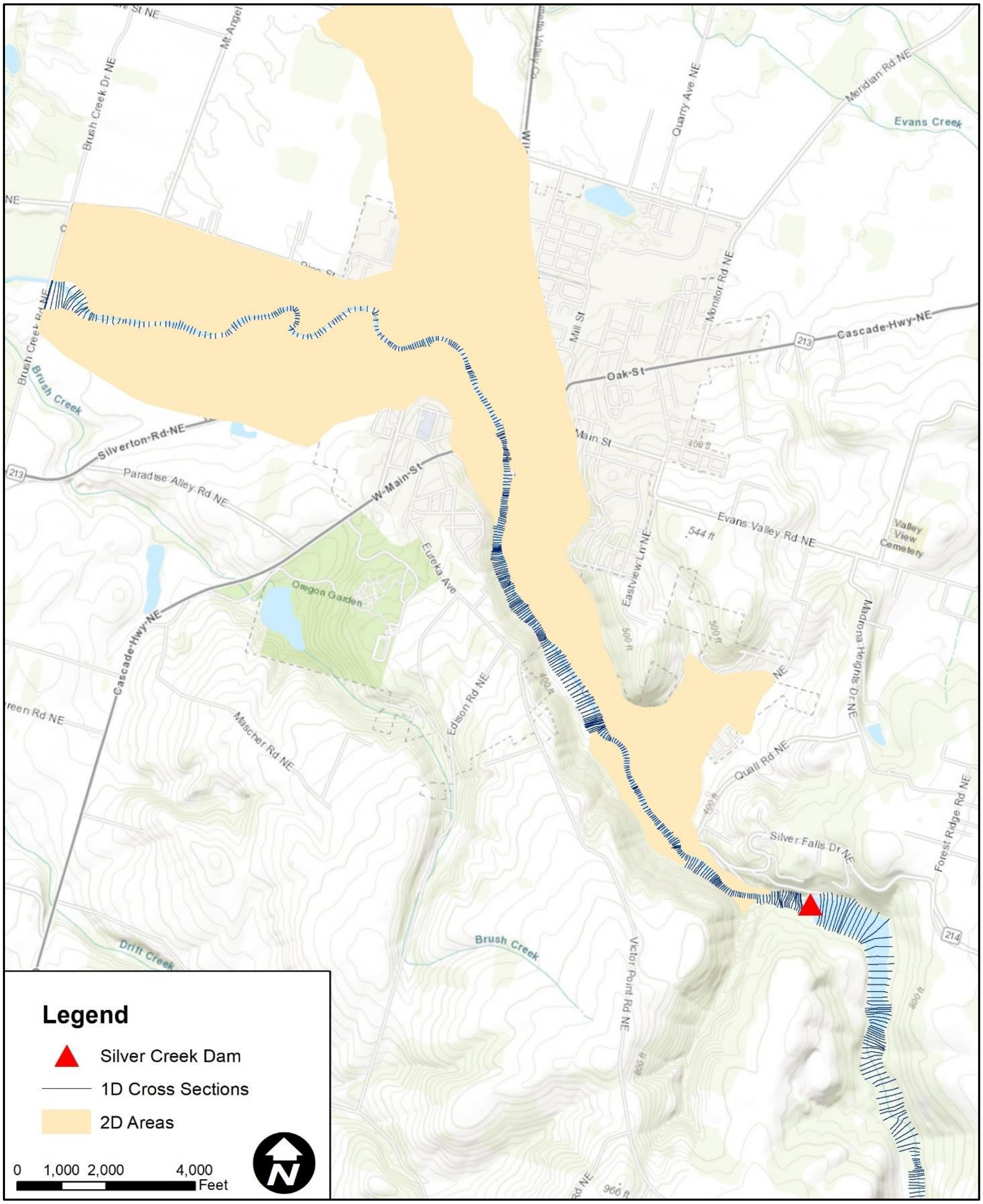


**Figure 5-1.** Plan view of HEC-RAS model geometry

### 5.2.3 Model Geometry

Cross sections are used to define the shape of the stream and characteristics such as hydraulic roughness and ineffective flow areas. A total of 753 cross sections were extracted from the Lidar DEM to define the bathymetry of the entire cross sections for the reach upstream of the dam and the main channel for the reach downstream of the dam. The cross sections were located to describe adequately geometric features such as roughness changes, grade breaks, expansions and contractions, and the numerical requirements for the solution scheme used by HEC-RAS. The cross sections were defined to be perpendicular to the expected maximum flood wave flow lines.

The 2D areas were defined for the overbank areas with a typical size of about 50 feet within urbanized areas and about 100 feet within the non-urbanized areas. The 2D areas are associated with the Lidar DEM and shapefile for the Manning's n-values, and several 2D breaklines were used to reflect adequately the flow along high ground features within the terrain.

