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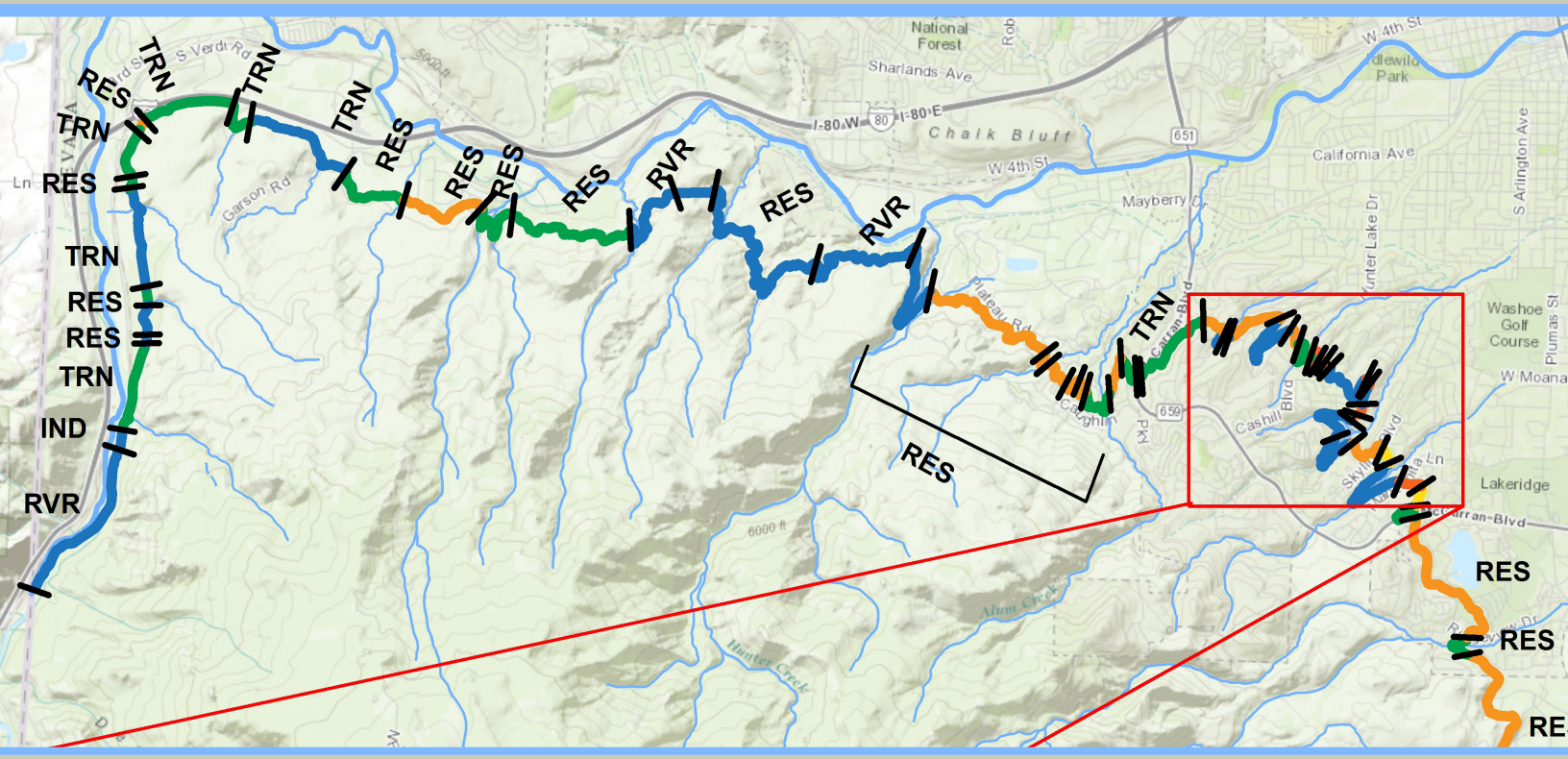
Nevada Bureau of Mines and Geology

Nevada Bureau of Mines and Geology Open-File Report 19-6

Flood Hazard and Mitigation Potential for Irrigation Ditches



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EXECUTIVE SUMMARY

INTRODUCTION, PROJECT GOALS, AND APPROACH

Irrigation ditches were generally designed to carry irrigation water, a steady flow along the ditch that is taken out at distribution point gates along the way. Urbanization around irrigation ditches changes the flood risk profile by increasing the consequences of flooding, increasing the number of tunnels and pipes along a ditch at road crossings, increasing the amount of storm water that can enter a ditch, and increasing the amount of debris in a ditch. One or more of these conditions can create the potential for damaging and/or dangerous flooding.

The ditch used for this study was the Steamboat irrigation ditch. We characterized the hydrologic setting of the ditch, historical flooding, conditions of the ditch, and potential downslope consequences to understand the causes and potential impacts of flooding.

FLOODING FROM IRRIGATION DITCHES IN RENO DURING THE 2005 NEW YEAR'S EVE STORMS

There were several instances of flooding from irrigation ditches in Reno during the 2005–2006 New Year's Eve storms. The ground was saturated from storms that occurred earlier in December 2005, when a series of storms dumped large amounts of rain on the city and adjacent western foothills over a period of about 48 hours. Runoff poured down the slopes of the foothills into western Reno and entered irrigation ditches in several places. Consequently, ditches swelled and overtopped their banks in several locations. Debris and sediment blockage of ditches occurred at low crossing bridges and where drainages intersected them causing water to back up, overtop embankments, and flood. Storm water also entered the ditch at drainage intersections and where diversion structures at major stream crossings were overwhelmed or failed, as well as from overland runoff and storm drainage discharge along the ditch. Flood waters from ditches passed through backyards, flowed long distances down streets, and entered and overwhelmed storm drainage systems. Flooding occurred as far away as 2 km (1.24 mi) from breaks in ditches where streets acted as conduits.

There were five areas of blockage in the ditches, three at road crossings and two at stream intersections. Three stream diversion structures were overwhelmed or failed, and there were at least 20 areas where ditches were overtopped or breached. A total of 33 locations in the Reno area reported problems related to flooding from the irrigation ditches.

The impacts of flooding ranged from minor landscape damage to the inundation of houses with as much as 1 m (3 ft) of water. Flooding occurred relatively quickly and without warning in most cases. In one instance the flood waters rose so rapidly that a water evacuation of a senior care facility was required. The costs of flooding were not directly reported but were estimated as a percentage of county assessed values of reported damaged properties and cross-checked with inspector estimates. The total of damage, repairs, and costs related to irrigation ditch flooding were estimated to be at least $1,300,000. Adding probable unreported damage and costs, the estimate increases to >$1,800,000.

A RELATIVE RISK INDEX FOR IRRIGATION DITCH FLOODING WITH AN APPLICATION TO THE STEAMBOAT IRRIGATION DITCH

We developed a simple, straightforward, and cost effective approach to indicate which sections of a ditch have higher relative risk than other sections. This approach can be used to prioritize additional investigations and/or flood mitigation actions along ditches throughout Nevada. We used the Steamboat irrigation ditch, constructed in ca. 1865, for the development of this index, because it is one of the largest irrigation ditches in the Reno area, and it has experienced historical flooding.

Risk is a consideration of hazard and consequences. In this study, we used two hazard factors (ditch construction and local setting) and two consequence factors (land usage and downslope distance) to develop the relative risk index. These factors were characterized using Google Earth imagery and limited field reconnaissance. We also produced a map of the hydrological points of interest that are important to consider when evaluating flood mitigation strategies.

About 15% of the ditch was characterized as having a relatively high risk potential for flood damage. It is recommended that those sections receive the first attention to flood issues, such as detailed investigations into ditch integrity, hydrologic modeling of the storm capacity of those sections, and identifying mitigation efforts that would reduce the risk of flooding. In addition, major and substantial streams should be separated from the irrigation ditch with robust diversion structures. Spillways and flood gates should be considered downstream of such intersections in the ditch to relieve storm water.

Hydrological points of interest were highlighted for consideration of flood risk potential. These points included the drainages studied by Pohll and Carroll (Appendix A) in their hydrological study of the Steamboat irrigation ditch, other drainages along the ditch, ditch spillways, road crossings over the ditch, and a tunnel through a hill. Road crossings can become debris and sediment choke points along the ditch if not designed and maintained properly. These hydrological points of interest should be acknowledged by the community, monitored if necessary, and considered in hydrological and flood mitigation studies.

ASSESSMENT OF FLOOD HAZARD FOR THE STEAMBOAT IRRIGATION DITCH—DESERT RESEARCH INSTITUTE HYDROLOGICAL STUDY

The most important parameter in considering whether a stream that crosses an irrigation ditch has significant potential to impact ditch flooding is the storm water discharge potential of the drainage. Pohll and Carroll (2013; see Appendix A) conducted a flood potential assessment of 35 watersheds along the Steamboat irrigation ditch in the area above Reno. For the six major drainages that cross the ditch, a numerical hydrological model was used that characterized the watersheds in terms of precipitation loss, direct runoff, stream base flow, stream flow routing, and snow melt potential. Twenty-four hours of precipitation from the 100-year design storm was used as input for the watersheds to estimate potential peak storm water discharge at the intersection with the ditch. The maximum flow capacity of the Steamboat irrigation ditch was estimated to be between 96 cfs and 131 cfs. Therefore, storm discharge entering the ditch of ≥100 cfs is considered likely to cause flooding of the ditch, and discharges of greater than 131 cfs are assumed to cause ditch flooding. The results indicate that major drainages have estimated peak discharges from a 24-hour 100-year-storm event that are a few to several times the capacity of the Steamboat irrigation ditch and have the potential to quickly overwhelm the ditch, cause it to overtop, and flood.

The analysis of the sub-watershed areas in the major drainages indicated that the Soil Conservation Service Curve Number Model and sub-watershed area were the most important factors in describing peak discharge. These parameters were used to develop a regression analysis to estimate the 24-hour, 100-year peak storm discharge for 29 smaller basins along the Steamboat irrigation ditch. The results were that seven of these drainages were expected to cause ditch flooding if the storm water entered the ditch (peak discharge >131 cfs), three were likely to cause flooding (peak discharge >100 cfs), and 19 of these drainages had peak discharge values of less than 100 cfs, which could be contained by the ditch, but could still contribute to other storm water discharge in the ditch and cause problems. The values from these smaller drainages can be used for future hydrological modeling that use different combinations of storm water contributions along sections of the ditch to evaluate its capacity to carry this amount of water.

IRRIGATION DITCH FLOODING SECTION OF THE NEVADA STATE HAZARD MITIGATION PLAN

Irrigation ditches are commonly not considered in flood scenarios, yet severe damage has occurred historically in Nevada from such flooding (see Appendix B). The failure of canals and ditches can result from severe storm runoff, structural weaknesses in levee walls, strong earthquake shaking, loss of dike integrity from rodent burrowing, diversion of storm water into ditches, and clogging of ditches with debris and sediment, particularly at road crossings.

FLOODING FROM IRRIGATION DITCHES IN RENO, NEVADA DURING THE 2005 NEW YEAR'S EVE FLOOD

Flooding in the Manzanita Lane area from the 2005 New Year's Eve storm.

INTRODUCTION

The 2005 New Year's Eve Flood in Reno, Nevada was caused by a series of Pacific storms that brought heavy rainfall to Northern California, southwest Oregon and western Nevada from December 17th through January 2nd. The heaviest rainfall occurred on December 30th and 31st with up to 8 inches (20.3 cm) of precipitation recorded in the Truckee / Tahoe region and up to 7 inches (17.8 cm) in the Reno area. The precipitation impacted local ephemeral streams, and caused the Truckee River in Reno to reach a peak flow of nearly 15,300 cfs on December 31, almost 20 times its average flow. Nearly 18 million dollars in damage was reported in Reno alone, making it the most costly and damaging flood in the area since the 1997 New Year's Flood. A considerable amount of flooding during the 2005 storms resulted from the overtopping and breaching of irrigation ditches in the Reno metropolitan area.

This report documents several damaging incidents from flooding of the Steamboat, Last Chance, and Lake ditches in Reno, including information on the weather event, flood descriptions, and damages. Reports, proposals, memorandums, photographs, and hand-written notes were reviewed to understand the details of the irrigation ditch flooding. As with many flood incidents, the story is incomplete because not all the effects were reported or recorded. Investigators have tried to supplement the incident records through field visits and resident interviews. A better understanding of the 2005 flooding from irrigation ditches highlights the causes and potential impacts of irrigation ditch flooding, creates an opportunity for greater awareness of flood potential and the development of mitigation actions.

WINTER 2005/2006 STORMS AND FLOODING

A series of storms from December 17, 2005 to January 2, 2006 brought large amounts of rainfall to western Nevada, the Sierra Nevada, and northern California. Earlier mid-December storms were not severe, but saturated the soils of watersheds that feed into the tributary streams in western Reno and the headwaters of the Truckee River. Ground saturation limited infiltration of precipitation in key areas, magnifying the effects of the larger rainstorms in late December. A major wet weather system was forecasted as early as December 26, but the rapid growth in the intensity of the storm on December 30 as it approached the Pacific Coast gave little time to issue general warnings and encourage preparation.

The largest of the storms came on December 30 and lasted late into New Year's Eve (December 31). A gage at Donner Lake measured 8.37 inches (21.3 cm) of precipitation during December 30 and 31. In the foothills above Reno, rainfall was measured at 5.4 inches (13.7 cm) over the same 48 hour period, contributing a large of volume of runoff to the already saturated watersheds in the western foothills. The Truckee River's daily mean flow dramatically increased from 2,220 cfs on December 30 to 12,300 cfs on December 31, with a maximum flow of 15,300 cfs (figure 1). Flooding occurred in downtown Reno, Sparks, and several areas throughout the basin.

While the 2005 flooding in Reno was not as extensive as the 1997 New Year's Flood, the impact of flooding from irrigation ditch overtopping and failure was greater in 2005. The 1997 flood was caused by rain on snow in the high elevation upper reaches of the Truckee River basin, resulting in flooding of the Truckee River floodplain in Reno and Sparks. The 2005 flood resulted from local, high-volume rainfall in the foothills west of Reno and on the valley floor, producing surface runoff that taxed and overwhelmed local drainages, irrigation ditches, and the city and county storm water systems. Irrigation water had been shut off earlier in October, and spill-point flash boards were pulled from spillways to drain water from the ditch into local drainages. However, the irrigation ditches were overwhelmed by the high volumes of runoff and locally were they were partially blocked with sediment and debris. In addition, the failure or overtopping of structures designed to keep crossing stream flow out of ditches contributed to the storm water in the ditches.

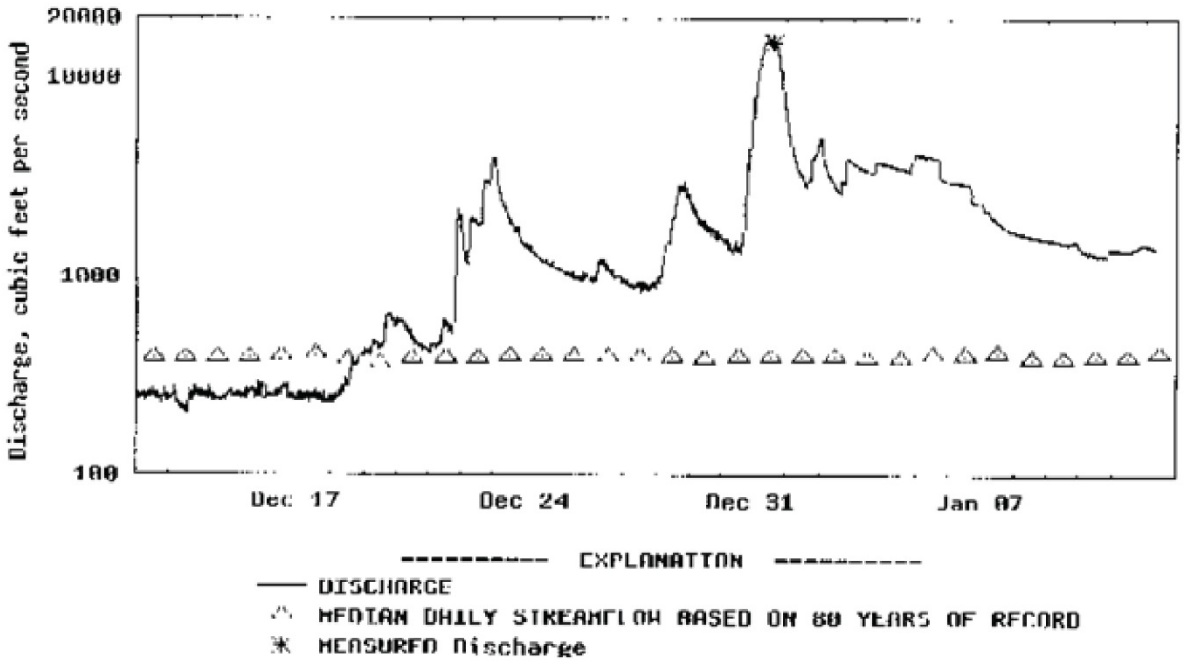

Figure 1. Truckee River discharge before, during, and after the flood event (USGS graph).

At least 20 documented areas of overtopping or breaching of irrigation ditches occurred between December 30, 2005 and January 2, 2006. Thirty-three locations reported damage from this flooding, and it caused over one million dollars in damage and cleanup costs. The hardest hit areas were in southwest Reno, where several overflow areas or embankment breaks occurred along the Last Chance, Steamboat, and Lake Ditches, inundating backyards, streets, and homes with flood water.

The storms eased as the New Year came, and flood waters receded rapidly. The event was declared a disaster by Reno, Sparks, and Washoe County, and the county Regional Emergency Operating Center was activated. On December 31st, the center issued several press releases that gave flood updates and flood safety information. The event was never declared a federal disaster, so those affected by the flood waters were not eligible for federal disaster relief loans. Total damage from the 2005 flooding was estimated at almost $18,000,000, making it the most damaging and costly flood in the Reno area since 1997.

2005 IRRIGATION DITCH FLOODING LOCATIONS

Individual cases of overtopping or breaks of irrigation ditches in 2005 are discussed in this section and their locations are shown in figure 2. Unless otherwise noted, the flooding occurred on December 31—the most significant day of rainfall—or early on January 1. The numbers in parentheses correspond with the locations labeled on figure 2. Only general areas are referenced. The numbers refer to the locations of several types of features, so missing numbers are mentioned in other discussions.

Mario Road and Dori Bell Lane (1)

The Steamboat Ditch traverses slopes south of the communities of Verdi and Mogul, west of Reno. This area is sparsely populated, but the ditch passes close to several homes. Near the intersection of Mario Road and Dori Bell Lane (location 1 on figure 2), the ditch is constructed near the base of the slope and a natural drainage is discharged directly into the ditch. During the storm on December 31 the ditch overtopped above Dori Bell Lane, flooding a residential property below and damaging one house, a private workshop, and the ditch maintenance road. Overtopping likely resulted from overland flow and runoff from the hillside above the ditch over about a 0.9 km (~0.6 mi) stretch from the last spillway and input from a drainage at about 0.25 km (~0.2 mi) up stream (figure 3).

Water Hole Road and Plateau Road (2)

The first large residential area the Steamboat Ditch flows through is the Caughlin Ranch development in the foothills of western Reno (figure 2). This area is below sparsely vegetated hillslopes and several natural drainages that discharge into the Steamboat Ditch. During the flood event, a diversion structure farther up the ditch at Hunter Creek (figure 4) was not functioning to full capacity, and stream water entered the ditch, in addition to local overland flow. On December 31, when the ditch was near capacity, the Caughlin Ranch Homeowner's Association excavated a breach and rerouted the water to an unoccupied drainage. However, the action was not successful in avoiding damage to all homes. Flood waters overwhelmed detention ponds on Juniper Hill

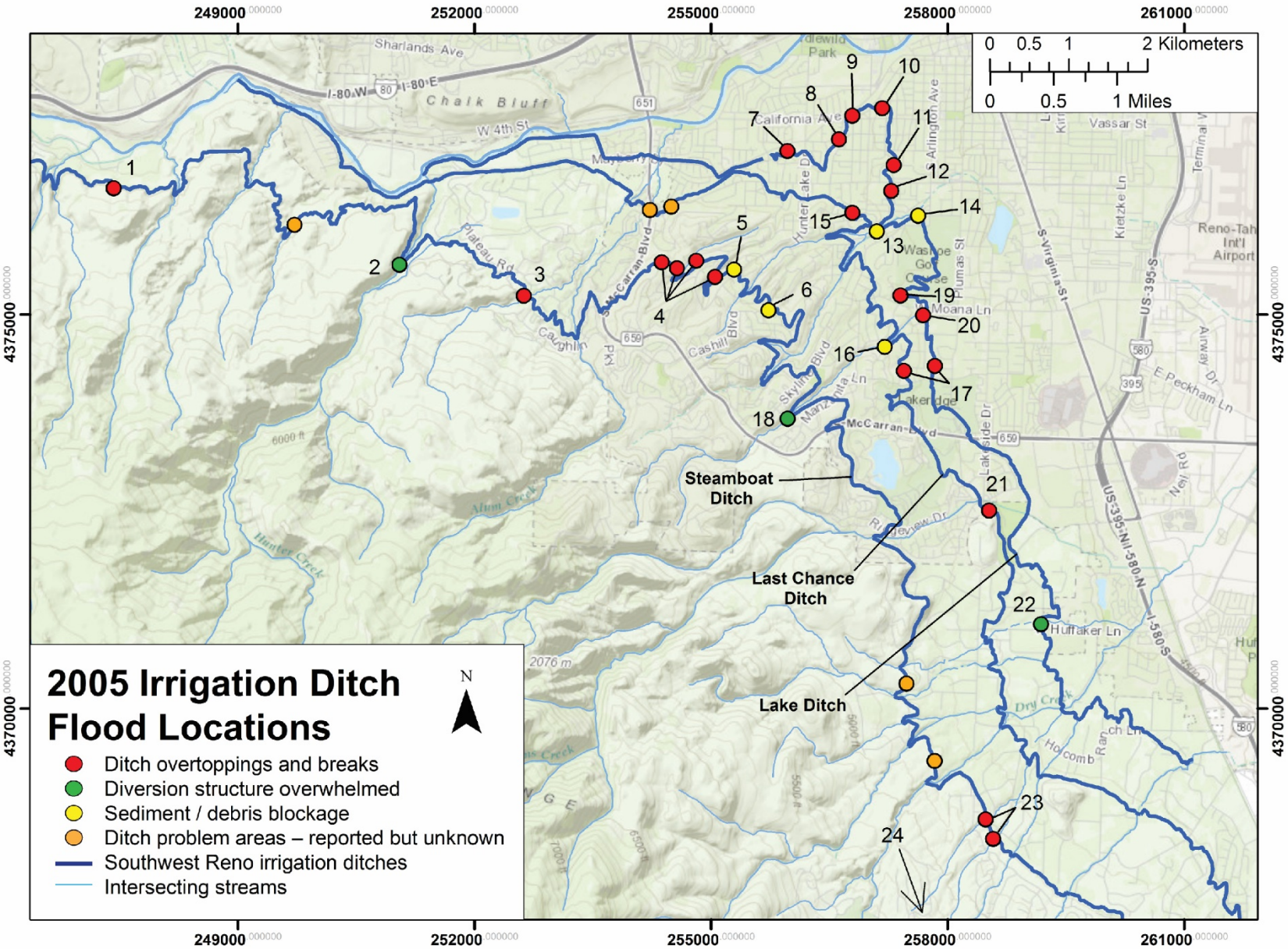

Figure 2. Locations of irrigation ditch overtopping, failure, and blockage from the 2005 New Year's Eve floods.