As discussed above, the recent bathymetric survey of the reservoir indicated that it has a storage volume less than the design storage volume. The cross sections upstream of the reservoir were modified to re-establish the volume reservoir at the spillway crest. The modified cross sections were used in this study.

### 5.2.4 Structures

Eleven bridges over Silver Creek are located within the study reach downstream of the dam. Four of these bridges are privately-owned and provide landowner access to their land, one is a private railroad bridge, two are publicly-owned pedestrian bridges, and four are publicly-owned vehicle bridges. It was assumed that all of the bridges would wash out during a breach event except for the bridges at James Street, C Street, Main Street, and the railroad bridge. This assumption results in localized increases in the water surface elevations within a short reach upstream of each bridge and no significant changes in downstream flood levels or floodway arrival times.

### 5.2.5 Hydraulic Roughness Values

The Manning's n values for the stream channel downstream and upstream of the dam were estimated to range from 0.05 to 0.055, dependent on the slope for the channel bed, and 0.12 for the channel banks. This is a similar value to that previously used in the FIS for the Silver Creek (FEMA, 2007). The overbank 2D areas were estimated per various land-use types using information provided in *Australian Rainfall and Runoff Revision Project: two Dimensionally in Urban and Rural Floodplains* (EAWE, 2012). The land-use was developed from the latest aerial photograph of the study site, and is shown in Figure 5-2. The roughness coefficient per land use is summarized in Table 5-1.

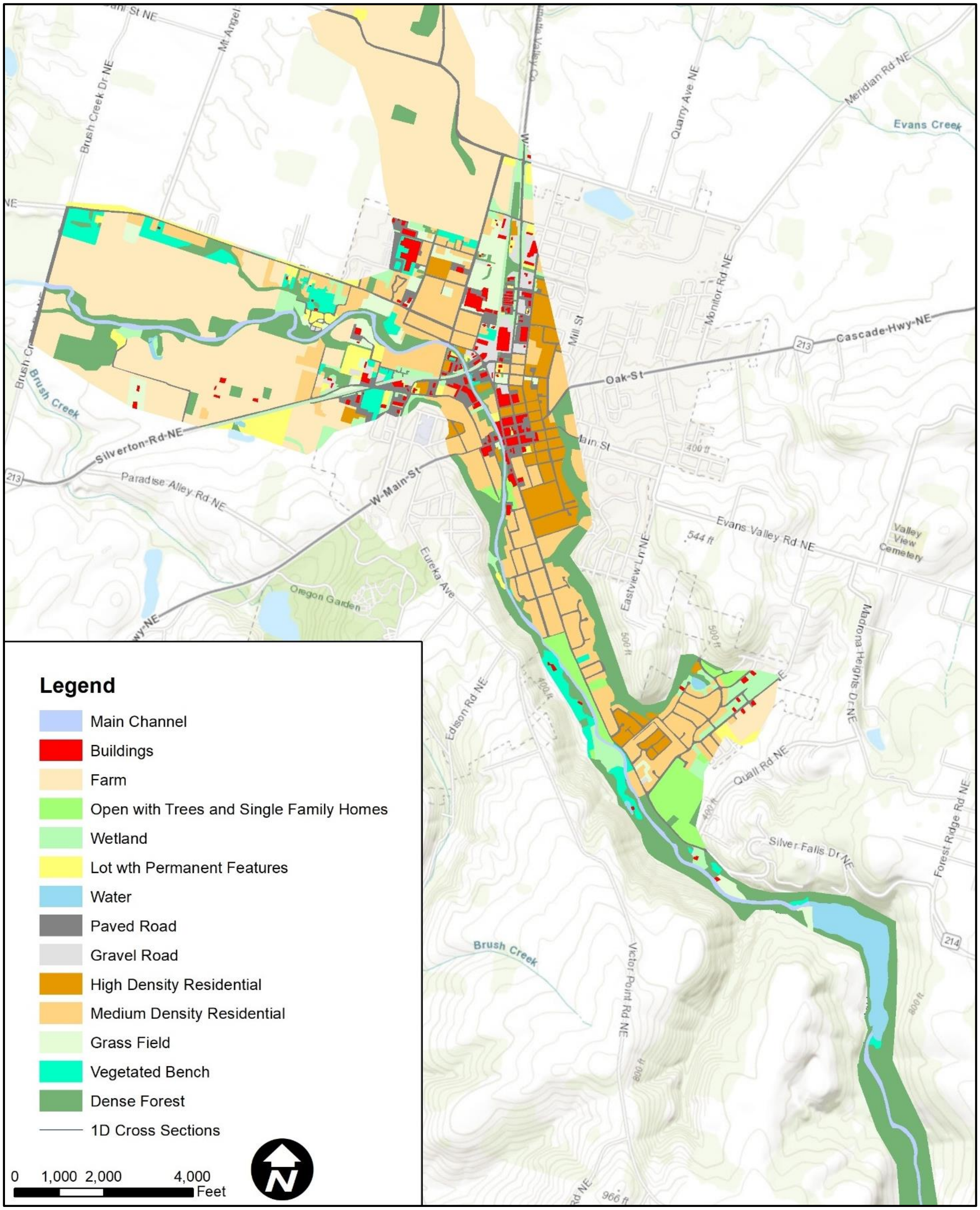


**Figure 5-2.** Plan view of land-use conditions

**Table 5-1.** Summary of Manning's n Coefficients per Land-use Type

| ID | Description | Manning's n Coefficient |
|---|---|---|
| 1 | Main Channel | 0.05 |
| 2 | Lake | 0.015 |
| 3 | Paved Road | 0.02 |
| 4 | Gravel Road | 0.028 |
| 5 | Residential HD | 0.2 |
| 6 | Residental MD | 0.15 |
| 7 | Grass Field | 0.03 |
| 8 | Vegetated Bench | 0.035 |
| 9 | Dense Forest | 0.12 |
| 10 | Buildings | 0.5 |
| 11 | Farm | 0.035 |
| 12 | Open with Trees and Single Family Homes | 0.045 |
| 13 | Wetland | 0.055 |
| 14 | Main Channel | 0.05 |

Because of the uncertainties associated with Manning's n values, a sensitivity analysis was performed by increasing and decreasing the Manning's n values by 20% to evaluate the effects of maximum and minimum roughness values.

### 5.2.6 External Boundary Conditions

For unsteady flow models, discharge hydrographs are typically used for defining the upstream boundary conditions. These input hydrographs may represent flood events such as the PMF or 0.2% ACE flood or a Sunny Day breach scenario with a constant base flow. For the Sunny Day breach scenario, a constant discharge of 50 cfs was defined at the upstream boundary, which represents a baseflow flow during the summer months. For the flood breach scenario, the hydrograph from the HEC-HMS model was used to define the upstream boundary hydrograph. Both the 0.2% ACE and PMF events were considered. Downstream boundary conditions can be set to normal depth, a rating curve, a known water surface elevation, or critical depth. The downstream boundary for Silver Creek was defined using a rating curve defined using the FEMA FIS report (FEMA, 2007). A normal depth slope of 0.0052 (0.52 percent slope) was used to define the downstream boundary of the split flow reach.

### 5.2.7 Computational Parameters

Often the most challenging aspect of dam breach modeling is maintaining computational stability for very dynamic and complex hydrodynamics. For this reason, options and tolerances providing the greatest model stability were selected for controlling the 2D computations in HEC-RAS. A theta of 1.0 provides for the greatest stability during the numerical iterations, but with a possible minor loss of accuracy. Theta was set to 1.0 to ensure the highest level of numerical stability for the thousands of different simulations carried out for the probabilistic analysis. The maximum water surface calculation tolerance was set to 0.01 feet. The Coriolis effect is only relevant in very large bodies of water and was not employed for this anlaysis. A time step of 1 to 2 seconds was used in the model. Finally, the Mixed Flow Option was used in the model for stability reasons.

# 6 DAM BREACH ANALYSIS

## 6.1 Breach Characteristics

The purpose of this study is to develop an inundation map for a potential breach of the Silver Creek Dam. Because this is a hypothetical event, the actual breach size, location, and timing are unknown and must be estimated. The estimation of the breach parameters provides a range of sizes and formation times and is discussed further in Section 6.2. Additionally, the location of the breach and the breach initiation must be estimated. A Potential Failure Modes Analysis (PFMA) was completed on Silver Creeks Dam by the USACE, Portland District (USACE, 2011). The PFMA workshop concluded that were four significant and credible potential failure modes with three of the modes being associated with the spillway performance and one being associated with overtopping of the dam. The workshop also indicated that failure modes related to seepage through the north abutment are considered credible but not as significant as the four modes previously mentioned. The most relevant credible potential failure mode for this study is overtopping of the dam embankment clay core. Piping failure from a 0.2% ACE event and a Sunny Day scenario were also considered for this study.

## 6.2 Determination of Breach Parameters

The parameters needed for the HEC-RAS dam breach model are rate of growth for breach progression, pool elevation at time of breach, breach mode, piping charateristics (breach discharge coefficients and initial piping elevation), breach geometry (shape, width, and breach side slope), and time to breach.