Figure 3. Google Earth image of location 1. Note the natural drainages that discharge into the ditch.

Figure 4. The problematic diversion structure at Hunter Creek that was overwhelmed during the flood event and added stream water into the ditch. This was later modified by the City of Reno. Image courtesy of Reno Public Works (January 10, 2006)

Road, resulting in a large amount of sediment being deposited on nearby residences.

McCarran Boulevard to Meridian Lane (4)

Storm runoff from small drainages and canyons near McCarran Boulevard caused waters to swell along a 2.5 km (~1.6 mi) section of the Steamboat Ditch (figure 2 location 4 and figure 5) Multiple locations of ditch overtopping and breaks occurred, partly due to blockage downstream where sediment and vegetation lodged in the culvert passing underneath Meridian Lane (see figure 21). Most of the flood waters from the ditch entered a natural drainage, combined with stream flow, and flooded several homes downstream. A witness described the flow as river sized. Floodwaters continued down Sagittarius Drive, Aquila Avenue, and Krupp Circle (figures 6–11), damaging low-lying homes and yards until dissipating at Pass Drive. The storm water system along this route was totally overwhelmed causing outflow from nearby storm drains (figure 11). Some of the floodwaters in this area may have entered the Lake and Last Chance ditches and accentuated downstream overtopping (locations 8–10).

Sediment deposition and water damage to roads reached as far as 2 km (1.2 mi) from the ditch breaks and overtopping locations. A large reconstruction project was carried out by the city to rebuild damaged roads, utilities, and drainage systems on the lower portion of Aquila Avenue.

Belford Road Area (6)

Overtopping of the Steamboat Ditch was reported in the Belford Road area where the ditch passes through a concrete box culvert. Overtopping flow in this area ran into a small natural drainage.

Fairbanks Court and Mayberry Drive (7)

This sizable overtopping, the farthest upstream in the Lake Ditch, eroded the edges of the concrete ditch to the point that shotcrete repairs were necessary. Floodwaters flowed into a backyard bordering the ditch and down Fairbanks Court before dissipating on Mayberry Drive. Though no damage was reported, photographs show damage to at least

Figure 5. Emergency repair areas along the Steamboat irrigation ditch (blue line) that were quickly repaired by contractors for the City of Reno after flooding. Image courtesy of Reno Public Works. (February 8, 2006)

Figures 6 and 7. Water channeling directly down Aquila Avenue (Left) and Sagittarius Drive (Right). Images Courtesy of Reno Public Works. (December 31, 2005)

Figure 8. Flood water that had been flowing down Aquila Avenue (left) crosses Plumb Lane (view west, north is to the right); photograph by Jon Price, NBMG. Water appears to be flowing down a topographic low, and here transitions from flowing mostly down streets to the left to flowing into people's yards on the right. White sand bags have been added to the black garbage bags at the driveway entrance (right center) that are highlighted in figure 9.

Figure 9. This house is on Plumb Lane directly across from where water was flowing down from Aquila Avenue. It effectively had a flood wall. Quick thinking residents filled black trash bags with dirt(?) and used them as sand bags to block the driveway entrance.

Figure 10. Water continued north of Plumb Lane into Krupp Circle. Water is gradually entering the storm water system, but at the end of Krupp Circle the excess water could only flow through yards and houses.

one home on the corner of Mayberry Drive and Fairbanks Court, as well as likely damage to landscaping on Mayberry Drive. Minor gullying in the slope from the overtopping was found during reconnaissance.

Secluded Circle and Charles Drive (8)

Water overtopped and breached the earthen banks of the Lake Ditch above Secluded Circle (figure 13). Flooding caused a retaining wall to fail and damage to private property.

Monroe Street and Marsh Avenue (9)

Where the Lake Ditch follows California Avenue, it is constructed into a steep embankment above several commercial businesses, and where it passes under Marsh Avenue, it passes through a few residential backyards. At least one home reported damage due to the ditch overtopping into their backyard near the top of the hill. This section was likely subjected to water backup aggravated by storm drain discharge into the ditch at Marsh Avenue. This area has previously overtopped and flooded during similar events, according to residents who border the ditch. Following the 2005 flooding, the Lake Ditch Company repaired and upgraded a 115-foot (35 m) section of the ditch in this area to help mitigate future overtopping.

Manor Drive and Marsh Court (10)

A small overtopping occurred where the Lake Ditch runs behind houses on Marsh Court. The flooding caused minor damage to at least one house and a small amount of sediment deposition. This overtopping may have been caused by water backup behind sediment and storm water discharged in the ditch at Rosewood Wash (figure 2, location 13), 300 feet (~100 m) downstream. Additional water likely entered the ditch from overland flow and from a storm drain on Marsh Avenue. Residents commented that the ditch regularly

Figure 11. The flood entered the storm water system and pushed it to its maximum capacity. In this photograph a significant amount of water was exiting out of a storm drain cover on Hunter Lake Drive.

Figure 12. Lake Ditch flowing directly onto Fairbanks Court and down to Mayberry Drive (foreground). Image courtesy of Reno Public Works. (December 31, 2005)

overtops in this area, but they do not recall any breaches of the embankment.

Dartmouth Drive and Nixon Avenue (11)

The Lake Ditch takes a relatively sharp turn near the intersection of Nixon Avenue and Dartmouth Drive. A section of the ditch immediately following this turn winds through backyards and according to a resident overtopped in 2005. The flooding pooled in a low-lying part of the neighborhood and damaged at least two properties. The overtopping was likely due to the sharp turn in the ditch at this point and from sediment deposited in the ditch downstream at Rosewood Wash (figure 2, location 13).

Margot Circle and West Plumb Lane (12)

The Lake Ditch passes underneath West Plumb Lane through a large culvert and continues through a neighborhood

Figure 13. Water from the Lake Ditch flowing out of the backyard of a Forson Drive residence, bordering Secluded Circle. Image courtesy of Reno Public Works (December 31, 2005)

Figure 14. A damaged home on Margot Circle. Sandbags are still up multiple weeks after the flood event. Image courtesy of Reno Public Works. (January 26, 2006)

passing under several smaller-volume overpasses. Near Margot Circle, the flow exceeded the capacity of the smaller culverts and overtopped, flooding a nearby road and several residential properties bordering Margot Circle and Upson Lane. Only two homeowners reported damage, but photographs and other information show that damage was likely more extensive. Sediment build up and water discharge downstream at Rosewood Wash (figure 2, location 13) and the resulting back-up likely caused the overtopping. There were reports from residents of other historical flooding in this area as well.

Homestead Place (14)

An underpass along the Lake Ditch downstream of Rosewood Wash clogged with sediment, and vegetation formed a debris dam and overtopping. A similar overtopping occurred during a more recent, much smaller rainstorm event.

Pineridge Drive and Sharon Way (15)

While the exact location of this overtopping is unknown, it likely occurred at a sharp turn in the Last Chance Ditch above Pineridge Drive. Water from the ditch channeled down Pineridge Drive onto Sharon Way (the Reno Gazette-Journal

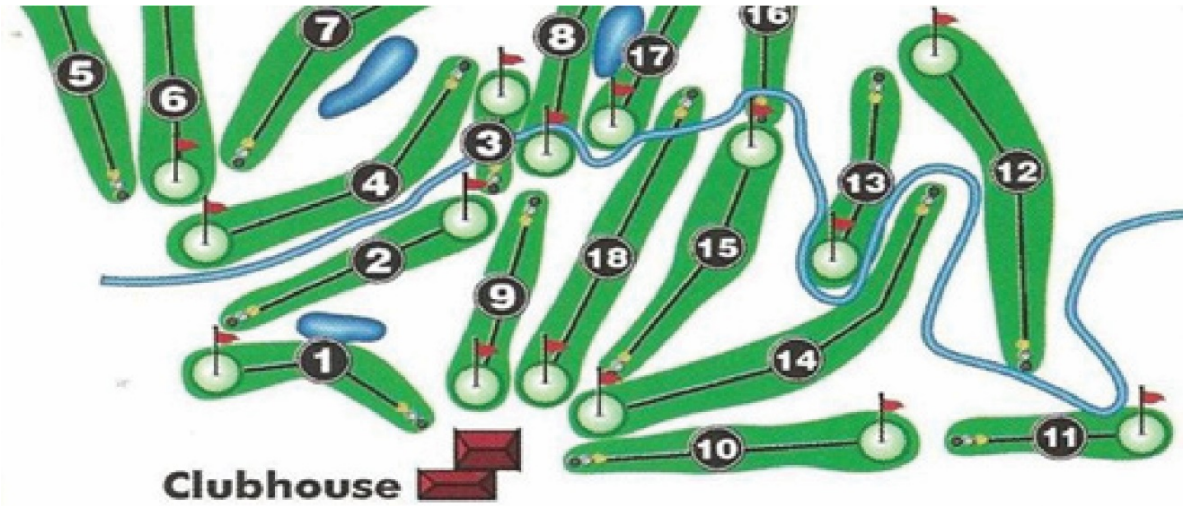

Figure 15. The layout of Washoe Golf Course. Notice sharp turns in the ditch near flooded holes, 11 and 12. Image from washoegolf.org.

noted that Lake Ditch contributed to the flow as well and flowed into Jessie Beck Elementary School, which received minor flood damage). Yards along Sharon Way also received some minor damage along the route of the flow. The overtopping may have been partly related to backed-up flow caused by water and sediment discharging in the ditch from Manzanita Wash (figure 2, location 16), 1.1 miles (1.8 km) downstream.

Moore and West Moana Lane (19)

Washoe County Golf Course is a public golf course, which sees roughly 30,000–45,000 games played per year. The Lake Ditch runs directly through the course, meandering back and forth serving as scenery and a water trap. This stretch of the ditch contains many tight turns in a low lying area that slow down flow increasing the likelihood of overtopping (figure 15). On December 31, Holes 11 and 12 of the course were flooded and suffered some sediment deposition, but the damage was relatively minor. Erosional damage to the ditch itself, though, was considerable and a sizable shotcrete repair had to be performed after the flood event.

Greenfield Drive and Pheasant Lane (20)

After passing through Washoe County Golf Course, the Lake Ditch crosses West Moana Lane and continues through a residential area where it ruptured, flooding private property on Greenfield Drive. The floodwater destroyed several concrete walls adjacent to the ditch, causing tens of thousands of dollars' worth of damage and flooded three homes damaging electronics, artwork, and other property.

The failure may have been caused by two very tight turns in rapid succession on either side of where the rupture occurred. Overland flow from city drainages at a bridge at Pheasant Lane and water from Manzanita Wash likely contributed to the high-volume flows at this location.

Figures 16 and 17. Up to two feet (~ .5 m) of water found its way inside five homes, and streets were inundated in the Manzanita Lane area. Flood waters came in so fast that folks in assisted living housing were trapped and had to be rescued (right). Image courtesy of Reno Public Works. (December 31, 2005)

Ayershire Court and Plumas Street (17)

Overtopping of both the Lake and Last Chance Ditches contributed to severe damage to neighborhoods near Manzanita Lane. Water overtopped the Last Chance Ditch several blocks north of Manzanita Lane, causing minor damage to roads and landscaping, and flowing down slope into Lake Ditch. This overtopping was likely partly caused by the increased volume from water discharged into the ditch at Manzanita Wash (figure 24). The overwhelmed Lake Ditch overtopped near Ayershire Court and floodwaters combined with overland flow from roads and neighborhoods, flowed down Plumas Street to Manzanita Lane and pooled into a group of five homes.

The flooding damaged roadways, vehicles, and garages and inundated several homes costing homeowners thousands of dollars in repair costs (figure 16). Repairs included the stripping of carpet, insulation, and drywall. A senior care facility was also flooded necessitating an evacuation using rafts (figure 17).

An account of a flood victim near Manzanita Lane

The inundated house was designed such that there are steps down from the entryway to the dining area and living room. The homeowner was at work on that very rainy day. Without warning, water started to flow into the house. One person who was inside escaped with the pets and nothing else. When the flooding stopped, there was 2 ½ to 3 feet (~1 m) of water standing in the house. Fortunately, the family had flood insurance (at a cost of $1,000 per year with a $5,000 deductible amount). Unfortunately, this meant the water could not be pumped out until an insurance adjuster could look at it. Three days passed, allowing the water to soak into the sheet rock and wall studs and for mold to set in. The family had to move out of their home for 6 ½ months into a rental that was over $1,000 per month in rent while all the sheetrock and carpet had to be torn out and most furniture disposed of. The insurance company allowed them to keep their antique furniture, which needed to be refinished and reupholstered and an antique piano, which need to be refurbished. They also lost two cars to the flooding. There was also a huge electric bill for the large fans that had to run to dry the studs out. Flood insurance didn't cover everything, and there was about $15,000 to $20,000 of out-of-pocket cost.

Flood damage caused a major disruption in a family's life for over six months. The total cost of the flooding to this house and family was estimated at $75,000 to $100,000. Even with flood insurance, the damage cost $10,000 to $15,000 in out-of-pocket costs. This underscores the value of flood insurance for those living near a ditch that has the potential of flooding.

Lakeside Drive and Evans Creek Drive (21)

The Last Chance Ditch between Bartley Ranch Regional Park and Lakeside Drive is cut into a relatively steep hill with hiking trails. On December 31, the hiking trails became

Figure 18. Water from the Last Chance Ditch flooding walking trails in the hills above Bartley Ranch Regional Park. http://tingandsteve.blogspot.com/2005/12/new-years-eve-2005-flood.html (January 1, 2005).

Figure 19a and 19b (below). Crossing of the Lake Ditch in a flume over a drainage channel near the intersection of Del Monte Lane and Huffaker Lane. This section of the flume was broken in 2005 and water from the ditch was added to the storm water going down the drainage.

conduits for runoff, and the park was inundated by floodwaters from overtopping of the ditch. Flooding damaged the Robert Z. Hawkins Amphitheater (ruining the stage, dressing rooms, seating, and some electrical equipment), a horse arena, and hiking trails at Bartley Ranch Regional Park (figure 18) Flooding from nearby Evans Creek also damaged streets, homes, and Bartley Ranch Regional

Park facilities. The overtopping of the ditch was likely caused by a combination of high flow down the ditch, water entering the ditch from Evan Creek, and overland flow into the ditch.

Del Monte Lane and Huffaker Lane (22)

The Lake Ditch in this area exists in a metal flume raised above the ground (figures 19a and 19b), adjacent to Del Monte Lane. During the storm, high water in a drainage gully knocked down supports for the flume collapsing about 30 feet (9 m) of it and releasing the ditch water into the drainage and flooding streets. No homes border the area and damage was limited to the flume itself, landscaping, and the nearby streets. The flume was found to be weak and poorly maintained after damage assessments, with old wooden supports and poorly constructed reinforcements. Heavy vegetation and debris-clogging culverts also contributed to the flooding.

Quilici Drive and Dryden Drive (23)

Excessive storm water ruptured the Steamboat Ditch in two nearby locations, releasing a large amount of water downhill. Homeowners on Quilici Drive had a series of detention ponds at the bottom of a natural drainage, but the heavy rains had already partially filled them and the water from the ditch caused them to overtop. Floodwaters flowed onto Quilici Drive and filled a barn with 6 to 8 inches (15 to 20 cm) of water causing significant property damage. Water also flowed from a second break directly down driveways and into basements, yards, and garages of four locations on Fairview and Quilici Drives. Significant erosion and sediment deposition caused damage to all of the properties affected. Storm drains that flow into the ditch from the above neighborhoods likely contributed to the overtopping of the ditch.

Old Ophir Road and Washoe Lake Drive (Location Not Shown)

On December 18, during a smaller storm event, the Galena Creek Diversion Ditch was breached, causing minor flooding in the Middlefield Place area. This resulted in complaints from the homeowners to the Galena Ditch Company, who in turn repaired the banks and repaired the damage. On December 31, a large volume of water from Galena Creek flowed into the ditch causing an overtopping that flooded five properties in Middlefield Place, resulting in greater damage than the December 18 event.

Sediment and Debris Dam Locations

Sediment and debris blockages occurred at road and major stream crossings (locations 5, 13, 14 and 16; figure 2) and were a major contributor to the overtopping of ditches. At location 13, sediment from Rosewood Wash (figure 20) dammed up Lake Ditch and resulted in backed-up flow and likely caused overtopping of the ditch at locations 11 and possibly 12. At location 5, a sediment and debris dam built up on the upstream side of a culvert under Meridian Lane along the Steamboat Ditch (figure 21), backed up flow, and

Figure 20. The intersection of Rosewood Wash and the Lake Ditch. The ditch flows from the upper right curving around to the lower right. Rosewood wash enters from the left center. A constriction of flow in the ditch occurs in the central part of the figure from sediment coming out of Rosewood wash. Image courtesy of the Reno Public Works Department.

Figure 21. Sediment and debris dam at the entrance of the tunnel under Meridian Lane along the Steamboat Ditch. Image courtesy of the Reno Public Works Department.

Figure 22. Back up of water and overtopping of the embankment in the Steamboat Ditch immediately up stream of Meridian Lane. The concrete shoulder of the tunnel and ditch are on the left-hand side of the photograph and the overtopped embankment is on the right. Image courtesy of the Reno Public Works Department.

caused the ditch to overtop in multiple places between Meridian Lane and McCarran Boulevard (figure 2, location 4; figure 22). At location 16, sediment built up at Manzanita Wash in the Last Chance Ditch (figures 24 and 25). Overflow from this blockage added flow in multiple locations to Lake Ditch below.

Overwhelmed Stream Diversion and Stream Crossing Structures

At least two stream diversion structures were overwhelmed during the storm. A diversion structure that carries water below the ditch at Hunter Creek (figure 2, location 2) and a structure that carries water from Manzanita Canyon over the ditch (figure 2, location 18) were both overwhelmed. This was likely the result of being partly clogged by debris. These added stream storm water into the Steamboat Ditch.

FACTORS THAT INFLUENCED DITCH FLOODING

The heavy rainfall from storms on December 30th and 31st caused a high volume of runoff to enter the irrigation ditches from storm drains, natural streams, and overland flow. This rainfall followed a series of storms that saturated the soils and exacerbated the volume of runoff that reached the irrigation system. Some overtopping of ditch embankments was simply because the flow capacity was exceeded. In several cases, debris and sediment that built up in the ditch at road-crossing tunnels and at drainage intersections caused water to back up in the ditch and overtopping to occur. Several diversion structures designed to keep natural streams and ditch waters separated were overwhelmed and added storm water to the ditches. Additionally, in southwest Reno the irrigation ditches are stacked upslope, so flooding from an upper ditch can flow into a lower ditch, increasing its storm water flow and flood potential.

IRRIGATION FLOODING COST ESTIMATES

Considerable damage was caused to public infrastructure, residential areas, and to the ditches themselves during the 2005 storm event. Unfortunately, clear accounting of the flood damage does not exist since many cleanup and repair costs were unavailable even though damages having occurred were reported. Additionally, there was damage that was not reported but could be inferred from photographs or the amount of flooding that occurred in an area. Damage costs were borne by ditch companies, individual homeowners, and the City of Reno. The City had entered into an agreement with the major ditch companies in 2002 to fund repairs, maintenance, and improvements at a percentage cost per foot in exchange for permission to use the ditches as storm water channels. Thus, the City had to cover the cost of the flood damage to infrastructure as well as some of the cost of the damage to the ditches.

Figure 23. Water from overflow and breaching of the Steamboat Ditch upstream from the Meridian Lane blockage. Image courtesy of the Reno Public Works Department.

Figure 24. High creek flow going down Manzanita Canyon, just below Dant Boulevard Detention Basin. Debris and water are entering the Last Chance Ditch in the center of the photograph. Image courtesy of the Reno Public Works Department.

Figure 25. Erosion of the bottom of the stream channel below Dant Boulevard Detention Basin. This eroded sediment went into the Last Chance Ditch.

Table 1. Reported Damage, Repair, and Cleanup Cost Estimates

	Sites 1–5 (Northwest)	Sites 6–20 (Central)	Sites 21–23 (South)	Overall Costs
Ditch Damage	$89,440	$230,290	$107,140	$426,870
Residential Damage	$206,190	$121,140	$94,880	$422,210
Public Damage	$265,500	$83,490	$61,100	$410,090
Total	$561,130	$434,920	$263,120	$1,259,170

Table 2. Reported and Unreported Damage, Repair, and Cleanup Cost Estimates

	Sites 1–5 (Northwest)	Sites 6–20 (Central)	Sites 21–23 (South)	Overall Costs
Ditch Damage	$89,440	$230,290	$107,140	$426,870
Residential Damage	$265,980	$346,840	$94,880	$422,210
Public Damage	$314,020	$222,770	$84,600	$621,390
Total	$669,440	$799,900	$286,620	$1,755,960

An estimate of the damage from ditch flooding was made using the City's repair costs, damage assessments, contractor quotes, and approximations of damage costs as a percentage of property values. Public infrastructure damage and cleanup costs were taken from contractor quotes and payment information for sediment cleanup. Residential damage costs were not reported, but were estimated using 10% of the county assessed value of properties that were damaged by the ditch flooding in all cases except for most of the damage along Aquila Avenue, the Middlefield Place damage area, and damage near Washoe Golf Course, for which 5% of assessed values were used because nearly all of the damage was to landscaping only. This is the same approach that has been used by the City of Reno and FEMA to estimate flood damage potential. An independent estimate of residential damage was generated from damage costs estimated by city and county inspectors and other known costs. This estimate was $500,000, which is close to the estimate made using a percentage of assessed property values ($422,210). All numbers reported are for damage and repairs only. Emergency response efforts, such as sandbagging and rescue, were not included.

Table 1 shows reported damage to homes, public infrastructure, and documented ditch damage estimates. There is evidence that several flood cleanups and repairs were not recorded; therefore, these estimates likely underestimate the overall cost. Using knowledge of the flooding routes, photographs, topographic information, resident interviews, and the relative size of the flow—along with comparative cleanup and repair costs—an estimate was made that includes the unreported costs (table 2).

SUMMARY

During the 2005 storms, there were a total of 33 reported locations of flood related problems along irrigation ditches. Irrigation ditches were overtopped, failed, or intentionally breached in 20 locations. There were five areas of blockage in the ditches, three at road crossings and two at stream intersections. Three stream diversion structures were overwhelmed or failed. Five other problems areas are noted, where we only know the location and no other details. There were likely other unreported cases of flooding, but we believe the most damaging incidents were recorded.

This post-flood analysis suggests that ditch blockages and overwhelmed diversion structures were a major contributing factor to ditch overtopping. Remediation of these problem areas would reduce the current irrigation ditch flood risk and improve the viability of the irrigation ditch system as flood mitigation structures.

SOURCES OF FLOOD INFORMATION

Codega and Fricke, Inc., Master Hydrology Report Index for Caughlin Ranch: Codega and Fricke, Reno, Nevada, June 1991.

Gray and Associates, Ditch Survey for Lake Ditch: Gray and Associates, Reno, Nevada, December 2nd, 2003.

Gray and Associates, Ditch Survey for Last Chance Ditch: Gray and Associates, Reno, Nevada, December 2nd, 2003.

Gray and Associates, Ditch Survey for Steamboat Ditch: Gray and Associates, Reno, Nevada, December 2nd, 2003.

Google Earth, Google Corporation, 2004–2014.

Hammond, S., Hammond, T., and Hammond, Z., 2005, The New Years Eve 2005 Flood: blogspot.com, January 31st, 2005.

National Water Information System: Web Interface, USGS 10348000 TRUCKEE RV AT RENO, NV: U.S. Geological Survey, December 28th 2005 to January 3rd, 2006.

National Water Information System: Web Interface, USGS 391930120165301 DONNER LK NR TRUCKEE CA: U.S. Geological Survey, December 28th 2005 to January 3rd, 2006.

Reno Gazette-Journal, 2006a, I-80 stretch may be closed until Monday: Reno Gazette-Journal, Reno, Nevada, January 1st, 2006, p. 4A.

Reno Gazette-Journal, 2006b, About 400 UNR sheep die after levee breaks: Reno Gazette- Journal, Reno, Nevada, January 3rd, 2006.

Reno Gazette-Journal, 2006c, Rivers lower as officials assess damage from flooding: Reno Gazette-Journal, Reno, Nevada, January 3rd, 2006.

Reno Gazette-Journal, 2006d, Snow makes damage hard to locate: Reno Gazette-Journal, Reno, Nevada, January 5th, 2006.

Reno Gazette-Journal, 2006e, Homes far from river get drenched when area ditches overflow: Reno Gazette-Journal, Reno, Nevada, January 8th, 2006, p. 13A.

Reno Gazette-Journal, 2006f, Region knew it was coming, but storm's strength had everyone scrambling: Reno Gazette-Journal, Reno, Nevada, January 8th, 2006.

San Francisco Chronicle, 2006, New Year's Storm Historic Regional Flooding: San Francisco Chronicle, San Francisco, California, January 1st, 2006, p. A11.

San Francisco Chronicle, 2005, Weather Forecast: San Francisco Chronicle, San Francisco, California, January 1st, 2006, p. E10.

Reno Rebirth, UNR farm, McCarran Crossing developments win Reno approval: Reno Rebirth, Reno, Nevada, May 8th, 2013.

Washoe County Assessor, GIS Map Warehouse: Quick Map, Nevada, 2014.

Washoe County Emergency Operations Center GIS Map, 2006.

Washoe County Public Works, Staff Report: Washoe County Public Works, Nevada, March 23rd, 2006.

A RELATIVE RISK INDEX FOR PRIORITIZING MITIGATION ACTIONS FOR IRRIGATION DITCHES

ABSTRACT

A relative risk index and hydrological points of interest were developed for the Steamboat irrigation ditch to use in prioritizing mitigation investigations and actions that reduce the chances of flooding. These were developed as prototype investigations that could be conducted by a ditch company or communities that lack the financial resources for more sophisticated and expensive probabilistic hazard analyses. The Steamboat irrigation ditch, one of the largest and highest elevation irrigation ditches in the Reno area, was used as an example.

The relative risk index was developed considering hazard and consequence factors. Factors considered in this study included ditch setting, ditch construction, downstream building type or land use, and downslope distance to potential consequences. These were characterized using Google Earth and limited field reconnaissance.

Two hazard factors for ditch flood potential were considered: ditch setting and ditch construction. The ditch setting can be related to the amount and character of ditch flooding that could occur. For example, where the ditch is on the side of steep terrain, a breach in the ditch embankment can release the entire flow in the ditch and the flooding can gain high velocities going downhill. In contrast, where the ditch is inset into relatively flat terrain, only overbank flow can flood. The settings index ranges from 0 for a tunnel (that confines the flow) to 5 for the ditch in a steep terrain setting. The consideration of ditch setting was motivated by the relatively high settings of the central part of the Steamboat irrigation ditch, where it is commonly located along hillsides. Ditch construction affects the potential for embankment failures and flooding. The Steamboat irrigation ditch is mostly constructed as unlined earth excavation and fill, which was assigned a value of 4. In some areas, the ditch is lined in concrete, a factor that reduces the chances of flooding by resisting water erosion and animal burrowing. These areas were assigned a value of 0 to give credit to the flood-resistant construction. The ditch setting and ditch construction values were added together to get a potential range of 0 to 9, with 9 having the highest flood potential. Although this index gives an indication of the ditch conditions, additional reconnaissance field review would be required to completely characterize these parameters.

Two consequence factors were considered, the building type or land usage downhill from the ditch and the distance to that feature. These were combined into a single consequence factor. Six categories of building type or land usage were considered, in order of increasing consequences with the assigned values in parentheses: agricultural features (1), recreational features (2), transportation features (3), industrial facilities (3), commercial buildings (4), and residential buildings (5). The distance categories considered were: 1–10 m (3), 10–100 m (2), 100–1000 m (1), and >1000 m (0). The consequences could be more completely characterized than the hazard in this study and the values from the two consequence factors were multiplied together to get an index that ranges from 0 to 15. This larger range added greater potential weight to the consequences than the hazard assessment in the relative risk index.

The relative risk index was determined by adding the two hazard factors to the consequence index, and ranges from 0 to 24. Areas with the highest index values of 17 to 24 made up about 15% of the ditch and should be considered first in prioritizing additional studies of the ditch or flood mitigation actions. Areas with intermediate values of 11 to 16 (41% of the ditch) should be considered next, followed by areas with values 0 to 10 (44% of the ditch). Although this index is only based on four factors and is derived in a simple way, it gives an indication of relative flood risk that intuitively makes sense. The areas with relatively high risk are those where downstream consequences are proximal to an unlined ditch along a steep hillslope.

Incidents that led to flooding from ditches in 2005 indicate that there are hydrological points of interest along ditches that should be specifically considered when evaluating flood potential and mitigation strategies. These include where ditches cross streams, tunnels or pipes under road crossings, and ditch spillway locations. Intersecting streams and tunnel entrances can cause flooding because of water and sediment discharge into a ditch. The most hazardous of these locations should be identified. These are commonly the largest streams, but may include a stream with a heavy sediment load or other condition. To reduce the potential for flooding, it is best if the highest risk locations are annually maintained, monitored for problems during storms, and are candidates for mitigation to reduce their potential hazard. Strategically placed spillways, storm drains, debris racks, and flood gates are the principal controls of storm water in irrigation ditches.

INTRODUCTION

A relative risk index can help prioritize ditch investigations and flood mitigation efforts, and can be made more easily and less expensively than a probabilistic flood study. In this study, we developed a relative risk index for the Steamboat irrigation ditch in Reno as a test case. Observations were limited to those that could be made on Google Earth, combined with limited field inspections to verify and measure some features. The Steamboat irrigation ditch is the largest and highest of the irrigation ditches in Reno, and is above three other ditches that can be impacted by its flooding.

Risk is generally a combination of hazard and consequences. In this study, we assessed two potential aspects of irrigation ditch flood hazards and two characterizations of consequences. Flood hazard input consisted of the setting of the irrigation ditch and the general

construction of the ditch as it related to resistance to ditch embankment failure. Consequences were described as buildings or land usage downslope from the ditch and the distance of these from the irrigation ditch. Consequences were estimated in 2014 and changes in consequences with time, such as future construction closer to the ditch, are anticipated.

The relative risk index is not intended to make specific detailed predictions of risk, rather it is a tool to help with the prioritization of future resources, such as detailed investigations. The main reason for exploring this tool is to design an approach that can be used by a wide range of irrigation canal owners throughout the state. Most ditch owners cannot afford detailed probabilistic studies of their canals or ditches, which require the collection of detailed information and a sophisticated analysis of these features and the potential impacts of flooding. By contrast, a ditch owner can conduct a relative risk index study inexpensively. It really just formalizes a series of straightforward observations. The index was constructed to cover a small range of values to make it easy to use and a relatively higher risk of flooding is indicated by larger values.

Other relative risk studies can use the factors used here or other factors as appropriate for the situation. Additional considerations to a relative risk index include making more detailed studies, such as field observations of the condition of ditches and their embankments. Additional factors might be hazard or consequence aspects, dependent on the intent, situation, setting, and resources.

The characterization of the indexes ranged from very straightforward calls to more equivocal situations, where the best call was made. Equivocal situations were in the minority of cases and the use of multiple inputs into the final relative risk value helps buffer their possible impacts. An example of an equivocal situation would be a building that is at an angle to the ditch and straddles two downslope distance categories. Equivocal situations are mentioned because they will likely be encountered in creating a risk index and we recommend that these not become the focus of great effort. A logical decision can be made expeditiously. In many cases, the higher hazard or higher consequence category can be assigned.

HAZARD-RELATED FACTORS

Two hazard-related factors were characterized: irrigation ditch setting and irrigation ditch construction.

Irrigation Ditch Setting

The Steamboat irrigation ditch crosses several different topographic settings. Some settings, such as steep slopes, can contribute to the flood potential, whereas other settings, such as a tunnel, tend to inhibit the chances of flooding. An index for the ditch setting with a relative ranking from 0 to 5 was created to describe these different topographic settings (table 3). A value of 5 has the most potential to contribute to flood hazard and a value of 0 has the least potential. Two conditions encountered in this study were added to this index: areas where large amounts of sediment are entering the ditch and tunnels under roads.

Table 3. Irrigation Ditch Setting Hazard Factors.

Factor	Setting
5	Ditch is built into a steep slope
4	Ditch is excavated into geologic unit along steep slope
4	Heavy sediment loads eroded into ditch
3	Culvert/tunnel under road – blockage potential
3	Ditch is built into a moderate slope
2	Ditch is built into a low slope
2	Ditch on flat terrain with houses along it
1	Ditch is surrounded by flat terrain
0	Tunnel through hill

Where the ditch was confined to a tunnel, the lowest setting value of 0 was assigned. This setting contributes towards preventing flooding, as long as the flow does not exceed the tunnel diameter at its entrance or blockage does not occur. A factor of 1 was assigned where the ditch was inset into relatively flat terrain. On flat terrain the ditch flow will be more confidently contained because the risk of embankment breaching is removed, but a ditch can be overtopped. A second category of flat terrain is considered where the ditch is surrounded by houses; this was given a factor of 2. Land development adds several impermeable elements, such as roadways and driveways, which increase storm runoff. Thus, surrounding houses can contribute runoff to the ditch, increasing the chances of flooding. Three downhill slope settings were considered: a low slope (2), a moderate slope (3), and a steep slope (5). Flooding from a ditch built on sloping ground is concentrated on the downhill side. Ditches built on lower slopes have a lower potential for high-velocity flooding that those built on steeper slopes. The low slope category includes landforms with a few degrees of slope. A common setting for the low slope category was where the ditch was on an alluvial flat at the base of hill. Moderate slopes (commonly several degree slopes) can have high-velocity floods, which can cause erosion that concentrates flood water. Steeper slopes (approximately >10°) have the highest potential flood velocities. A slightly lower factor (4) was used where the ditch was cut into geologic units on steep slopes. The potential for embankment failure was considered to be less for excavations in original geology versus constructed dirt embankments.

During the 2005 floods, water backed up and overtopped ditches because of sediment build up and blockage. In some areas with steep upper slopes and erodable geologic units, sediment eroded in large volumes was deposited into the ditch below (figure 26). These areas were assigned a high factor of 4 and are places were annual ditch cleaning maintenance is critical. Road crossings were assigned a factor of 3 as they are also places where potential water flow blockage can occur.

The results of the ditch setting analysis are presented in figure 27. Thirty-three sections were characterized. The most common ditch settings were a moderate slope (34%) and a low slope (27%). About 11% of the ditch was built along steep slopes and 10% was built in geologic units on a steep slope. About 0.8% of the ditch was built on relatively flat terrain and 14% on flat terrain surrounded by houses. The ditch was in a tunnel about 2% of its length.

Irrigation Ditch Construction

In some areas, the ditch was constructed in a way that resists breaching, such as with concrete lining. A few types of ditch liners could be observed on Google Earth and are listed in table 4. Most of the ditch embankments are constructed out of earthen materials that appear to be locally derived. These have functioned well as irrigation ditches for over 100 years, indicating some care was taken in design, construction, and maintenance. The potential hazard factors related to construction range from 0 to 4, with 4 representing a bare earthen ditch with no enhancements to resist flooding, 2 representing some enhancement to resist breaching, and 0 representing mitigation that resists flooding, such as concrete lining.

Figure 26. Loose sand and cobbles that are immediately upslope of the Steamboat irrigation ditch are a constant source of sediment into the ditch.

Table 4. Irrigation Ditch Construction Related to Flood Potential

Factor	Construction Feature
4	Earthen embankment(s) and ditch bottom
2	Riprap-lined ditch
2	Wood-line ditch
2	Culvert/tunnel/pipe under the road
0	Ditch is in tunnel through hill
0	Concrete-lined ditch

Earthen construction provides the least amount of protection against flooding and is the most common construction type for the Steamboat irrigation ditch (and for most irrigation ditches in Nevada). Figure 28 shows an example of an earthen construction. Earthen embankments vary in steepness and height, and embankments can have relative low points that can be more susceptible to overtopping. Earthen dikes commonly host vegetation that can help resist erosion, but debris from vegetation falling into a ditch and growth into a ditch can contribute to blockage. Because earthen construction offers the least amount of protection against failure, it was assigned the highest value of the index, 4.

Certain sections of the Steamboat ditch are lined with rock riprap, wood, or concrete. Figure 29 shows a section of the ditch that is lined with concrete. Where the ditch is lined, it resists erosion and the chances for breaching and flooding are lowered, substantially if the lining is in good condition. The most common form of lining found in the ditch was concrete, especially near major roads.

Concrete lined construction was considered to have the most resistance to breaching and was assigned a 0. Other types of lining offer some resistance to erosion, but can be eroded during high water flow, and are assigned an intermediate value of 2. This is a general assignment, and it is acknowledged that some rock riprap linings may offer more resistance to erosion than this value indicates.

In this study, the ditch was not inspected for the condition of the floor or embankment that might diminish the effectiveness of a structural enhancement. Because poor conditions, such as erosion behind a concrete lining in an embankment, can lead to ditch failure, such an inspection and addition of this hazard index is recommended.

The Steamboat irrigation ditch flows underneath many roads through a concrete tunnel or pipe. The entrances of these tunnels and pipes can be the locus of blockages that can cause flooding when large debris gets trapped. In several cases, there is concrete lining adjacent to tunnels helps counter the effects of local erosion if blockage occurs. Once water is in a tunnel or pipe, however, it is confined. Tunnels and pipes under roads are assigned a midrange value of 2, recognizing some benefits and some hazard.

The only other general construction type encountered was where the ditch flows through a tunnel in a hill. Given

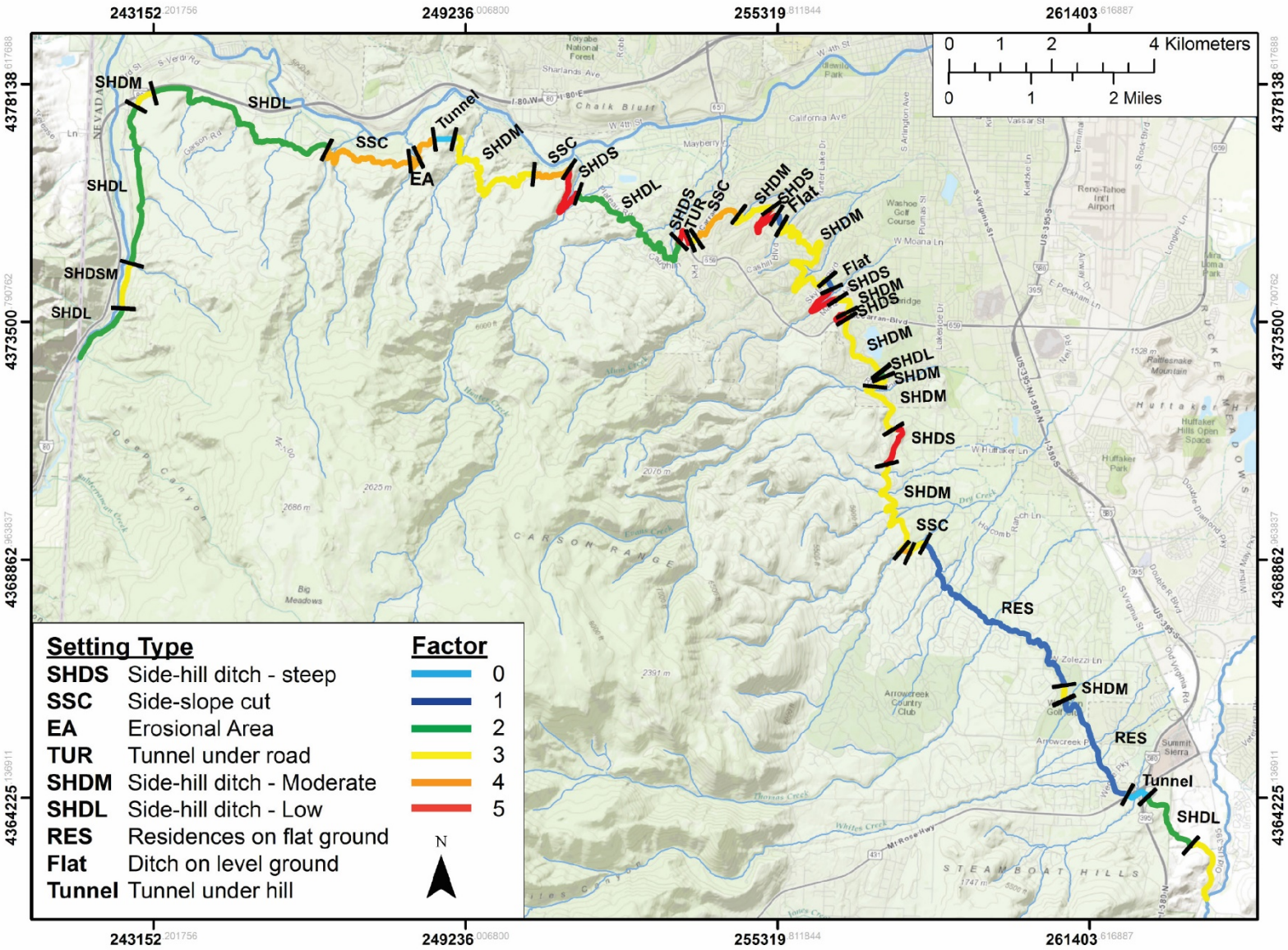

Figure 27. Topographic and runoff settings along the Steamboat irrigation ditch.