Breach progression can be characterized as either a linear function or as a sine wave. The sine wave relationship is believed to be a more physically realistic representation of a breach opening progression, enlarging gradually at first, then increasing rapidly, and finally slowing near the end of the breaching process. So, a sine wave was considered for this study.

**Figure 6-1.** Silver Creek Dam crest (looking south)

The pool elevation at time of breach was set to the spillway elevation for the Sunny Day scenario and at the maximum pool elevation for the 0.2% and PMF Rainy Day scenario. The breach mode is piping for all of the scenarios except for the PMF where an overtopping mode was defined. The piping conditions are based on a breach discharge coefficient of 2.6. An initial piping elevation was set at 1 foot above the minimum elevation at the outlet conduit, which is at 375 ft NAVD88.

The breach parameters for breach geometry and time to breach were estimated using the empirical breach parameter equations (Wahl, 1996): (1) Froehlich (1995 & 2008), (2) Von Thun & Gillette, and (3) MacDonald & Langridge-Monopolis. The selected model parameters are based on the Froehlich (1995) given the performance of this equation as noted in *Guidelines for Dam Breach Analysis* (Colorado Dam Safety Branch, 2010). Past research shows that there is a high level of uncertainty associated with predicting breach parameters. While case study data exist in literature, the relationships between individual parameters are not fully understood. Therefore, the analysis took into account this uncertainty by considering Low and High Breach flow conditions for the Sunny Day and PMF Rainy Day scenarios. The low parameter values result in the lowest expected peak flow from a breach, based on using the most non-conservative breach parameter values within an acceptable and realistic range. The high parameter values result in the highest expected peak flow from a breach, based on using the most conservative, yet realistic, breach parameter values within an acceptable and realistic range. The dam breach parameters used for this study are summarized in Table 6-1.

**Table 6-1.** Breach Parameters for Silver Creek Dam.

| Parameter | Breach Conditions | | |
|---|---|---|---|
| | **Low** | **Adopted** | **High** |
| **Sunny Day Scenario** | | | |
| Bottom Width (ft) | 47 | 47 | 134 |
| Formation Time (hrs) | 0.4 | 0.4 | 0.5 |
| **0.2% Rainy Day Scenario** | | | |
| Bottom Width (ft) | 59 | 59 | 155 |
| Formation Time (hrs) | 0.5 | 0.5 | 0.6 |
| **PMF Rainy Day Scenario** | | | |
| Bottom Width (ft) | 68 | 68 | 171 |
| Formation Time (hrs) | 0.6 | 0.5 | 0.6 |
| Side Slope | Vertical | 0.9 | 0.5 |
| Selected Breach Equation | Froehlich (1995) for Width/ Froehlich (2008 for Side Slope and Formation Time | Froehlich (1995) | Von Thun & Gillette |

## 6.3 Breach Analysis Results

The Silver Creek Dam breach analysis considered the Sunny Day, 0.2% Annual Chance Flood, and PMF breach scenarios. The breach analysis results for various locations along Silver Creek downstream of the dam are summarized in Table 6-2. Similar results for various points of interest within the City of Silverton are summarized in Table 6-3. The PMF breach scenario produced the largest peak discharge, downstream water surface elevations, and inundation extents. In all scenarios, there would be pronounced flooding in the City of Silverton with the arrival time at the Schooley Road bridge being 18 minutes for the Sunny Day, 13 minutes for the 0.2% ACE scenario, and 12 minutes for the PMF scenario. Relative to the points of interest, the biggest concerns are Silver Gardens Care Facility and the intersection of S. Water and Mooney Road.

Dam breach inundation boundaries for the study reach were delineated to the DEM using HEC-RAS for the three breach scenarios. Dam breach inundation maps for each scenario are provided in Appendix B, and all digital files used to create these maps are provided in the DVD included in Appendix C.