Figure 28. Topographic and runoff settings along the Steamboat irrigation ditch.

Figure 29. A concrete lined section of the Steamboat irrigation ditch, where it is built in the side of a steep slope. This lining resists erosion of the embankment and helps in the maintenance of an area where there is a high sediment influx into the ditch.

the water confinement of the tunnel and protection against flooding, this was assigned a 0.

The results for the Steamboat irrigation ditch (figure 30) were that 95% of the ditch was unlined earthen material. The other construction types were relatively minor, but 1.6% was lined with concrete and 1.6% was made up of tunnels under roads.

Another hazard factor that should be considered in other relative risk indexes would be to characterize the condition of the ditch embankment itself. This could reveal weaknesses, such as excessive rodent burrowing, that could lead to flooding, even during regular irrigation flow.

POTENTIAL FLOODING CONSEQUENCES, FACTORS, AND A CONSEQUENCE INDEX

The two consequence factors considered were downstream building type or land usage and the distance from the irrigation ditch to that feature. These features and distances could be directly observed on Google Earth. An important caveat to consider with consequences is that they change with time, and these observations were made in 2014.

Downstream Building Type or Land Usage

Six categories of building type or land usage were used to describe potential consequences: residential, commercial, industrial, transportation, recreational, and agricultural. Residential structures can have continuous occupation and flooding can be a life safety threat as well as cause large financial losses to homeowners. Thus, residences are considered to have the highest consequences to flooding. Commercial buildings are generally occupied during the day (business hours) and have more random occupation after hours. Potential economic losses to commercial buildings include direct losses (business stock and building infrastructure) and indirect losses (business interruption). Industrial buildings and facilities are similar to commercial in that they are principally occupied during working hours, but with generally fewer workers and customers, although these facilities may operate 24 hours a day. Industrial buildings can have significant economic losses because of the potentially expensive equipment and materials they contain. Transportation features are used for the movement of people and material, especially during a disaster, and their loss is a complicating factor during a flooding event. In a city, roads can be the main conduit for flooding and economic losses related to roadways are common with nearly all urban floods. Recreational features include parks and golf courses. These features can have moderate economic impact from flooding, particularly when their infrastructure is damaged. Although agricultural fields can actually benefit from some flooding occurring at the right time of the year, there are still potential economic losses of equipment, crops, and livestock with flooding at inopportune times and/or places. The building types and land uses were ranked in approximate order of their vulnerability or relative loss potential on a scale of 1 through 5, with 5 having the largest potential consequences (table 5).

Table 5. Building Types or Land Use Downslope from Irrigation Ditch

Factor	Building Type of Land Use
5	Residential building
4	Commercial building
3	Industrial building or facility
3	Transportation feature
2	Recreational area
1	Agricultural land
0	River or Tunnel under hill

The consequences downslope from the Steamboat irrigation ditch are shown in figure 31. Residences in

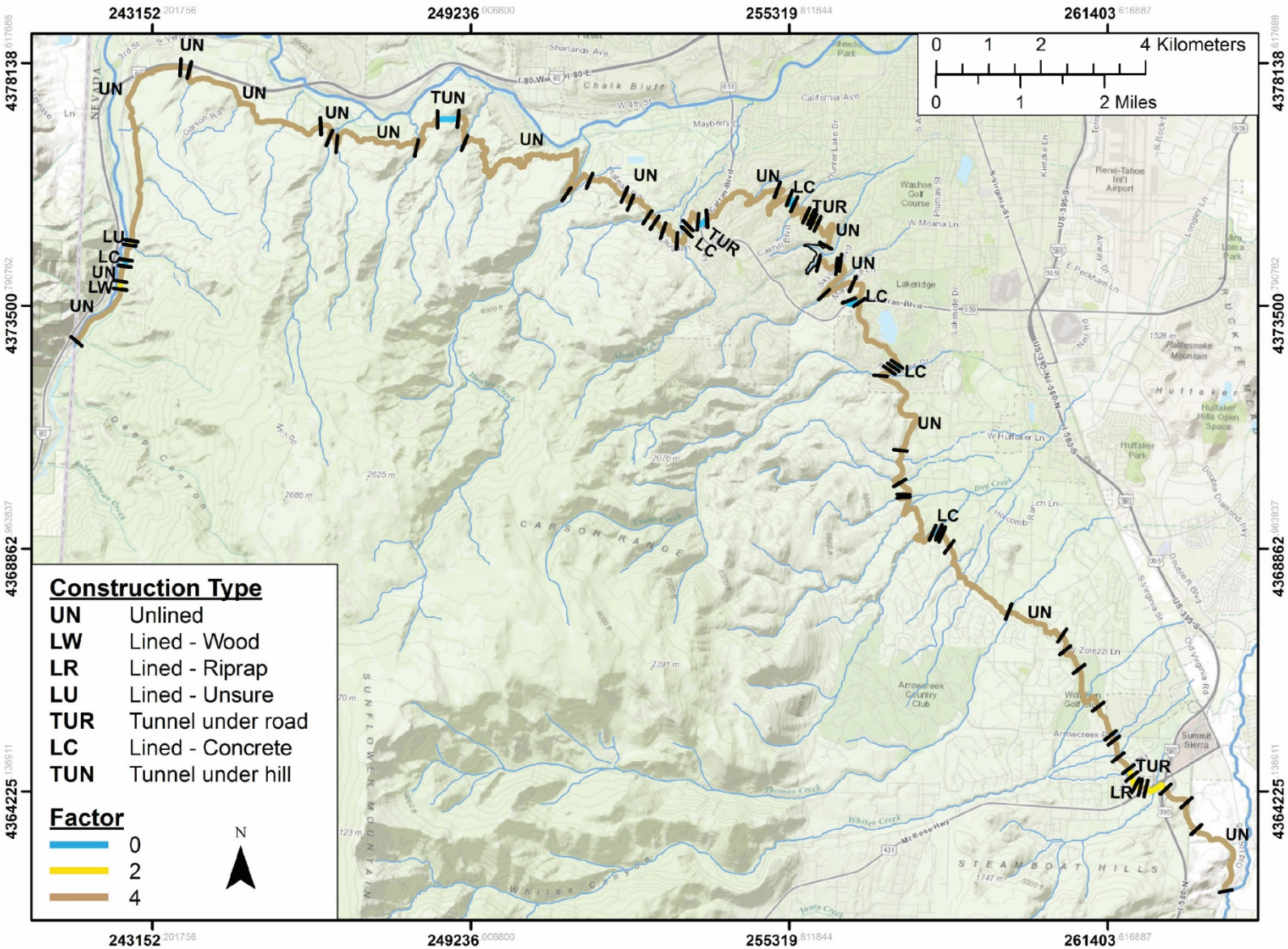

Figure 30. Construction type along the Steamboat irrigation ditch. Most of the ditch is made up of unlined earthen materials.

varying densities are downslope over about 51% of the ditch and in some cases the ditch is surrounded by residences in suburban settings. Other categories generally had small contributions. No commercial buildings were noted and the only industrial consequence was the geothermal plant at the southern end of the ditch.

Downslope Distance from Ditch

The downslope distance to features that could be impacted by flooding is a factor in the likelihood that flooding will occur at a given location. If a building is adjacent to a ditch it is more likely to be impacted by damaging flooding than a building that is significantly farther away, where flooding can be channelized or spread out lower the depth. Four distance categories were used: locations that are adjacent to the irrigation ditch (1–10 m; 3–32 ft), locations within a near flood area (10–100 m; 32–320 ft), locations within a far flood area (100–1000 m; 320–3200 ft), and locations that are beyond the general flood area with exception of circumstances where flooding is concentrated (>1000 m; >3200 ft). These four distance categories are ranked in order from 0 to 3, with 3 being the closest to the ditch category (table 6). Although these are small values for order of magnitude differences between categories, they will be multiplied in a consequence index, which gives them more weight. In this scheme, the 0 value for features that are >1000 m (>3200 ft) from the ditch eliminates the consequence from the relative risk index. Although not done here, there could be exceptions to the >1000 m (>3200 ft) category having a 0 value, such as if there are areas that have been historically flooded from a ditch or if hydrological modeling indicates an area of flooding potential.

The distances downslope from the Steamboat irrigation ditch to the nearest consequence are shown in figure 31 and consequence index in figure 32. Fifty-one percent of the ditch had a downslope feature within 10 to 100 m (32–320 ft) and 28% of the ditch had a feature within 100 to 1000 m (320–3200 ft). Seventeen percent of the ditch length had features beyond 1000 m (3200 ft) and about 4% of the ditch had features close to it, within 10 m (32 ft).

Table 6. Distance Downslope from the Irrigation Ditch

Factor	Distance
3	1–10m (3–32 ft)
2	10–100m (32–320 ft)
1	100–1000m (320–3200 ft)
0	>1000m (> 3200 ft)

Consequence Index

An overall consequence index was developed by multiplying the two consequence factors together (table 7). This seemed to work well at convolving the nearness of the different levels of consequence so both factors could be compared at once. As mentioned, distances that are greater than 1000 m (3200 ft) are considered generally inconsequential, and the 0 value removes the consequence from the relative risk index.

Table 7. Consequence Index

Index Value	Consequence Type	Downslope Distance
15	Residential	1–10 m (3–32 ft)
10	Residential	10–100 m (32–320 ft)
5	Residential	100–1000 m (320–3200 ft)
0	Residential	>1000 m (> 3200 ft)
12	Commercial	1–10 m (3–32 ft)
8	Commercial	10–100 m (32–320 ft)
4	Commercial	100–1000 m (320–3200 ft)
0	Commercial	>1000 m (> 3200 ft)
9	Industrial	1–10 m (3–32 ft)
6	Industrial	10–100 m (32–320 ft)
3	Industrial	100–1000 m (320–3200 ft)
0	Industrial	>1000 m (> 3200 ft)
9	Transportation	1–10 m (3–32 ft)
6	Transportation	10–100 m (32–320 ft)
3	Transportation	100–1000 m (320–3200 ft)
0	Transportation	>1000 m (> 3200 ft)
6	Recreational	1–10 m (3–32 ft)
4	Recreational	10–100 m (32–320 ft)
2	Recreational	100–1000 m (320–3200 ft)
0	Recreational	>1000 m (> 3200 ft)
3	Agricultural	1–10 m (3–32 ft)
2	Agricultural	10–100 m (32–320 ft)
1	Agricultural	100–1000 m (320–3200 ft)
0	Agricultural	>1000 m (> 3200 ft)

RELATIVE RISK INDEX

A relative risk index was developed using the hazard and consequence observations and was calculated using a GIS program. The ditch setting index, ditch construction type index, and downslope consequence index were added together to get the final relative risk index. This produced the smallest range of values for the final index, making it easier to use. Relative risk calculations were made using ArcMap and were done for every 10 m (32 ft) along the ditch. The results of this relative risk index are shown in figures 33a, 33b, and 33c.

Three general relative risk categories are proposed: relatively high, intermediate, and relatively low (colored yellow, green, and blue, respectively on figures 33a, 33b, and 33c). The highest risk index values range from 17 to 24, the intermediate values range from 11 to 16, and the lower risk

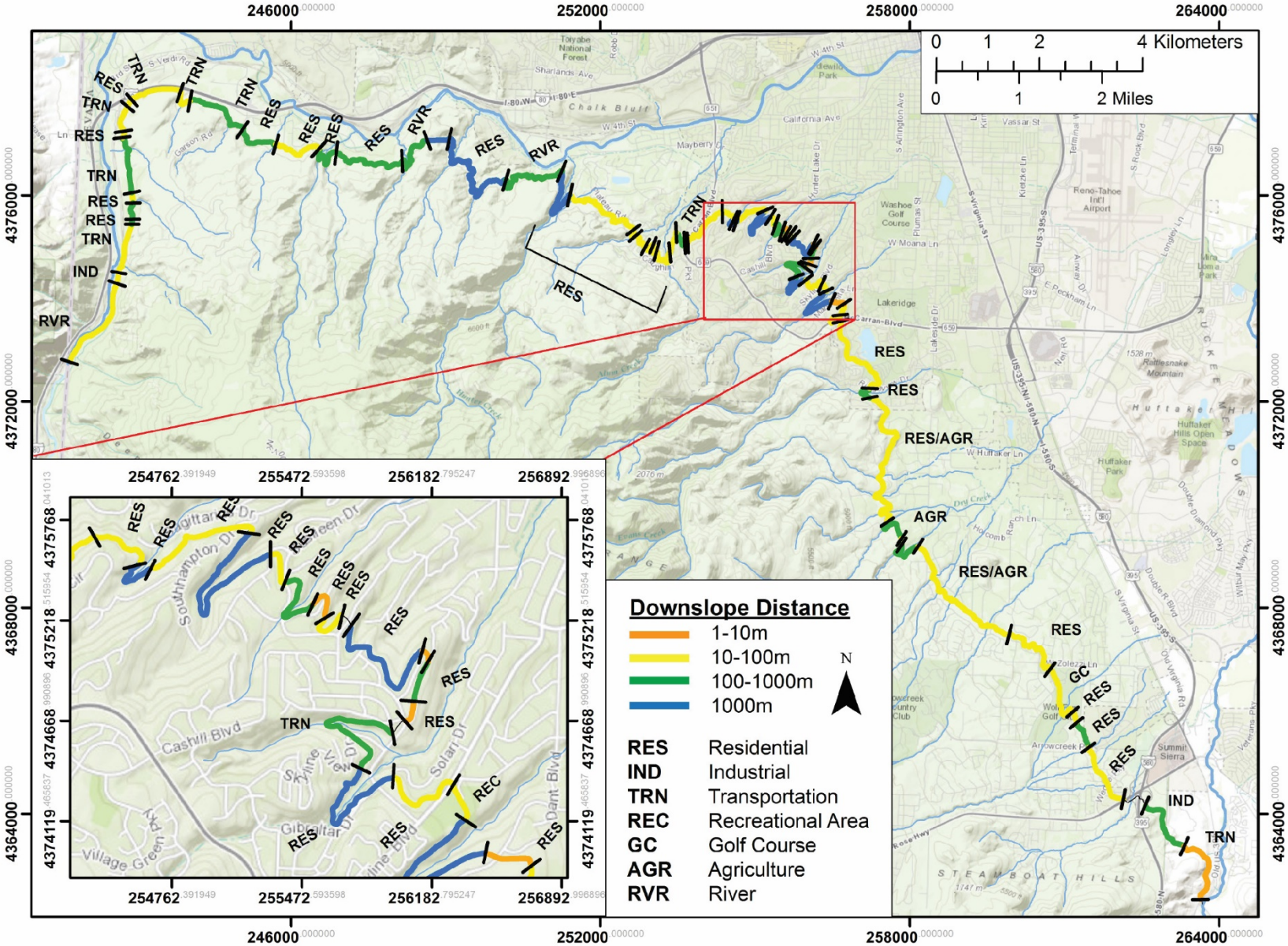

Figure 31. Downslope features and the distance downslope from the irrigation ditch of the nearest feature.

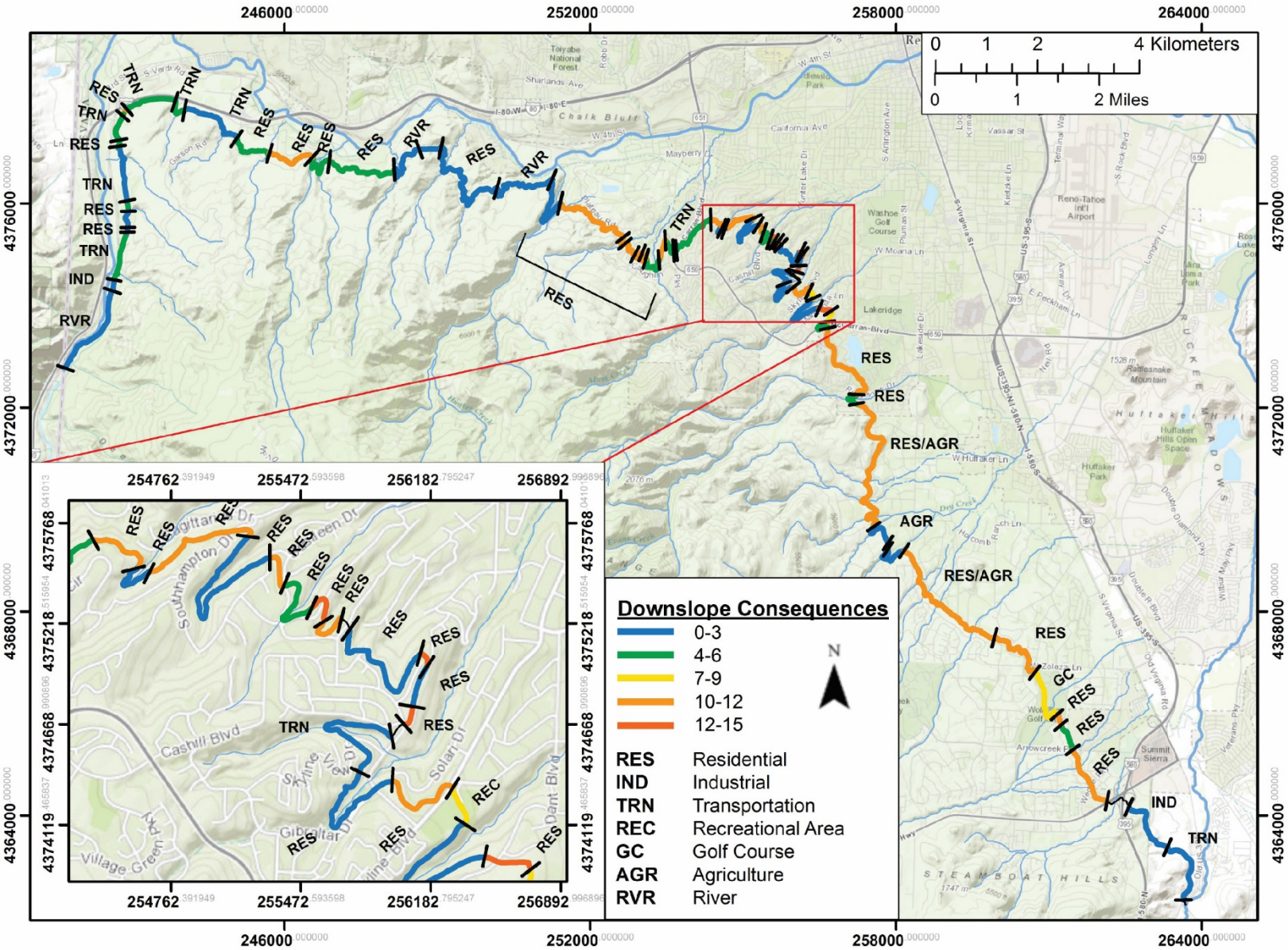

Figure 32. Consequence index for the Steamboat irrigation ditch. Higher values indicate more potential consequences of flooding. The ditch is generally located on the western side of Reno and is bounded by residences in many places.

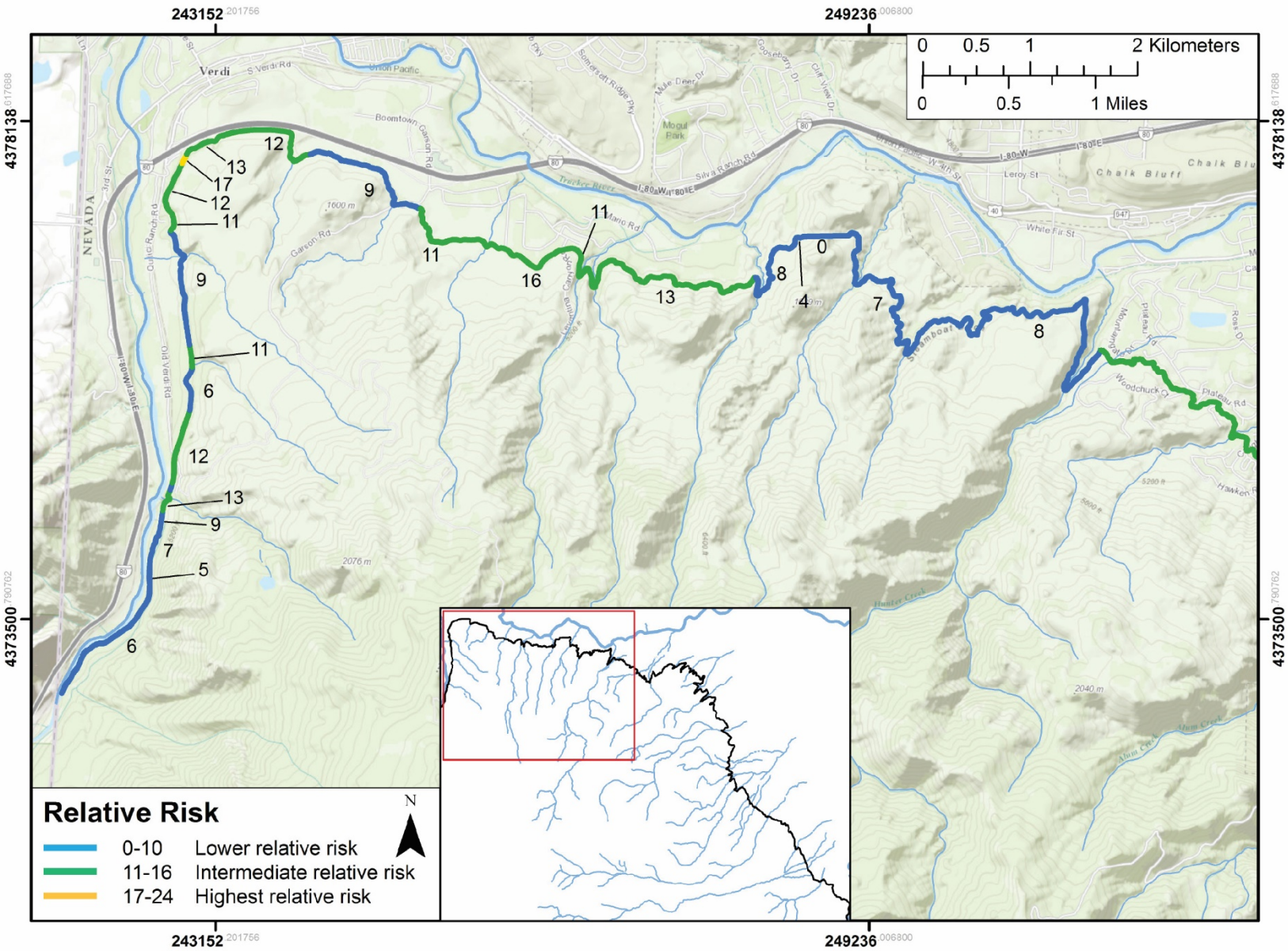

Figure 33a. Relative flood risk index for the western third of the Steamboat irrigation ditch.

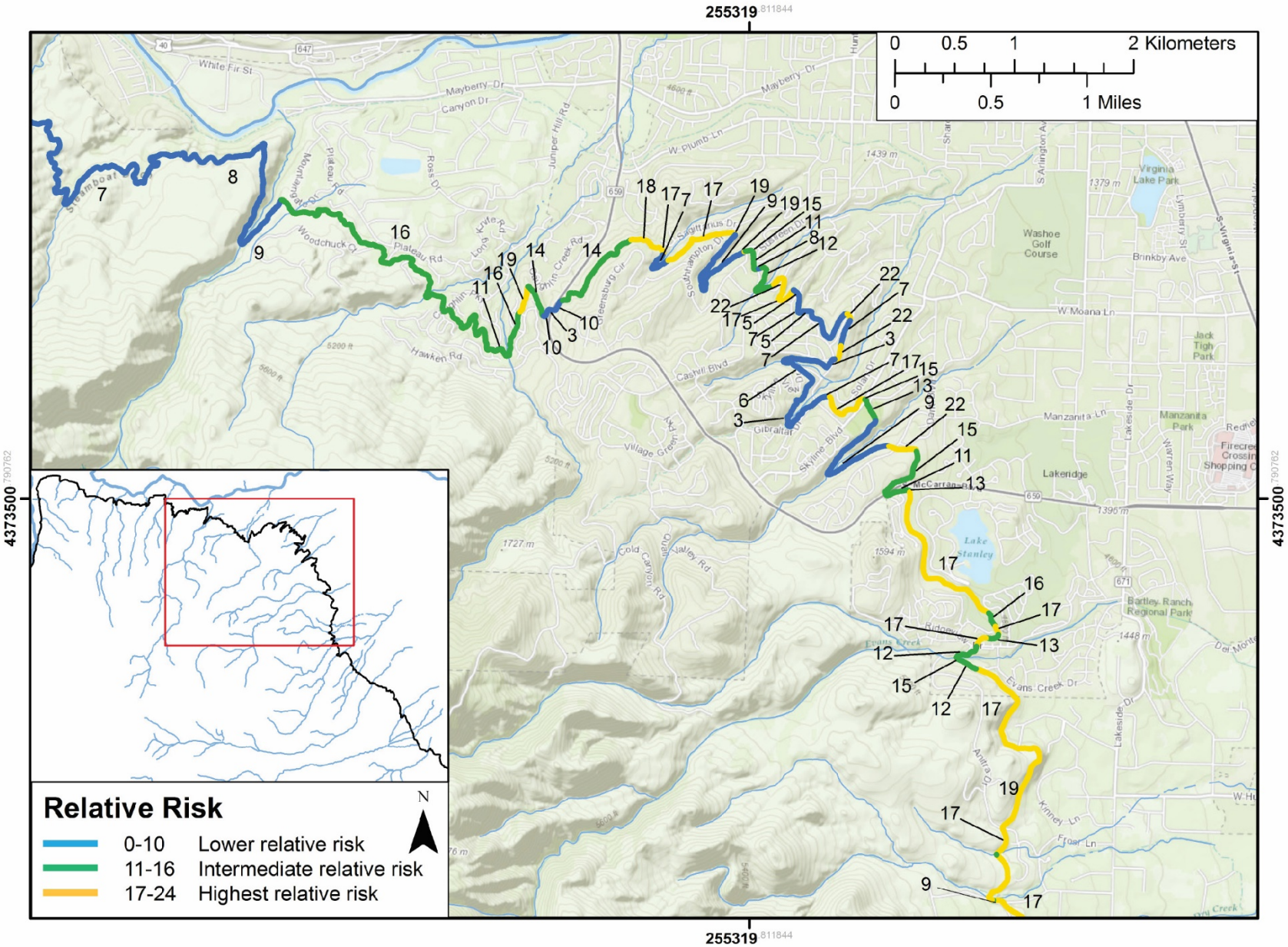

Figure 33b. Relative flood risk index for the central third of the Steamboat irrigation ditch. This section covers the section of the ditch that is within the City of Reno and has some of the highest index values.

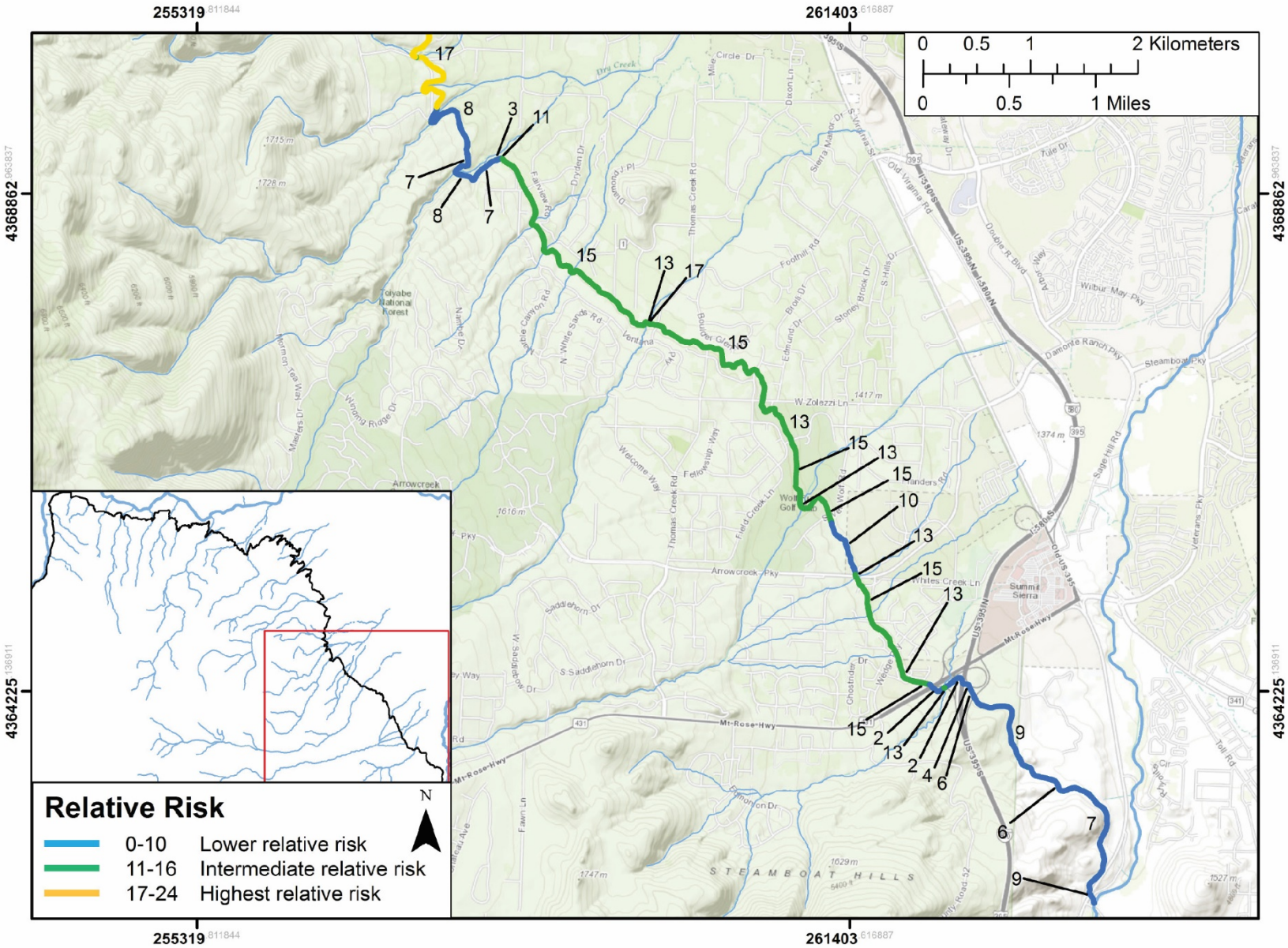

Figure 33c. Relative flood risk index for the southern third of the Steamboat irrigation ditch.

values range from 0 to 10. The specific values are indicated on figures 33a, 33b, and 33c, but this is mostly for research purposes to illustrate the variability and range of values within these categories. The results of this study indicate that about 15% of the ditch has a relatively high risk index, about 41% of the ditch has an intermediate risk index, and that 44% of the ditch has a relatively low risk index. Thus, the index identified 15% of the total ditch length that could get priority for further research or mitigation.

In this study, the relative risk index has been weighted more strongly by the consequences than the hazard because the consequences were more completely characterized. Hazard considerations can create values as high as 9, whereas consequence considerations can create values as high as 15. This relative weighting is flexible and can be tailored to any particular situation. In an area where consequences are evenly exposed along a ditch, hazard considerations might dominate the relative index.

It is important to note that relative risk index alone does not indicate reaches of the ditch that are dangerous. Irrigation ditches in mountainous settings are commonly located along slopes and when built and maintained well, they can handle storm runoff quite effectively. The historical data reviewed indicate that sediment and debris loads, flow impediments, side-stream inflow, and rodent burrowing of embankments have been the main factors leading to ditch flooding.

HYDROLOGICAL POINTS OF INTEREST

In addition to the relative risk index, locations that are hydrological points of interest were compiled and are shown in figures 34a, 34b, and 34c. Hydrological points of interest include drainage intersections with the ditch, tunnels under roadway crossings, and spillways that release flow from the ditch. Some of these locations have been problematic during storms and have acted as trigger points for flooding, such as the blockage of a tunnel entrance with debris. There are other hydrological points of interest that were not considered here, such as storm water discharge into the ditch, vegetation threats (e.g., a tree that is being undercut by the ditch and threatens to fall in), and ditch complexity or sinuosity. These were not compiled here mostly because they involve field surveys to discover and characterize and this study was mostly limited to in house work, but they are noted as potential considerations for other ditch surveys.

Runoff in drainages has historically entered ditches adding storm water, sediment, and debris to them. In some cases, drainages discharge directly into the ditch and in other cases there are diversion structures to separate the water in the drainage from the water in the ditch. Diversion structures at stream intersections are important mitigation actions that can help avoid flooding. These can get clogged with debris or occasionally fail during storm events, however, so they need to be watched. Another historical problem has been tunnels and culverts under roadways that are not designed for storm water flow and become choked with debris and sediment. This can cause water to back up in a ditch and potentially overtop its embankment. In some cases, the blockage at specific crossings has been a repeated occurrence and these are naturally the focus of future mitigation and storm maintenance.

The most important parameter in considering whether runoff from an intersecting drainage has a significant potential to impact ditch flooding is the amount of storm water discharge it can produce. Storm water discharge potential for drainages above the Steamboat irrigation ditch was estimated by Pohll and Carroll (2013; Appendix A). These results have been divided into three categories of ditch-drainage intersections that are shown in figures 9a, 9b, and 9c. Major drainage intersections are shown as red triangles, substantial drainage intersections are orange triangles, and other drainage intersections are yellow triangles. Major drainages have estimated potential discharges from a 24-hour, 100-year-storm event that are a few to several times the capacity of the Steamboat irrigation ditch and have the potential to quickly overwhelm the ditch, cause it to be overtopped, and flood (Pohll and Carroll, 2013; Appendix A). Pohll and Carroll (2013) estimated the general capacity of Steamboat irrigation ditch and determined that drainage discharges into the ditch of over 131 cfs would exceed the ditch capacity and make flooding imminent (especially considering that the ditch will likely be partially full with storm water from other sources). Most of the major drainage crossings of the ditch are diversion structures that separate the stream and ditch flows, but as mentioned these can be overwhelmed or fail during major storm events if they are not robustly engineered and maintained. Several other smaller drainages are capable of equaling or exceeding ditch flow by a few times its capacity (Pohll and Carroll, 2013; Appendix A). These are considered substantial drainage intersections (orange triangles), and are watersheds that Pohll and Carroll (2013; their table 12) have determined could discharge over 100 cfs during the 24-hour 100-year-storm event. This amount roughly matches the capacity of the Steamboat irrigation ditch considering a slightly diminished capacity due to sediment and vegetation. If significant storm water discharge enters a ditch, overtopping and flooding is much more likely. The rest of the drainage intersections are indicated by yellow triangles. These drainages are two types, 1) drainages that Pohll and Carroll (2013) estimated discharges to be less than 100 cfs, or 2) drainages from watersheds in the western part of the ditch that were not studied by Pohll and Carroll (2013). The western part of the ditch has a lower relative risk profile than the more urbanized parts of the ditch (figures 8a and 8b) and there are spillways for some of the larger drainages (figure 9a), so the western drainages were noted on figure 8a, but the storm water discharge was not estimated. It is noted, however, that some of these western drainages may have substantial discharge potential. Even though the runoff potential of a particular drainage does not exceed the estimated capacity of a ditch, it can contribute to the existing flow within the ditch such

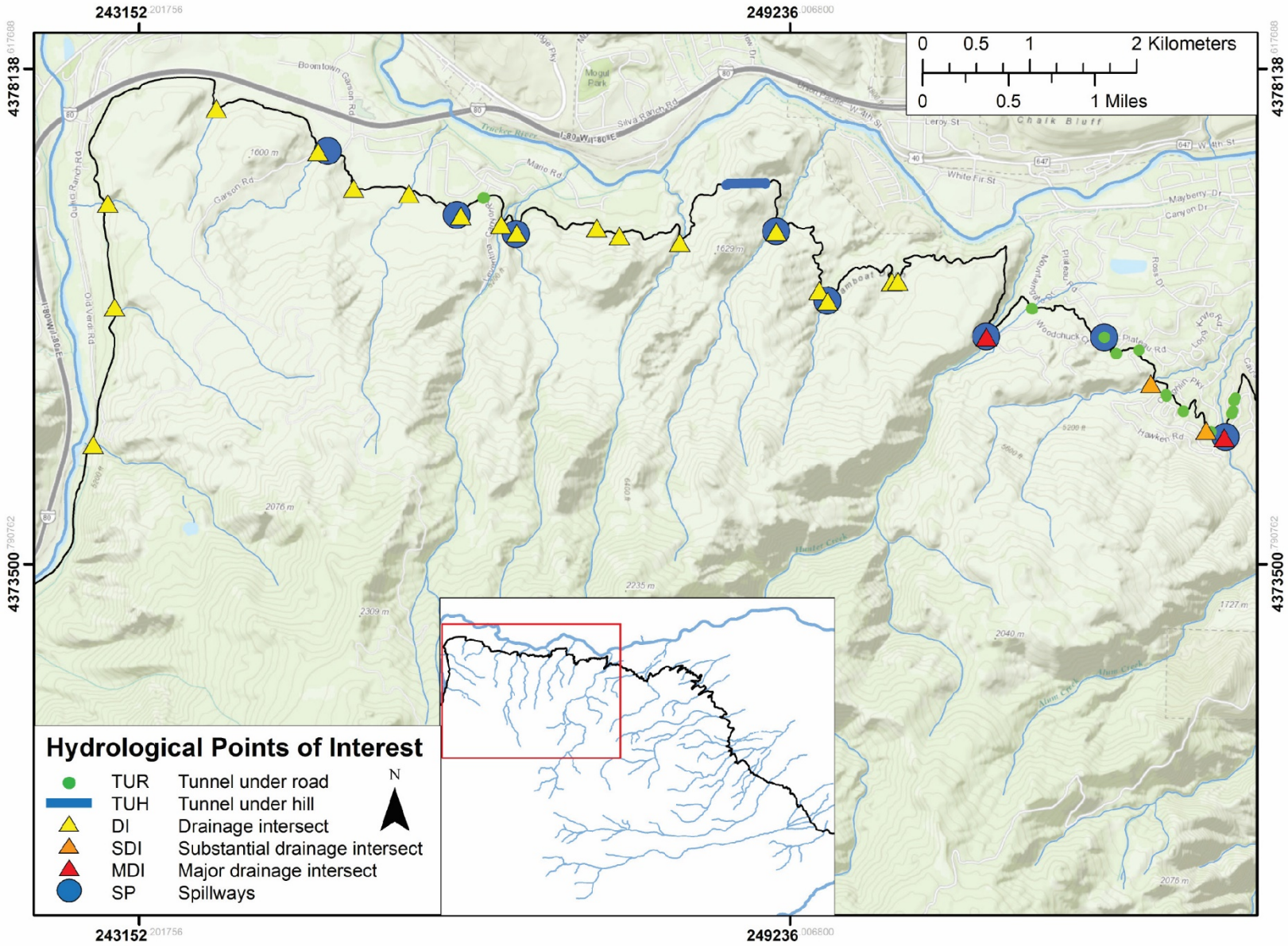

Figure 34a. Hydrological points of interest for the western third of the Steamboat irrigation ditch. Three types of drainage intersections, spillways in the ditch, and tunnels under roads (road crossings) and through a hill are shown.

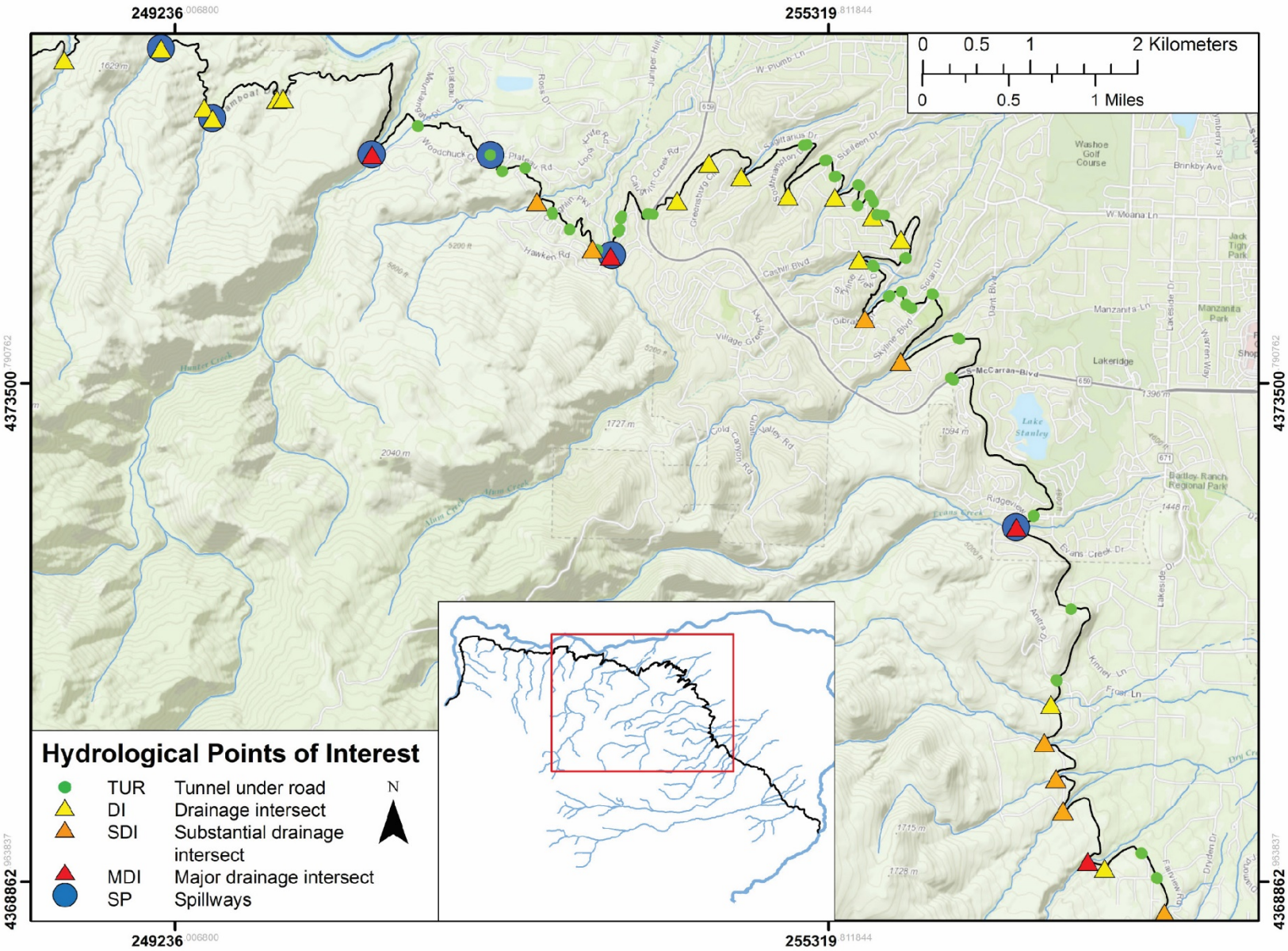

Figure 34b. Hydrological points of interest for the central third of the Steamboat irrigation ditch.