**Table 6-2.** Silver Creek Dam Breach Analysis Results along Silver Creek

| HEC-RAS Cross Sections | Location ID | Discharge (cfs) | Max WSEL (ft) | Max Depth (ft) | Arrival Time (min) | Time to Peak (min) | Area of Inundation (acres) |
|---|---|---|---|---|---|---|---|
| **Sunny Dam Breach Scenario** | | | | | | | |
| 28159.97 | Dam | 32,100 | - | - | - | - | 814 |
| 25022.45 | | 30,500 | 353.1 | 15.2 | 12 | 23 | |
| 22999.56 | | 24,700 | 329.7 | 16.7 | 15 | 25 | |
| 21120.14 | Schooley | 26,300 | 316.6 | 18.3 | 18 | 29 | |
| 17005.38 | | 23,000 | 278.2 | 17.5 | 25 | 39 | |
| 14033.54 | Main St | 21,400 | 258.9 | 23.7 | 29 | 43 | |
| 12361.20 | C St | 20,900 | 245.1 | 18.0 | 31 | 47 | |
| 11559.40 | James St | 18,600 | 235.8 | 12.2 | 33 | 52 | |
| 8087.07 | | 17,200 | 217.2 | 14.8 | 40 | 62 | |
| 4101.66 | | 16,800 | 192.6 | 11.9 | 50 | 73 | |
| 49.94 | | 15,400 | 178.0 | 12.3 | 61 | 88 | |
| Split Flow Reach | | 1,830 | - | - | - | - | |
| **0.2% ACE Breach Scenario** | | | | | | | |
| 28159.97 | Dam | 46,800 | | | | | 1,048 |
| 25022.45 | | 45,400 | 355.7 | 17.8 | 9 | 29 | |
| 22999.56 | | 44,200 | 331.9 | 18.9 | 11 | 31 | |
| 21120.14 | Schooley | 41,100 | 318.0 | 19.8 | 13 | 32 | |
| 17005.38 | | 36,200 | 280.9 | 20.2 | 18 | 44 | |
| 14033.54 | Main St | 31,800 | 262.2 | 26.9 | 21 | 50 | |
| 12361.20 | C St | 31,300 | 246.9 | 19.8 | 23 | 54 | |
| 11559.40 | James St | 26,000 | 237.8 | 14.2 | 27 | 57 | |
| 8087.07 | | 24,900 | 218.4 | 16.0 | 33 | 66 | |
| 4101.66 | | 25,100 | 193.6 | 12.9 | 41 | 74 | |
| 49.94 | | 24,600 | 178.7 | 13.0 | 53 | 82 | |
| Split Flow Reach | | 5,080 | - | - | - | - | |
| **PMF Breach Scenario** | | | | | | | |
| 28159.97 | Dam | 77,600 | - | - | - | - | 1,180 |
| 25022.45 | | 73,700 | 359.9 | 21.9 | 10 | 25 | |
| 22999.56 | | 69,500 | 334.2 | 21.1 | 12 | 27 | |
| 21120.14 | Schooley | 61,500 | 319.1 | 20.8 | 12 | 31 | |
| 17005.38 | | 57,000 | 283.7 | 23.0 | 19 | 42 | |
| 14033.54 | Main St | 49,700 | 265.5 | 30.3 | 22 | 47 | |
| 12361.20 | C St | 46,700 | 249.0 | 21.9 | 25 | 51 | |
| 11559.40 | James St | 35,200 | 239.5 | 15.9 | 25 | 54 | |
| 8087.07 | | 33,800 | 219.5 | 17.2 | 34 | 61 | |
| 4101.66 | | 33,800 | 194.4 | 13.7 | 41 | 70 | |
| 49.94 | | 33,400 | 179.2 | 13.5 | 51 | 75 | |
| Split Flow Reach | | 12,640 | - | - | - | - | |

**Table 6-3.** Silver Creek Dam Breach Analysis Results at Points of Interest in the City of Silverton

| Location | Arrival Time (min) | Time to Peak (min) | Max WSEL (ft) | Max Depth (ft) |
|---|---|---|---|---|
| **Sunny Dam Breach Scenario** | | | | |
| City Hall | Not Flooded | Not Flooded | - | - |
| Eugene Field | Not Flooded | Not Flooded | - | - |
| S. Water/Main St | Not Flooded | Not Flooded | - | - |
| S. Water/Ike Mooney Rd | 22 | 30 | 322.0 | 2.0 |
| First St/"C" St | Not Flooded | Not Flooded | - | - |
| Silver Gardens Care Facility | 35 | 53 | 234.9 | 4.6 |
| Silverton Middle School | 48 | 61 | 230.7 | 2.0 |
| WWTP Building | 51 | 64 | 212.9 | 3.1 |
| Split Flow Reach | 58 | 68 | 218.1 | 1.1 |
| **0.2% ACE Breach Scenario** | | | | |
| City Hall | 50 | 60 | 261.5 | 0.7 |
| Eugene Field | 49 | 61 | 247.0 | 1.3 |
| S. Water/Main St | 53 | 63 | 256.4 | 1.1 |
| S. Water/Ike Mooney Rd | 21 | 35 | 324.3 | 4.3 |
| First St/"C" St | 46 | 60 | 240.0 | 0.9 |
| Silver Gardens Care Facility | 28 | 58 | 236.3 | 6.0 |
| Silverton Middle School | 43 | 64 | 232.0 | 3.3 |
| WWTP Building | 37 | 68 | 213.4 | 3.5 |
| Split Flow Reach | 52 | 71 | 218.9 | 1.8 |
| **PMF Breach Scenario** | | | | |
| City Hall | 37 | 50 | 262.6 | 1.8 |
| Eugene Field | 34 | 55 | 248.5 | 2.9 |
| S. Water/Main St | 41 | 53 | 257.4 | 2.2 |
| S. Water/Ike Mooney Rd | 14 | 31 | 326.8 | 6.8 |
| First St/"C" St | 32 | 65 | 241.9 | 2.9 |
| Silver Gardens Care Facility | 27 | 54 | 238.0 | 7.7 |
| Silverton Middle School | 33 | 60 | 233.5 | 4.9 |
| WWTP Building | 40 | 63 | 213.8 | 3.9 |
| Split Flow Reach | 40 | 74 | 220.2 | 3.1 |