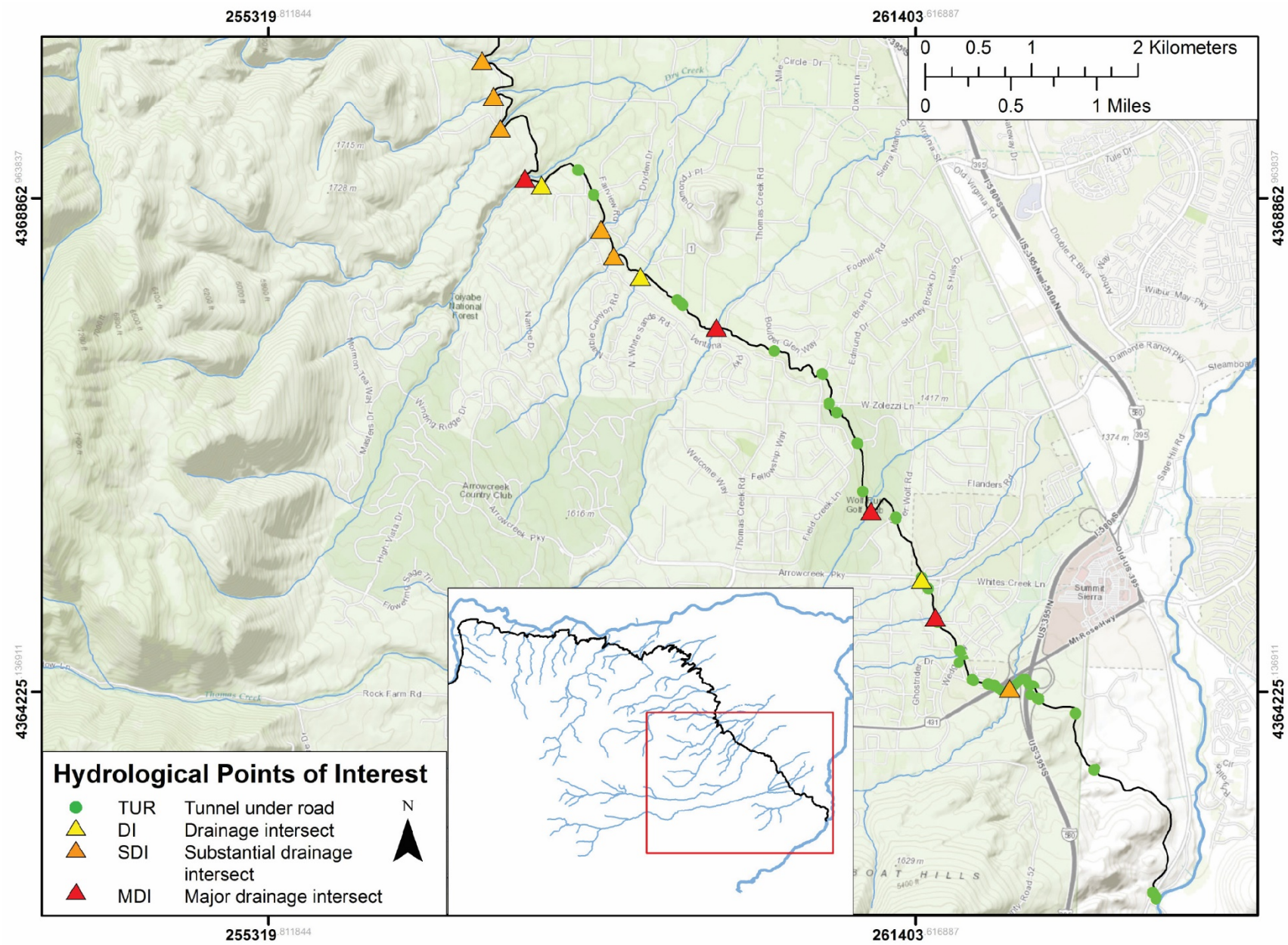

Figure 34c. Hydrological points of interest for the southern third of the Steamboat irrigation ditch.

that the ditch capacity is exceeded. Pohll and Carroll (2013; their table 12) give peak discharge estimates for the 24-hr, 100-year storm for the drainages they studied and these can be used to explore combinations of storm water contributions.

The primary mitigation of drainage discharge entering a ditch is having a robust diversion structure that can carry a 100-year storm over or under the ditch. Additionally an open spillway or sewer intake in the ditch downstream of the drainage can help relieve storm water flow within the ditch. Another mitigation tool is placing flood gates or flash boards immediately downstream of significant drainage intersections or spillways to close or limit the ditch flow at that point. Both of these techniques are used in several places along the Steamboat irrigation ditch.

Existing spillways in the ditch are shown by the blue circles on figures 9a, 9b, and 9c. Several spillways have been strategically placed to allow water that enters the ditch from side drainages to exit the ditch. The spillways are closed with flash boards during the summer irrigation season and are open during the winter months. Thus, the spillways can help relieve the ditch of storm water in winter storms, but would not likely be available for summer storms unless an action was taken to stop irrigation flow and remove the flash boards.

The Steamboat irrigation ditch flows through tunnels and pipes where it is crossed by roads; these are shown in figures 34a, 34b, and 34c. Historically, debris and sediment has become lodged at the entrances of some of these tunnels during storms, blocking the ditch, causing a backup of water, and causing an overtopping of the embankment. Many of these tunnels have low profiles, especially in suburban settings, and are designed primarily to pass irrigation flow. The lowest tunnels tend to be wider, making them more susceptible to annual sedimentation, which in turn limits the flow volume through the tunnel. Low tunnels and tunnels that have significant amounts of sediment in them have less freeboard to pass a piece of floating debris during storm water flow, increasing the chances of blockage. Once some debris becomes lodged at a tunnel entrance it creates a cascading effect of trapping additional debris and sediment. In some cases, the capacity of the tunnels themselves can be the limiting factor to passing storm water. For problematic tunnels that have repeatedly been blocked with debris during storms, an aggressive strategy for mitigating the capacity and freeboard should be considered. The locations of road crossings are sites that should be regularly maintained, monitored during storm events, and aggressively cleaned out as debris becomes lodged during storms to minimize flood potential. Over time, the tunnel capacities of road crossings can be increased when road modifications occur.

Figure 35. Steamboat ditch where it passes under McCarran Boulevard in tunnel (downstream end). The tunnel is relatively free of sediment on the bottom, with exception of a few small boulders at the exit. The waterline from irrigation flow levels is visible just below the paint. There is a large amount of freeboard above the irrigation water line to allow storm water and debris to flow through the tunnel. On the other side of the tunnel (out of sight) is a debris-catching rack to further reduce the chances of debris blockage in the tunnel.

Two examples of tunnels under roads are given for illustration: the McCarran Boulevard crossing and the Meridian Lane crossing. The tunnel where Steamboat irrigation ditch passes under four-lane McCarran Boulevard is an example that has a lower flood hazard potential because it is relatively large, the adjacent ditch is lined with concrete, and there is a debris catcher on the upstream end (figures 10 and 11). The tunnel has a height of 60 inches (152 cm) with a width of 124 inches (315 cm). Adjacent parts of the ditch are concrete lined, reducing the potential for erosion and embankment failure and making the ditch easier to clean in the vicinity of the tunnel.

The tunnel where Meridian Lane crosses the Steamboat irrigation ditch has a lower profile and capacity that the McCarran Boulevard tunnel (figures 12, 13, and 14). The

Figure 36. Cross section of the tunnel under McCarran Boulevard. Irrigation flow levels are indicated with the blue horizontal lines.

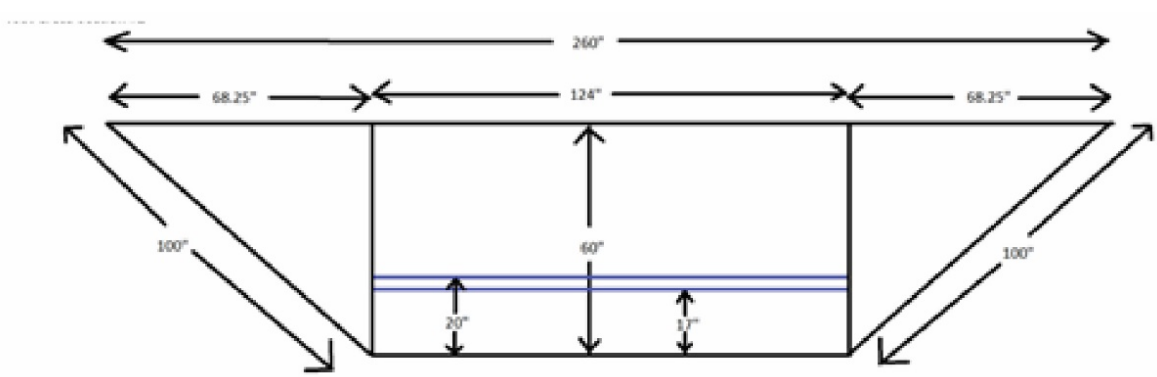

Figure 37. Tunnel under Meridian Lane. The bottom of the tunnel is covered with sediment that further constrains the water capacity of an already low tunnel. This was the site of a sediment dam in the 2005 flood event.

Meridian Lane tunnel has repeatedly been clogged with debris, notably during the floods in the winter of 2005 (figure 39), which caused a backup in the ditch and an overtopping of the ditch embankment (figure 40).

The Meridian Lane tunnel has a designed height of 40 inches (102 cm) and a width of 134 inches (340 cm). Lower tunnels are more difficult to clean and in the case of the Meridian an estimated 8 inches (20 cm) of sediment was deposited on the bottom of the tunnel when it was visited in the spring of 2014 (figure 37).

The McCarran Boulevard tunnel is much better suited for handling above-average levels of water and floating debris than the Meridian Lane tunnel, which appears to be designed to handle principally irrigation water with little external storm water entering the system. Compare the freeboard over the water marks from irrigation flow in figures 35 and 38. Additionally there is a sewer discharge within the Meridian

Figure 38. Inside of the Meridian Lane tunnel. Note the relatively high water line from irrigation flow. A layer of sediment has built up constraining the flow of water through the tunnel and freeboard above the water to pass floating debris. Within the tunnel is a sewer discharge as well.

Figure 39. Debris and sediment blocking the entrance of the Meridian Lane tunnel in the winter of 2005 *(photograph courtesy of the City of Reno)*.

Lane tunnel that can further add water volume and disrupt the water flow (figure 38). The cross sectional area of these tunnels illustrates differences and impacts in their flow capacity. The design cross section of the Meridian Lane tunnel was 5,360 square inches (34,581 sq. cm). However, with 8 inches (20 cm) of sediment in the bottom, the vertical dimension is reduced from 40 to 32 inches (102 to 81 cm), and the corresponding area left to convey water is 4,288 square inches (27,664 sq. cm), a 20% decrease. The channel upstream from the Meridian Lane tunnel was measured (figure 42) and has a cross sectional area of 8,085 square inches (52,161 sq. cm). Thus, the design cross section of the tunnel is about 66% of the bank-full ditch cross section. Eight inches (20 cm) of sediment reduces this cross sectional area of the tunnel to be 53% of the bank-full ditch on the upstream side. The McCarran Boulevard tunnel has a cross sectional area of 7,440 square inches (48,000 sq. cm), about 39% larger than the design cross section of the Meridian Lane tunnel and consequently a better chance of passing storm water.

In the development of these hydrological points of interest, fairly sophisticated hydrological information was considered, in this case, the study by Pohll and Carroll (2013). Such information may be available for other areas in the form of FEMA flood studies and the like, or it will have to be developed.

CONCLUSIONS

A relative risk index for considering the flooding potential of irrigation ditches was developed using mostly remote techniques to help determine locations along a ditch that could be focused on for further study and for potential mitigation. We assessed the construction of the ditch and its setting, and convolved these with the local consequences to gain a sense of the relative risk. About 15% of the ditch was characterized as having a relatively high risk potential for flood damage. It is recommended that those sections receive the first attention to flood issues. In addition, major and substantial drainages should be separated from the irrigation ditch with robust diversion structures and ditch flood gates

and spillways should be considered downstream of such intersections to relieve the ditch of storm water. This relative risk approach can be tailored and applied to any irrigation ditch in Nevada. In addition to limiting the extent of initial focus along a ditch, the effort involved in developing the index forces a careful examination of the full extent of a ditch system. This can lead to further insights into flood potential.

ACKNOWLEDGMENTS

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Figure 40. Water backed up on the upstream side of the ditch and overtopping the embankment caused of the blockage at the Meridian Lane tunnel in 2005 *(Photograph courtesy of the City of Reno).*

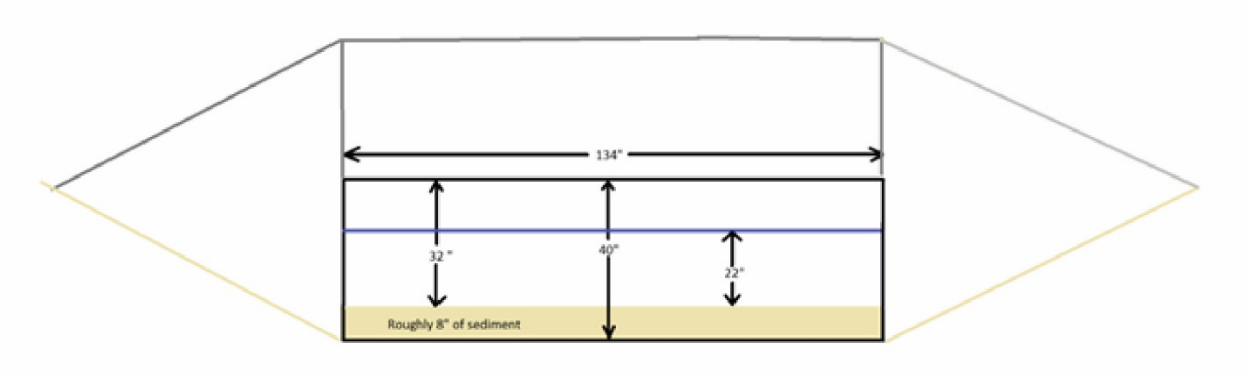

Figure 41. Cross section of the tunnel under Meridian Lane. Irrigation water level is shown by the blue line and approximate sediment build up in tunnel in the spring of 2014 is shown with brown shading at the bottom.

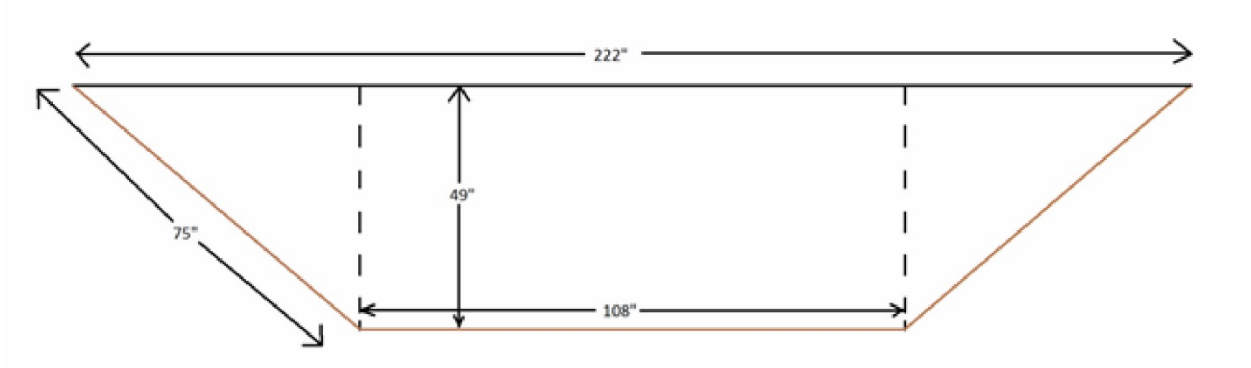

Figure 42. Cross section of the Steamboat irrigation ditch just upstream from the Meridian Lane tunnel.

Science Environment Solutions

Appendix A
Assessment of Flood Risk for the Steamboat Ditch

Prepared by: Desert Research Institute

Investigators: Greg Pohll and Rosemary Carroll

Presented to: Nevada Bureau of Mines and Geology

December 30, 2013

EXECUTIVE SUMMARY

Steamboat Ditch's primary purpose is to convey irrigation water from the Truckee River to farms, golf courses and irrigation ponds south of Reno. Steamboat Ditch has also been used to mitigate Truckee River flooding during large runoff events as well as collect and convey stormwater originating at several major, and numerous minor, drainage basins uphill of the ditch. Occasionally, stormwater has overtopped the ditch causing damage to both the ditch infrastructure and to populated areas located in its vicinity.

The principal goal of this study was to develop hydrologic flood frequency analysis within each watershed traversed by Steamboat ditch. Rainfall-runoff simulations were conducted using the U.S. Army Corps of Engineers HEC-HMS (USACE, 2000a) for the six most significant watersheds as determined by flow direction and accumulation. These six basins include Hunter Creek, Alum Creek, Evans Creek, Dry Creek, Thomas Creek and Whites Creek. A full discussion is provided on model construction and calibration along with a comparison of peak discharges for the 24-hour event given 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 200-year, 500-year and 1.000-year return intervals. The 100-year event was used to assess those sub-watersheds most likely to contribute significantly to peak discharge.

Model results suggest that all major drainages traversing Steamboat Ditch have the potential to contribute significant discharge during even modest storms to cause flooding along Steamboat Ditch. Large discharges are the result of poorly draining soils that occupy much of the region surrounding Streamboat Ditch. During the 100-year event, estimated peak discharge from major drainage basins exceeds ditch capacity by 3.9 to 11.6 times. Simulated discharges are largest for Hunter Creek (1,160 cfs) and lowest for Evans Creek (390 cfs). Dry Creek is modeled as most responsive, or flashy, while Whites Creek has the longest lag-time to peak discharge. Flooding is expected to be exacerbated during rain-on-snow events but is extremely sensitive to air temperature distribution across the study site. Of the six basins modeled, Thomas Creek has the greatest potential to see a significant rise in peak discharge as a result of snowmelt.

A regression model was developed relating significant model parameters to the 100-year, 24-hour peak discharge. The regression model was used to estimate the 100-year, 24-hour peak discharge for other smaller drainages adjacent to the Steamboat Ditch. An exponential function was the best predictor (r^2=0.92) using independent variables of curve number, basin area and total storm precipitation depth. The regression model estimated that 10 out of 29 the smaller, unnamed basins investigated are capable of contributing 100-year flows in excess of ditch capacity. Two of these are located in the Caughlin Ranch sub-division, two in the vicinity of Skyline Blvd, three are located north of Dry Creek, two in the Arrow Creek sub-division, and one defining the Steamboat Hills.

Recommendations include prioritizing areas where drainage needs to be diverted away from the ditch based on localized ditch conditions, ability to diffuse peak flows through outflow drains and possible damage if Steamboat Ditch fails. It is not recommended that Streamboat Ditch be used as a stormwater management tool for the Truckee River in the winter given the potential for large rain events and possible runoff above the ditch. Lastly, engineering solutions to improve ditch conveyance and/or improve stream crossings may need consideration.

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1.0 INTRODUCTION

Within the Truckee Meadows six irrigation ditches (Steamboat, Last Chance, Lake, Cochrane, Highland, and Orr) are used primarily to provide irrigation water for agriculture, golf courses and urban ponds. It is widely known that these irrigation ditches collect, convey, and occasionally overflow with stormwater. At least three hydrologic studies have been conducted to map out the general characteristics for each of the canal systems and identify potential flooding risks associated with excessive storm runoff into these canals (Manhard Consulting, 2010; Gray and Associates, 2003; Kennedy/Jenks Consultants, 2004). Two of the studies (Manhard Consulting, 2010; Gray and Associates, 2003) recommended that public ditch ownership and maintenance should be contemplated due to the common public benefits and public safety concerns surrounding the ditches. It was also suggested that a detailed hydrologic study be conducted prior to any negotiations over public ownership and maintenance.

Manhard Consulting (2010) provided specific recommendations regarding the types of analysis that should be conducted:

- Develop a hydrologic flood frequency analysis within each watersheds traversed by a drainage ditch including all upstream flows, downstream discharges and facility impacts,
- Dynamic ditch conveyance considering inflows, outflows, channel bank overflows, crossing structures, and closed conduit sections
- Extensive ditch condition assessments including the stability, level of repair, vegetative management, sedimentation, and access opportunities of the channel banks/bottoms, crossing structures, and closed conduits sections
- Full title searches should include parcel boundary, ownership, easement, and prescriptive right considerations for the ditches and any parallel access roads

This study focus only on the highest priority items, with analysis limited solely to Steamboat Ditch because it is known to intercept and convey stormwater runoff, is the longest of the six ditches, resides in both Washoe County and the City of Reno, is the most elevated and therefore is likely to collect large amounts of surface runoff, and it traverses at least four public facilities including Caughlin Ranch, Horsemens Park, Lake Ridge Golf Course and Wolf Run Golf Course.

As part of the listed goals of this project, this portion covers rainfall-runoff simulations conducted using the U.S. Army Corps of Engineers' (USACE) HEC-HMS (USACE, 2000a) for the six most significant watersheds that provide runoff directly to the ditch via stream crossings. Model construction and calibration are provided in detail along with a comparison of peak discharges for the 24-hour event given 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 200-year, 500-year and 1.000-year return intervals. A regression model is developed relating significant model parameters to peak discharge and used to estimate the 100-year 24-hour event for an additional twenty-nine basins contributing flow to the Steamboat ditch. Historically, rain-on-snow events produce the largest floods in the area, therefore rain-on-snow events are modeled and discussed in the context of

important snowmelt parameters and compared to the 100-year, rain-only event. Lastly, management recommendations for Steamboat Ditch are provided.

2.0 SITE DESCRIPTION AND HISTORICAL CONTEXT

Figure 1 provides an overview of the region under consideration. Steamboat ditch is shown traversing the foothills on the western edge of the Truckee Meadows. The primary purpose of Steamboat Ditch is to convey water from the Truckee River to farms located as far south as Steamboat Hills. In addition, Steamboat Ditch has become a tool for storm water management by diverting water out of the Truckee River to mitigate floods during large precipitation events. The primary question posed is whether using Steamboat Ditch to covey storm runoff is a good strategy based on the runoff potential of watersheds uphill of the ditch.

Six watersheds have been identified using flow direction and accumulation, at the 30 m digital elevation model (DEM) resolution, as having the greatest potential to generate significant discharge into Steamboat Ditch. Furthest north is Hunter Creek. Hunter Creek occupies 11.3 mi^2 with elevations rising from approximately 4,900 ft to more than 9,780 ft. Slopes are fairly steep, averaging 22% at elevations less than 6,500 ft and 33% for elevations greater than 6,500 ft. No development occurs in Hunter Creek and vegetation is dominated by trees containing Lodgepole Pine, Ponderosa Pine, Douglas Fir, Red Fir and Aspen-mixed Conifer forests. Shrubs (Big Sagebursh, Chaparral, Mountain Mahogany, and Alpine Dwarf) occupy nearly one-fifth of the basin while Pinyon-Juniper and grasses occupy less than 10%. Soils are predominantly a gravely loam of the Duckhill-Hirschdal-Fraval association. Lower elevations in the basin contain a mix of colluvium and residuum derived from volcanic rocks (Tristan-Barshaad-Arzo association), sandy-stony loam (Stodick), and very cobbly loam (Fraval-Booford-Jumbo association). The highest elevations in the basin contain very stony to very cobbly loam of the Jorge-Boomtown-Fugawee association and Fraval-Hirschdale-Jumbo association. Soils in the lower basin generally have low water transmitting capabilities but water conductance of soils tends to improve with increased elevation.

Alum Creek is much smaller in area than Hunter Creek occupying only 2.4 mi^2 with elevations reaching 7,950 ft. Nearly 80% of the basin falls below 6,500 ft with slopes averaging 18%, while elevations greater than 6,500 ft contain slightly lower slopes on the order of 13%. Introduced grass occurs in the lower reaches of the basin while Chaparral and Ponderossa Pine exist in the middle reaches of the Basin, Aspen groves and Red Fir appear only in the basin's upper elevations. Soils are similar to those found in the lower portions of Hunter Creek, principally the Duckhill Stony loam and Duckhill-Hirschdale-Fraval association of gravely loam. Soils in Alum Creek have poor infiltration and transmission capabilities.

Evans Creek is about 8.4 mi^2 with elevations extending from Steamboat Ditch up to 9,130 ft. Nearly 40% of the basin area occurs at elevations less than 6,500 ft, while about 50% of the basin occurs at elevations between 6,500 ft and 8,000 ft. Slopes below 8,000 ft average 16% while those above 8,000 ft lessen slightly and average 10%. Residential development only occurs at the very lowest portions of the basin, just uphill from Steamboat Ditch. Big Sagebrush along with isolated stands of Pinyon-Juniper, Douglas

Fir/Ponderosa Pine and Chaparral occupy the lower elevations of the watershed. Ponderosa and Red Fir forests dominate the middle and lower reaches of the basin, respectively. The upper portion of the basin contains mostly the Fraval-Hirschdale-Jumbo association and Fraval-Booford-Jumbo association, or very stony to cobbly loam with slightly better draining capabilities than the soils at lower elevations. The lower reaches of the watershed are a complex array of poorly draining stony, sandy loams, gravelly, sandy loams, cobbly clay, and rock outcrop complexes. .

Dry Creek is on the same order of size as Alum Creek, occupying only 3.2 mi^2. Basin elevations extend from Steamboat Ditch (approx.. 4,800 ft) to 7,820 ft, with more than three-quarters of the basin occurring at elevations less than 6,500 ft. Slopes average 13% with vegetation primarily Big Sagebrush shrubland, though Ponderosa Pine and Chaparral do occur in the upper reaches of basin. Residential development occupies much of the eastern edge of the basin. Soils in Dry Creek are predominantly poorly draining Indian Creek stony sandy loam, Verdico extremely stony sandy loam and Stodick stony loam. Upper reaches of the basin do contain soils in the Fraval-Hirschdale-Jumbo association (very cobbly loam) with slightly improved infiltration capabilities.

Thomas Creek and Whites Creek exhibit a similar shape with drainage patterns producing long, narrow watersheds occupying flat (5% slope) low lying elevations with significant development and extending up to higher, undeveloped elevations with relatively steep terrain (up 27% slope). Whites Creek i, at 10.5 mi^2 and reaching to elevations greater than 10,780 ft, is slightly larger and higher than Thomas Creek at 9.4 mi^2 and a maximum elevation of 10,100 ft. Vegetation in both basins is mostly Big Sagebrush in the lower elevations, Ponderosa Pine the middle portions of the basin, with Douglas/White Fir grading into Red Fir forest at the higher elevations. Subalpine woodland and parkland occurs along the highest ridge. Soils in the lower reaches of these basins have larger infiltration capacities than lower elevations in most of the other watersheds investigated. Soils in the middle narrows and lower watersheds are a mixture of Indian Creek stony sandy loam, Leviathan very stony sandy loam, Notus stony loamy fine sand and Oest extremely stony sandy loam. Soils in the upper elevations is predominantly Toiyabe-Corbett Haypress association (gravelly sand) and Fraval-Hirschdale-Jumbo association (very cobbly loam), both with improved infiltration capabilities. Ridges contain a mix of poorly transmitting soils, mostly Meiss-Sibelia-Rock outcrop, Carioca-Sibelia variant Fungawee asscoiation with poor water transmission.

Only Hunter Creek has observed flow collected at the US Geologic Survey gaging station (10347600). Data was collected from October 1, 1961 to September 30, 1971, October 1, 1977 to September 30, 1981 and October 1, 2002 to present. On average, baseflow conditions of 5 cubic-feet per second (cfs) are obtained by late summer and last through the fall and winter months, observed maximum discharge averages 44 cfs as a result of snowmelt (late March to early July). In contrast, the largest recorded floods have occurred during the months of December and January as a result of rain-on-snow. In particular, the largest recorded discharge occurred January 31, 1963 at 986 cfs. To compare, the next largest events recorded happened on December 31, 2005 at 162 cfs and December 22, 1964 at 117 cfs. Unfortunately, the January 3, 1997-flood was not recorded at Hunter Creek.

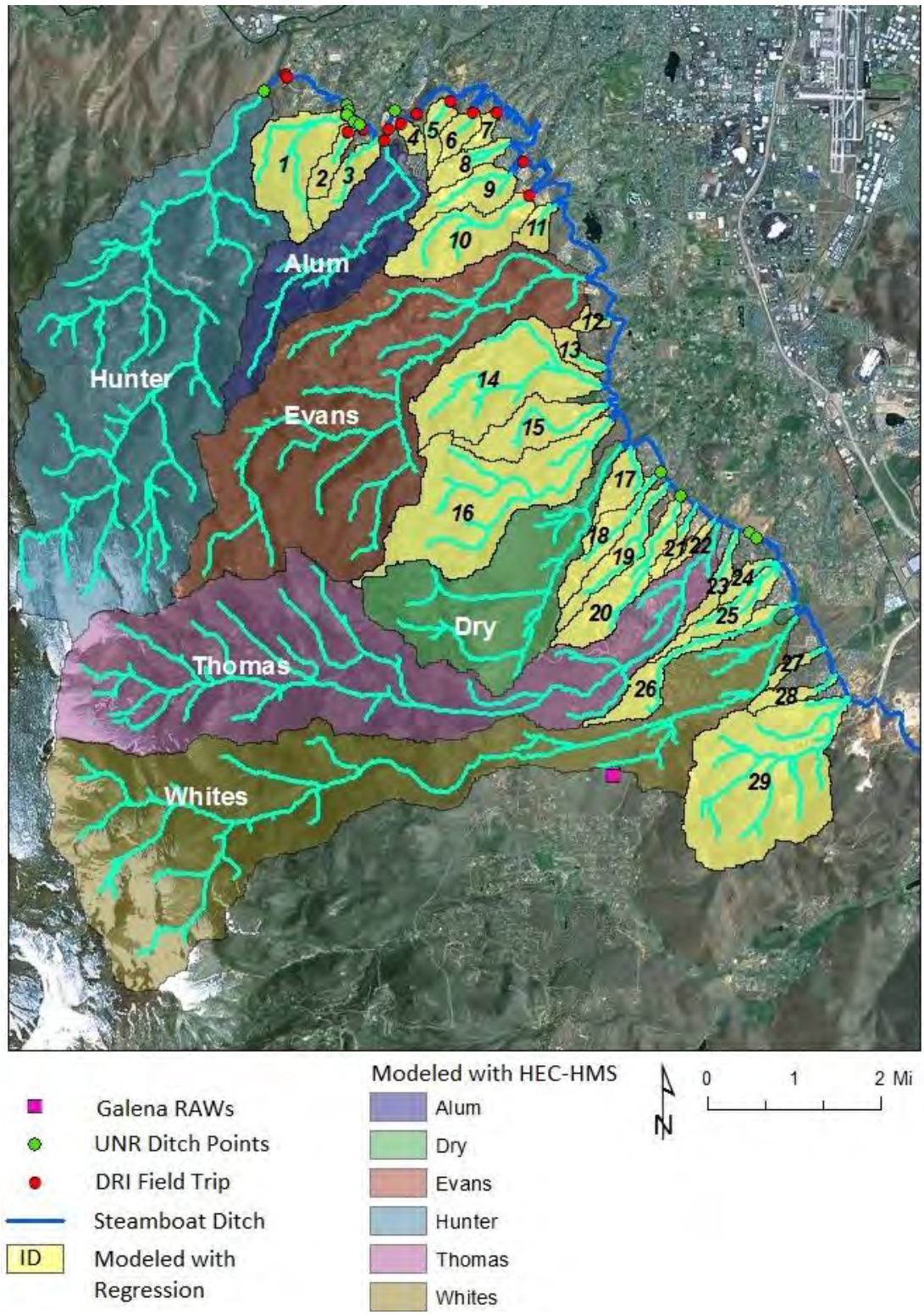

Figure 1: Location of Steamboat Ditch and watersheds modeled with either HEC-HMS or a regression. The Galena RAWs station is marked just south of the Whites Creek watershed, as are UNR ditch points and DRI field trip locations.

3.0 PREVIOUS MODELING EFFORTS

In 1990, the Federal Emergency Management Agency (FEMA) contracted Nimbus Engineers to conduct a restudy of Dry and Thomas Creek for flood insurance purposes. The Nimbus (1990) study predicted peak discharge for Dry Creek (at Steamboat Ditch) during a 100-year, 24-hour storm event to equal 1,343 cfs. Predictions for Thomas Creek in the vicinity of Steamboat Ditch were estimated at 2,640 cfs. Manhard (2010) constructed a planning level, calibrated regional hydrologic model for the City of Reno and simulated a range of return periods given a 24-hour precipitation event for select basins. Estimated peak discharges for Hunter Creek at the intersection with the Truckee River (below Steamboat Ditch) had simulated peak discharges of 253 cfs (2-year), 393 cfs (5-year), 527 cfs (10-year), 768 cfs (25-year), 1,072 cfs (50-year), and 1,521 cfs (100-year). Alum Creek peak discharges at the Truckee River (again, below Steamboat Ditch) were estimated at 315 cfs (2-year), 448 cfs (5-year), 570 cfs (10-year), 750 cfs (25-year), 919 cfs (50-year), and 1,148 cfs (100-year). Alum Creek 100-year estimated discharge at Steamboat Ditch was equal to 590 cfs, or about ½ the rate further downstream at South Virginia Street. Simulated peak discharge for Evans Creek at South Virginia were estimated at 212 cfs (2-year), 297 cfs (5-year), 516 cfs (10-year), 491 cfs (25-year), 1,569 cfs (50-year), and 2,281 cfs (100-year), with the 100-year event at Steamboat Ditch equal to 1,024 cfs, or ½ the flow simulated at South Virginia Street. Whites Creek splits into several branches with Manhard (2010) listing discharges in four channels at the split which occur above Steamboat Ditch with the sum of discharges in these channels simulated as 174 cfs (2-year), 538 cfs (5-year), 828 cfs (10-year), 1,490 cfs (25-year), 1,955 cfs (50-year), and 2,519 cfs (100-year). Estimated discharge 100-year discharge of Whites Creek north and south fork at Steamboat Ditch were 452 cfs and 306 cfs. To compare with this study, it would be necessary to combine Whites Creek estimates with basin ID# 27 and #28.

4.0 STEAMBOAT DITCH CAPACITY

Maximum flow capacity in the ditch is estimated assuming normal depth for which Manning's equation uses the channel bottom slope (S_o),

$$Q = \frac{1.4859}{n} A R^{0.667} S_o^{0.5} \qquad (1)$$

Where Q is flow in cubic-feet per second (cfs), A is cross sectional area of flow (ft^2), R is hydraulic radius (ft), n is Manning's roughness coefficient (dimensionless) and S_o is channel bottom slope (ft/ft). Steamboat ditch dimensions were measured near the junction of Cashill Boulevard and Gibraltar Drive, located just downstream from the exit point of basin ID #9 (refer to Figure 1). At this location, Steamboat ditch is best described with a trapezoidal shape with a bottom width of 9 feet, a top width of 12 ft and a depth of 2.1 ft. These dimensions produce a cross sectional area equal to 21.9 ft^2 and hydraulic radius of 1.5 ft at maximum flow capacity. A ditch bottom slope equal to 0.0014 ft/ft is taken from Manhard (2010). Figure 2 shows that bankfull discharge is sensitive to the choice of Manning's n. Concrete lined channels are defined with a

Manning's n ranging from 0.012 to 0.013, while unlined/earth channels range from 0.017 to 0.035 with values rising as ditch condition deteriorates. Natural river channels range from 0.025 to 0.050. A value of n = 0.017 is assumed representative of Steamboat ditch to produce a bankfull discharge equal to 96 cfs. For comparison, Manhard (2010) estimates ditch capacity at 131 cfs. Given Steamboat ditch is not uniform in geometry or condition along its entire length with only slight changes in channel geometry or Manning's roughness coefficient can alter ditch capacity. Therefore, for discharge entering Steamboat Ditch greater than 100 cfs will be designated as likely to flood the ditch, while discharge greater than 131 cfs will be assumed to cause flooding.

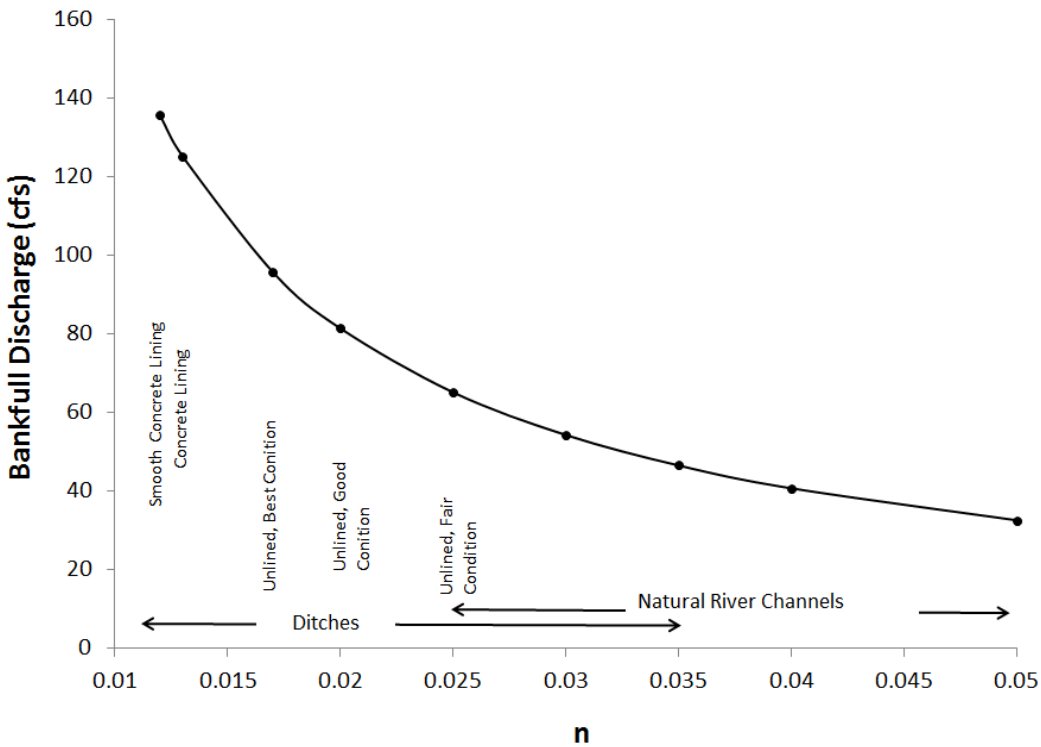

Figure 2: Normal depth calculations to estimate bankfull discharge in Steamboat Ditch given a range of Manning's roughness coefficients (n). A ditch depth of 2.1 was measured near the intersection of Cashill Boulevard and Gibraltar Drive.

5.0 NUMERIC MODEL AND THEORETICAL CONSIDERATIONS

The USACE Hydrologic Engineering Center Hydrolgoic Modeling System (HEC-HMS) computer model simulates precipitation-runoff and is capable or routing both natural and engineered flows. It is the successor of HEC-1 (USACE, 1998), a long-held industry standard for a single-event, lumped parameter, rainfall-runoff model. HEC-HMS improves upon HEC-1 by providing additional capabilities for distributed modeling as well as continuous simulation.

In general, uses of HEC-HMS do not require a detailed analysis of water storage and movement through the soil zone or saturated groundwater system. Instead, one is primarily interested in peak discharge, total runoff volume and/or hydrograph response to storm events. Under these circumstances, water from the near surface and overland flow are combined into a single term, direct runoff, while groundwater flow to/from streams is a single, user-defined baseflow. In order to parameterize HEC-HMS, one must define precipitation depths and distribution across the watershed as well as chose a method to compute precipitation losses, direct runoff (i.e. overland flow and interflow), baseflow and channel flow routing. Temperature and snowmelt must also be specified for snowmelt modeling. Evapotranspiration (ET) can be modeled directly with HEC-HMS with its soil-moisture accounting (SMA) algorithm, but for our purposes ET is lumped into all system losses and the SMA is not parameterized.

5.1 Meteorological Information

Rain-only simulations only require one to define precipitation depths and distributions. Two types of rain-only scenarios are conducted: (1) historic simulations and (2) hypothetical event-based simulations. Historic simulations require observed data at a specified interval (e.g. 1 hour) and to distribute this precipitation across the watershed using a variety of interpolation techniques.

Hypothetical event-based simulations use the U.S. Department of Agriculture (USDA) Conservation Service (SCS) 24-hour dimensionless hyetographs described in TR-55 (USDA, 1986). Type II storm distributions are assigned to Nevada with the dimensionless 24-hour cumulative rainfall distribution given Table 1. Actual precipitation depth is determined by multiplying the site-specific precipitation depth for a given return interval (e.g. 100 year, 24-hour event) with the dimensionless values, and subtracting successive values over discrete time periods to yield the required hyetograph for HEC-HMS input.

5.1 Precipitation Loss

HEC-HMS describes both directly-connected impervious surfaces as well as pervious surfaces. Impervious surfaces allow all precipitation to runoff, while pervious surfaces experience some amount of precipitation loss. HEC-HMS offers four loss models in which precipitation losses are computed for a given time interval and subtracted from precipitation depth. The excess precipitation depth is then uniformly distributed across the watershed area to become runoff volume.

For Steamboat Ditch, the SCS curve number (CN) model for precipitation loss is used. The SCS CN methodology is based on the assumption that direct runoff, after some initial abstraction (e.g. loss to storage depressions, interception, and plant uptake) will depend on land surface cover, land use, soil type and antecedent moisture conditions.

Table 1:SCS 24-hour fraction of storm total for Type II storm patterns.

Hours	Fraction
0	0
2	0.022
4	0.048
6	0.08
7	0.098
8	0.12
8.5	0.133
9	0.147
9.5	0.163
9.75	0.172
10	0.181
10.5	0.204

Continued....	
Hours	Fraction
11	0.235
11.5	0.283
11.75	0.357
12	0.663
12.5	0.735
13	0.772
13.5	0.799
14	0.82
16	0.88
20	0.952
24	1

The SCS approach is defined as,

$$P_e = \frac{(P-I_a)^2}{P-I_a+S}, \quad (2)$$

Where P_e is accumulated precipitation excess, or runoff, at time t, P is accumulated rainfall depth at time t, I_a is the initial abstraction, or loss, at time t and S is the potential maximum retention of water in the watershed.

The principal assumptions inherent to equation (1) include:

The depth (amount measured in length, i.e., inches) of cumulative excess precipitation (P_e) for runoff is always less than cumulative total precipitation (P).

The depth of precipitation that recharges (F_a), or is lost from the calculation of runoff volumes, is less than some potential maximum retention, or storage, in the basin (S).

There is some amount of rainfall (I_a) that occurs before any runoff can occur such that the maximum runoff is P-I_a.

The ratio of actual-to-potential (maximum) for infiltration and runoff are equal.