Due to high level of uncertainty in the model parameters, a sensitivity analysis was completed on the breach parameters, removal of all bridges, and Manning's roughness conditions. The uncertainty in the breach parameters was accounted for by simulating the low and high breach conditions provided in Table 6-1. The uncertainty in Manning's roughness values was accounted for by adjusting them by ±20%.

The results of a sensitivity analysis are provided in Table 6-4 for the Sunny Day breach scenario and in Table 6-5 for the PMF scenario. The results indicated that breach parameters would have the most significant influences on the model result with the average

**Table 6-4.** Sensitivity Analysis Results of Silver Creek Dam for Sunny Day Breach Scenario

| | Adopted | Low Breach Condition | High Breach Condition | No Bridge | Low n Conditions | High n Conditions |
|---|---|---|---|---|---|---|
| **Silver Creek Dam** | | | | | | |
| Max Discharge (cfs) | 32,100 | 26,200 | 43,000 | 32,100 | 32,100 | 32,100 |
| Area of Inundation (acres) | 814 | 753 | 939 | 798 | 772 | 857 |
| **Main Street** | | | | | | |
| Arrival Time (min) | 29 | 29 | 27 | 29 | 26 | 32 |
| Time to Peak (min) | 43 | 42 | 43 | 42 | 38 | 47 |
| Max Depth (ft) | 23.7 | 22.3 | 25.5 | 17.7 | 25.1 | 22.3 |
| Max WSEL (ft) | 258.9 | 257.6 | 260.7 | 252.9 | 260.3 | 257.5 |
| Max Discharge (cfs) | 21,400 | 18,700 | 26,600 | 21,700 | 23,600 | 19,500 |
| **Silver Gardens Care Facility** | | | | | | |
| Arrival Time (min) | 35 | 35 | 32 | 35 | 32 | 38 |
| Time to Peak (min) | 53 | 53 | 51 | 51 | 47 | 59 |
| Max Depth (ft) | 4.6 | 4.1 | 5.5 | 4.6 | 4.1 | 5.0 |
| Max WSEL (ft) | 234.9 | 234.4 | 235.8 | 234.9 | 234.4 | 235.3 |
| Max Discharge (cfs) | 18,600 | 16,600 | 22,600 | 19,300 | 20,200 | 17,200 |
| **Split Flow Reach** | | | | | | |
| Arrival Time (min) | 58 | 60 | 53 | 56 | 51 | 64 |
| Time to Peak (min) | 68 | 69 | 63 | 66 | 60 | 75 |
| Max Depth (ft) | 1.1 | 0.9 | 1.4 | 1.0 | 1.1 | 1.1 |
| Max WSEL (ft) | 218.1 | 218.0 | 218.5 | 218.1 | 218.1 | 218.1 |
| Max Discharge (cfs) | 1,830 | 1,280 | 3,220 | 1,510 | 2,150 | 1,620 |

**Table 6-5.** Sensitivity Analysis Results of Silver Creek Dam for PMF Breach Scenario

| | Adopted | Low Breach Condition | High Breach Condition | No Bridge | Low n Conditions | High n Conditions |
|---|---|---|---|---|---|---|
| **Silver Creek Dam** | | | | | | |
| Max Discharge (cfs) | 77,600 | 59,900 | 94,700 | 77,600 | 77,800 | 77,500 |
| Area of Inundation (acres) | 1,180 | 1,150 | 1,208 | 1,176 | 1,146 | 1,202 |
| **Main Street** | | | | | | |
| Arrival Time (min) | 22 | 25 | 19 | 19 | 23 | 22 |
| Time to Peak (min) | 47 | 53 | 43 | 38 | 51 | 48 |
| Max Depth (ft) | 30.3 | 29.2 | 31.9 | 22.8 | 30.8 | 29.6 |
| Max WSEL (ft) | 265.5 | 264.5 | 267.2 | 258.0 | 266.1 | 264.9 |
| Max Discharge (cfs) | 49,700 | 43,500 | 60,300 | 49,500 | 51,600 | 48,500 |
| **Silver Gardens Care Facility** | | | | | | |
| Arrival Time (min) | 27 | 30 | 24 | 25 | 30 | 28 |
| Time to Peak (min) | 54 | 61 | 49 | 49 | 58 | 54 |
| Max Depth (ft) | 7.7 | 7.2 | 8.2 | 7.1 | 8.1 | 7.4 |
| Max WSEL (ft) | 238.0 | 237.6 | 238.5 | 237.5 | 238.4 | 237.7 |
| Max Discharge (cfs) | 35,200 | 32,400 | 39,100 | 36,200 | 37,900 | 33,000 |
| **Split Flow Reach** | | | | | | |
| Arrival Time (min) | 40 | 44 | 36 | 37 | 41 | 40 |
| Time to Peak (min) | 74 | 84 | 65 | 58 | 78 | 75 |
| Max Depth (ft) | 3.1 | 2.7 | 3.8 | 2.9 | 3.4 | 3.0 |
| Max WSEL (ft) | 220.2 | 219.8 | 220.8 | 219.9 | 220.5 | 220.0 |
| Max Discharge (cfs) | 12,600 | 9,900 | 17,800 | 11,700 | 13,500 | 11,500 |

increase in the maximum flow depth ranging from -1.1 ft to 1.2 ft, the breach discharge ranging from -18% to 34%, and resulting in the area of inundation ranging from -2.6% to 2.4%. The sensitivity results also indicate the following: (1) the inundation extents do not significantly change with a more pronounced change occurring for the Sunny Day scenario, (2) the removal of the bridges results in localized changes near the bridge structures and does not have a significant influence on the flooding depths within the overbank areas, (3) the lower n-value will result in slightly higher water surface elevations at Main St. and C St. bridges due to more flow being conveyed through the main channel, (4) the arrival times will be fast with the PMF having slightly faster arrival time, and (5) the arrival times do not significantly change (arrival time to Brush Creek Dr. SE bridge at about 5.1 miles downstream of dam ranges from 44 to 56 minutes for the PMF scenario and from 52 to 61 minutes for the Sunny Day scenario.

As previously stated, the breach analysis was completed using the re-established design storage volume for Silver Creek reservoir. The sensitivity analysis also considered the existing storage volume relationship. The sensitivity analysis results related to the reservoir storage volume is summarized in Table 6-6.

**Table 6-6.** Sensitivity Analysis Results of Silver Creek Dam Storage Volume

| | Sunny Day Breach Scenario | | PMF Breach Scenario | |
|---|---|---|---|---|
| | **Design Storage Volume Reservoir** | **Existing Storage Volume Reservoir** | **Design Storage Volume Reservoir** | **Existing Storage Volume Reservoir** |
| **Silver Creek Dam** | | | | |
| Max Discharge (cfs) | 32,100 | 27,200 | 77,600 | 69,500 |
| Area of Inundation (acres) | 814 | 723 | 1,180 | 1,165 |
| **Main Street** | | | | |
| Arrival Time (min) | 29 | 27 | 22 | 21 |
| Time to Peak (min) | 43 | 38 | 47 | 45 |
| Max Depth (ft) | 23.7 | 22.0 | 30.3 | 29.7 |
| Max WSEL (ft) | 258.9 | 257.2 | 265.5 | 264.9 |
| Max Discharge (cfs) | 21,400 | 18,200 | 49,700 | 45,200 |
| **Silver Gardens Care Facility** | | | | |
| Arrival Time (min) | 35 | 33 | 27 | 26 |
| Time to Peak (min) | 53 | 49 | 54 | 52 |
| Max Depth (ft) | 4.6 | 3.9 | 7.7 | 7.4 |
| Max WSEL (ft) | 234.9 | 234.2 | 238.0 | 237.7 |
| Max Discharge (cfs) | 18,600 | 16,000 | 35,200 | 33,300 |
| **Split Flow Reach** | | | | |
| Arrival Time (min) | 58 | 57 | 40 | 39 |
| Time to Peak (min) | 68 | 65 | 74 | 74 |
| Max Depth (ft) | 1.1 | 1.0 | 3.1 | 2.9 |
| Max WSEL (ft) | 218.1 | 218.0 | 220.2 | 220.0 |
| Max Discharge (cfs) | 1,830 | 1,040 | 12,600 | 10,200 |

The results indicate that changes in the reservoir volume have more of an influence on the Sunny Day scenario results than for the PMF scenario results: (1) breach discharge reduces about 15% for the Sunny Day scenario compared to about 10% for the PMF scenario, (2) inundation area extents reduces about 11% for the Sunny Day scenario compared to about 1% for the PMF scenario, (3) time to peak to Main Street bridge reduces by 5 minutes for the Sunny Day scenario compared to 2 minutes for the PMF scenario, and (4) flow depths at Silver Gardens Care Facility reduced by 0.7 feet for the Sunny Day scenario compared to 0.3 feet for the PMF scenario.

A comparison of the peak discharge, time to peak, and water surface elevations at key locations in the city from the current study to the 2000 study are provided in Table 6-7. A review of the Sunny Day scenario comparison indicates the following: (1) the peak discharges are slightly higher for the current study than for the 2000 study (average increase of about 19%), (2) the current study has larger time to peak values (average increase of about 15 minutes), and (3) the water surface elevations are higher (average increase of about 2 ft) for the current study than for the 2000 study except near the WWTP building. Possible reasons for the differences are faster breach formation time (0.4 hrs compared to 0.63 hrs), how breach parameters were estimated, modeling approach for routing the breach flow hydrograph, and differences in the elevation data reflected in the models. The decrease in WSEL at the WWTP building for the current study is most likely caused by the 2000 study not including the flow split east of the WWTP near Pine Street.

**Table 6-7.** Comparison of Current Study Results to 2000 Study Results

| Item | Sunny Day Breach Scenario | | | PMF Breach Scenario | | |
|---|---|---|---|---|---|---|
| | 2000 Study | Current Study | Difference | 2000 Study | Current Study | Difference |
| **Peak Discharge (cfs)** | | | | | | |
| Silverton Dam | 28,040 | 32,100 | 14.5% | 107,165 | 77,600 | -27.6% |
| Schooley Road | 24,425 | 26,300 | 7.7% | 84,431 | 61,500 | -27.2% |
| Main Street | 16,455 | 21,400 | 30.1% | 61,183 | 49,700 | -18.8% |
| James Street | 15,163 | 18,600 | 22.7% | 56,693 | 26,000 | -54.1% |
| **Time to Peak (min)** | | | | | | |
| Schooley Road | 12 | 29 | 17 | 15 | 31 | 16 |
| Main Street | 21 | 43 | 22 | 40 | 47 | 7 |
| C Street | 36 | 47 | 11 | 49 | 51 | 2 |
| James Street | 39 | 52 | 11 | 51 | 54 | 3 |
| **Water Surface Elevation (ft)** | | | | | | |
| City Hall | Not flooded | Not flooded | - | 265.0 | 262.6 | -2.4 |
| Eugene Field | Not flooded | Not flooded | - | 252.0 | 248.5 | -3.5 |
| S. Water/Main Street | Not flooded | Not flooded | - | 258.3 | 257.4 | -0.9 |
| S. Water/Ike Money Road | 320.0 | 322.0 | 2.0 | 326.0 | 326.8 | 0.8 |
| First Street/ C Street | 238.5 | Not flooded | 4.4 | 245.2 | 241.9 | -3.3 |
| Silver Gardens Care Facility | 234.8 | 234.9 | 0.1 | 241.0 | 238.0 | -3.0 |
| WWTP | 214.2 | 212.9 | -1.3 | 219.0 | 213.8 | -5.2 |

A review of the PMF scenario comparison indicates the following: (1) the peak discharges are lower for the current study than for the 2000 study (average decrease of about 32%), (2) the current study has slightly larger time to peak values (average increase of about 7 minutes), and (3) the water surface elevations are lower (average increase of about 3 ft) for the current study than for the 2000 study except near the intersection of S. Water and Ike Money Road where there is an increase of about 1 ft. Possible reasons for the differences are longer breach failure times (0.5 hrs compared to 0.22 hrs), modeling approach for routing of the breach flow hydrograph, differences in the elevation data reflected in the models, and differences in the PMF inflow hydrograph (Figure 4-1). The significant decrease in the WSEL at the WWTP building for the current study is associated with the current including the flow split east of the WWTP near Pine Street.

# 7 CONCLUSIONS

The breach analysis of Silver Creek Dam indicates that a breach of the dam would be devastating to the City of Silverton. In all scenarios, there would be pronounced flooding in the City of Silverton with the arrival time being less than 20 minutes for all three breach scenarios. The PMF breach scenario produced the largest peak discharge, downstream water surface elevations, and inundation extents. However, the consequences associated with the Sunny Day scenario would be higher than for the rainy-day scenario because there would be warning of a breach where more advanced warning would be associated with an extreme flood event from advancements in forecasting of storm events.

Dam breach inundation maps for each scenario are provided in Appendix B. These maps should replace the existing maps in the Emergency Action Plan (EAP) for the dam. Appendix B also include maps showing damage zones based on the criteria documented in *Damage to Residential Buildings due to Flooding of New Orleans after Hurricane Katrina* (Pistrika and Jonkman, 2010).

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