Empirical data suggests that no runoff will occur until cumulative precipitation reaches 20% of the basins maximum storage capabilities.

$$I_a = 0.2S, \quad (3)$$

Combining equation (2) and (3), cumulative runoff at time t equals

$$P_e = \frac{(P-S)^2}{P+0.8S}. \quad (4)$$

The maximum storage (inches) in the basin is a function of the CN,

$$\mathrm{S} = \frac{1000-10\mathrm{CN}}{CN}, \quad (5)$$

in which CN ranges from 100 for impervious materials (and water bodies) to 30 for very permeable soils with high infiltration rates.

5.2 Direct Runoff

Direct runoff is the process by which excess precipitation is transferred into point runoff from the watershed. One empirical method to perform this transform is the unit hydrograph which defines basin outflow response to a uniform rainfall rate over a specified time interval. The principal assumption is that runoff is linearly related to rainfall, such that a doubling of rainfall results in a doubling of runoff.

The SCS unit hydrograph model is used for Steamboat Ditch (Figure 3) in which the unit hydrograph peak discharge (U_P) and time to peak (T_P) are related by the watershed area (A). Using English units (foot-pound system) the relationship is expressed as,

$$U_P = 484\frac{A}{T_P}. \tag{6}$$

Time to peak discharge is a function of the duration of excess precipitation (Δt) and lag time (t_{lag}),

$$T_P = \frac{\Delta t}{2} + t_{lag}, \tag{7}$$

where t_{lag} is defined as the time difference between the center of mass of rainfall excess and the peak of the unit hydrograph. With tlag defined, HEC-HMS solves equation (7) for T_p, and then solves equation (6) for U_P. These values then populate the dimensionless unit hydrograph to provide a basin-specific unit hydrograph.

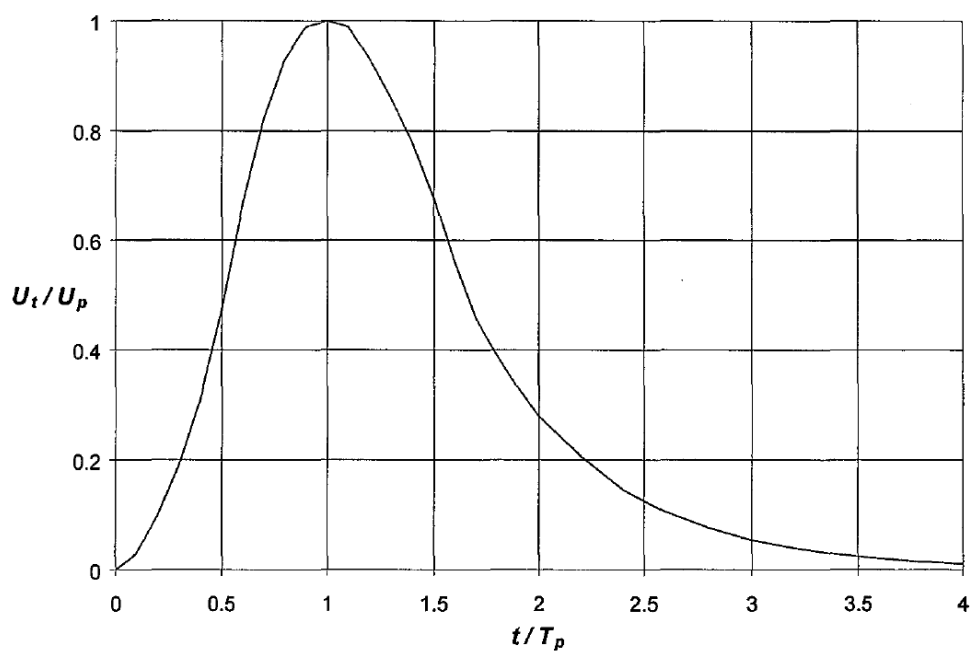

Figure 3: SCS dimensionless unit hydrograph (obtained from USACE, 2000b, p. 59).

The SCS parameter t_{lag} is often used as a calibration parameter for gaged basins. To estimate t_{lag} for ungagged basins, or to provide a starting value for parameter calibration, lag time is related to time of concentration (t_c)

$$t_{lag} = 0.6t_c. \quad (8)$$

Time of concentration is estimated as the sum of travel time related to sheet flow (t_{sheet}), shallow flow in rills and rivulets ($t_{shallow}$) and channel flow ($t_{channel}$). Sheet flow is estimated with an approximate solution to the kinematic wave equation,

$$t_{sheet} = \frac{0.007(NL)^{0.8}}{(P_2)^{0.5}S^{0.4}}, \quad (9)$$

Given N = overland roughness coefficient, L = flow length, P2 is the 2-year, 24-hour rainfall depth (inches) and S is land slope. Both shallow and channel flow times are estimated as the quotient of length of channel to velocity of flow. Velocity for shallow flow is estimated as $16.1345(S)^{0.5}$ for unpaved surfaces and $20.3282(S)^{0.5}$ for paved surfaces. Velocity for channel flow is calculated using Manning's equation (refer to equation (1)).

5.3 Baseflow

Baseflow over time (t) is modeled using the HEC-HMS recession model,

$$Q_t = Q_o K k^t, \quad (10)$$

given Q_o is the initial baseflow and k is the exponential decay constant. For hypothetical simulations, Q_o is set to an average flow that would occur at the start of a storm event. The decay constant, k is generally less than 1.0 with values dependent on source of baseflow. For groundwater systems, the decay rate generally falls between 0.95 and 1.0 such that baseflow approaches a constant rate. For interflow driven baseflow, baseflow recedes more quickly with k falls between 0.8 and 0.9.. Surface runoff derived baseflow is more flashy with a more rapid rate of recession. This translates into a lower range of decay rates, generally between 0.3 and 0.8.

5.4 Stream Flow Routing

Using a finite difference approximation of the continuity equation and a simplification of the momentum equation (USACE, 2000b), HEC-HMS solves the kinematic wave equation in 1-dimension to calculate a downstream hydrograph. Required information includes channel width, channel bottom slope, cross sectional shape and associated channel side slope, Manning's roughness coefficient, and downstream initial flow conditions. HEC-HMS calculates boundary conditions of upstream, lateral and tributary inflow hydrographs.

5.5 Snowmelt

In addition to those parameters defined for rain-only simulations, rain-on-snow requires one to parameterize temperature and snowmelt. Temperature can be distributed using lapse rates determined from observed data. Snowmelt uses both precipitation and temperature to determine if precipitation falls as rain or as snow and added to accumulated. Accumulation snow is subject to melt with melt water added to rainfall to become the basin hyetograph subject to possible losses and runoff.

The temperature index approach to snowmelt is essentially a degree-day approach in which there is a fixed amount of snow melt for each degree above freezing (or a defined temperature for which melting begins). Some memory is contained in the system by applying coefficients to the melt rate based on number of degree-days experienced. In addition snowpack internal conditions and changes in temperature will modify the melt rate. Important parameters for the temperature index method are described below (USACE, 2000a):

- ICSWE: Initial snow water equivalent (SWE) must be specified prior to model execution in terms of inches of water.
- INITCC: The initial cold content that exists at the beginning of the simulation as defined as the heat required to raise the temperature of the snowpack to 32°F and is expressed as inches of accumulated water.
- ICLIQ: The initial liquid water held in the snowpack. For water to exit the snowpack the liquid water holding capacity (PLWHC) must be met. The initial value is set to zero if there is no initial SWE or if the snowpack is cold at the start of the simulation.
- ICATICC: Initial cold content antecedent temperature index and should be set to the temperature of the snowpack at the start of the simulation. If this temperature is not known, then it is usually set to 32°F.
- BASE: Base temperature used to estimate heat transfer. The meltrate is multiplied by the difference between BASE and air temperature, or the temperature index, to calculate snowmelt amounts. If the air temperature is less than BASE then no snowmelt occurs. Generally this value is close to 32°F.
- PX: temperature to distinguish between precipitation falling as rain or snow. When air temperature is less than PX, then all precipitation falls as snow, while temperatures above PX force all precipitation to fall as rain. PX is often a few degrees above freezing.
- WET-MELTRATE: The snow melt rate when precipitation falls as rain at rates greater than the RAIN-RATE-LIMIT.
- RAIN-RATE-LIMIT: The rain-rate limit that differentiates between dry melt and wet melt. When rain falls at rates larger than RAIN-RATE-LIMIT then WET-MELTRATE is used to melt accumulated snowpack. If rain falls at lower rates, then snowpack is melted as if no rain is falling.
- ATIMR: A paired function that describes the antecedent temperature index melt rate function. The function defines melt rates over a range of melt rate index

values encountered during the simulation. The melt rate index is defined as the number of degree-days for which melt occurs consecutively. The melt rate index generally ranges from 0.015 to 0.550.

- ATI-MELTRATE: A cold melt rate is specified for when precipitation is less than RAIN-RATE-LIMIT. A coefficient used to update the antecedent melt rate index from one time interval to another when precipitation is less than RAIN-RATE-LIMIT. A value of 0.98 is typically used to describe a maturing snowpack.
- COLD-LIMIT: Accounts for rapid changes in temperature that the snowpack may undergo during high precipitation events. When precipitation rates exceed the specified COLD-LIMIT the antecedent cold content index is set to the temperature of the precipitation. If temperature is above BASE then the cold content index is set to BASE. If temperature is below BASE then the cold content index is set to the actual temperature. If precipitation falls at a rate less than COLD-LIMIT, then the cold content index is computed as an antecedent index.
- ATI-COLDRATE: A coefficient used to update the antecedent cold content index from one interval to the next and simulate a maturing snowpack. Generally this coefficient is on the order of 0.84.
- PLWHC: The percent liquid water holding capacity that must occur before water becomes available for infiltration or runoff. Values of 3-5% are typical.
- GMLT: heat from the ground can cause snowmelt to occur. A fixed value or a annual pattern can be specified.

6.0 MODEL PARAMETERIZATION

6.1 Precipitation

Historic simulations relied on 1-hour observed data collected at Galena RAWs (NWS ID 260108) metrological station. Specifically, precipitation data spanning October 24, 2010 through October 28, 2010 were used to calibrate the rain-only scenario, while data from December 18, 2010 through December 22, 2010 was used to calibrate snowmelt parameters. Galena RAWs precipitation was then distributed across the study site using the ratio of the 30-year average annual (years 1981-2010) PRISM (Parameter elevation Regressions on Independent Slopes Model, (http://www.prism.oregonstate.edu/) precipitation depth between the observed gage and locations across the study site. Spatial distribution of these ratios at the 800 m resolution is provided in Figure 4.

Hypothetical, event-based storms are simulated with return periods of 2-, 5-, 10-, 25-, 50, 100-, 200-, 500 and 1000-year. Twenty-four hour total storm precipitation for each return period was obtained from the NOAA's Atlas-14 (Bonnin et al., 2011). Data have been spatially distributed across the landscape at the 0.0083° resolution (http://hdsc.nws.noaa.gov/hdsc/pfds/index.html) and then resampled to 100 m. Actual precipitation depth is determined by multiplying the site-specific precipitation depth with the dimensionless values for Type II storms, given in Table 1, and subtracting

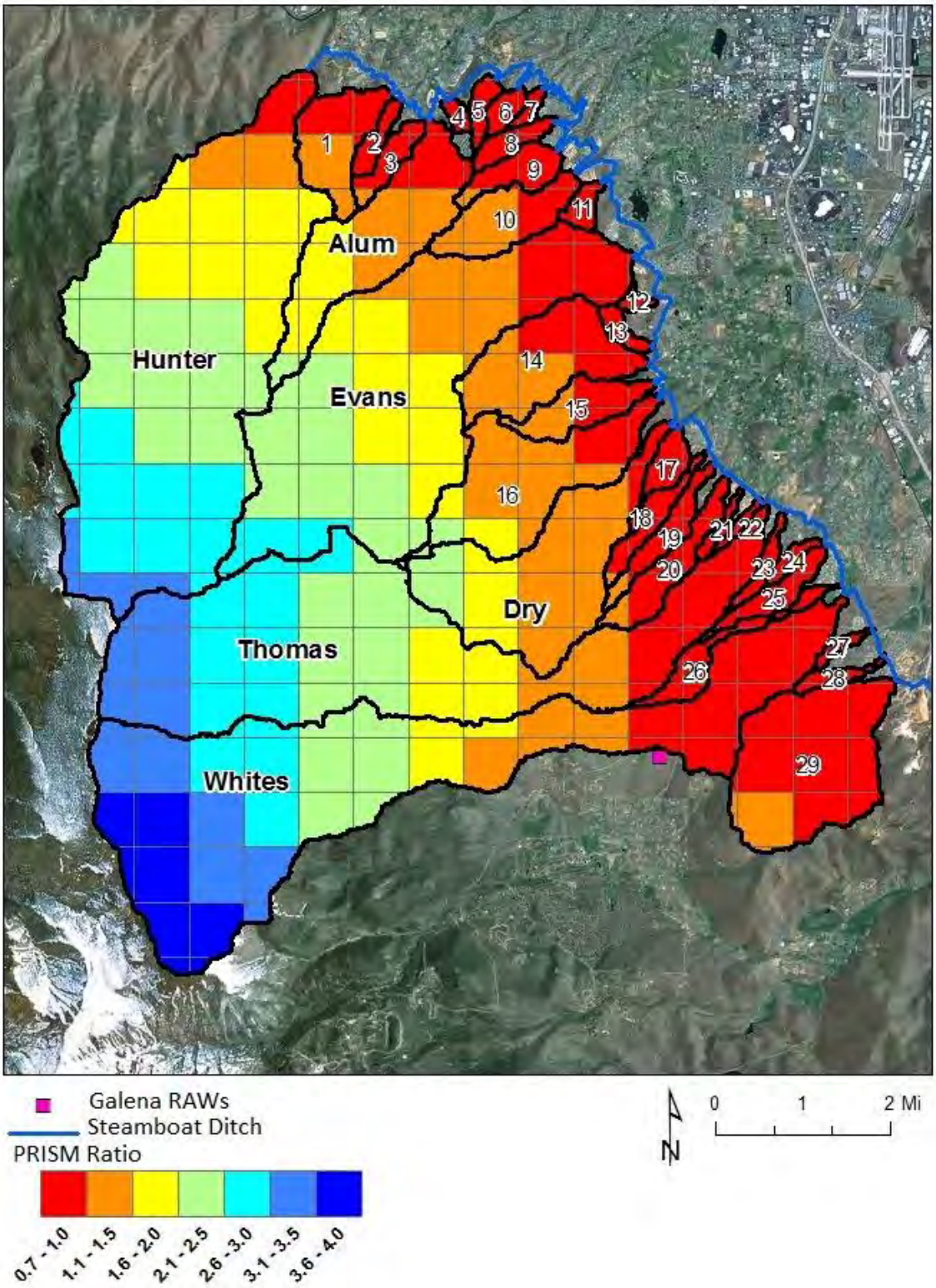

Figure 4: Ratio of 30-year average annual PRISM precipitation to the Galena RAWs n 30-year annual average PRISM precipitation. Resolution is 800 m. Values used to distribute observed RAWs data across the study site.

precipitation losses over discrete time periods to yield the required hyetograph for HEC-HMS input.

6.2 Precipitation Loss

Precipitation losses are primarily defined by the percent impervious area and the CN. Percent impervious is based off of values used by Manhard (2010). Undeveloped land and agricultural conditions contain 0% impervious land Low, medium and high development area assumed to occur at the 2 acre, 1/2 acre and 1/8 acre-scale and contain 12%, 20% and 65% impervious conditions, respectively. Roads are assigned 98% impervious while water is defined as 100% impervious. Figure 5 shows the spatial distribution of percent impervious across the study site. Development is primarily focused in the lower portions of each watershed, with Dry, Thomas and White Creek containing the most areally extensive development. In contrast, Hunter Creek contains no development and 0% impervious area.

With respect to the CN, US Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS, 1986) has compiled CN values for a range of conditions, including semi-arid, as a function of hydrologic soil group (HSG) as well as vegetation type and condition (Table 2). Soils are classified into one of four HSG categories (A, B, C and D) according to their minimum infiltration rate on bare soil and after prolonged wetting (NRCS, 1986; USDA, 2009). Group A soils have high transmission rates (> 0.30 inches per hour) and low runoff potential. These generally consist of deep, well to excessively drained sands and gravels. Group B contains moderately to well drained soils of moderately fine to moderately coarse texture. Transmission rates are on the order of 0.15 to 0.30 inches per hour. Group C soils have low infiltration rates based on moderate to moderately fine texture and generally contain a layer that impedes the downward movement of water. Transmission rates are 0.05 to 0.15 inches per hour. Finally, group D soils have very low infiltration rates with high runoff potential. These soils consist primarily of clays with high swelling potential; a permanently high water table; claypan or clay layer at or near the land surface; and/or shallow soils over an impervious material. Transmission rates range from 0 to 0.05 inches per hour.

Table 3 provides soil names and USDA soil identification numbers pertaining to the study site grouped by HSG. Soil data was obtained from the USDA web-based data repository (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey .aspx). Figure 6 shows the project area's spatial distribution of HSGs. Lower elevations are dominated soils with poor infiltration capacities. Soil infiltration capacity generally improves with gains in elevation, except for those portions of the upper basin containing rock outcrops.

Vegetation type and density were obtained from the US Geologic Survey (USGS) Landfire data repository at the 30 m resolution (http://www.landfire.gov/vegetation.php). A cover density less than 30% is considered in poor condition, equal to or greater than 30% but less than 70% is ranked as fair condition while greater than or equal to 70% cover density is listed as good condition. Developed areas were assumed to contain primarily grass, unless specifically designated as forest or shrubland. Low, medium and high

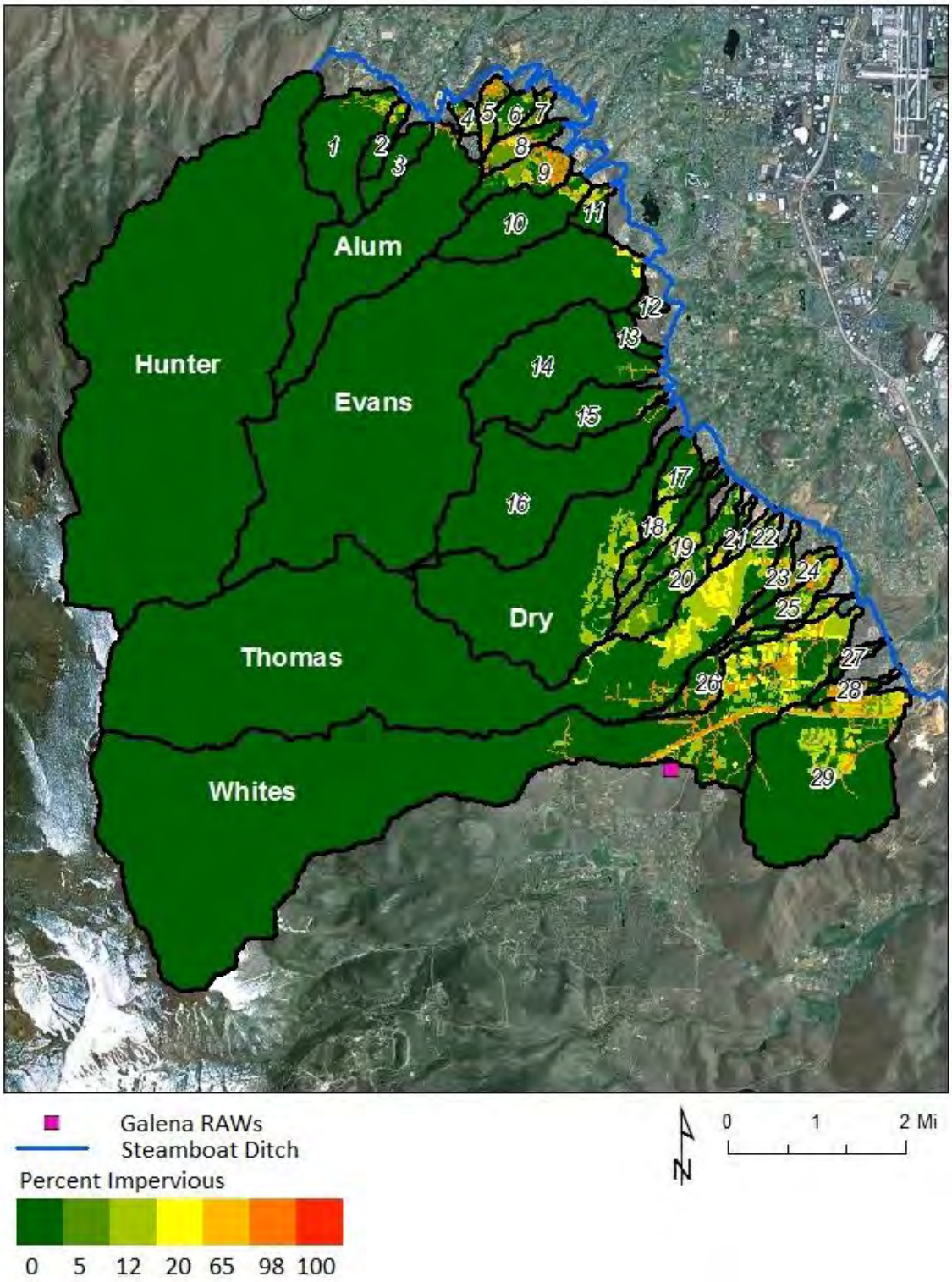

Figure 5: Spatial distribution of percent impervious.

Table 2: CN designation based on cover type and condition as well as hydrologic soil group, or HSG for semi-arid conditions (modified from NRCS, 1986).

Cover Type	Condition[a]	HSG			
		A	B	C	D
Grass	poor	65	80	87	93
	fair	54	71	81	89
	good	44	62	74	85
Trees	poor	54	66	74	79
	fair	36	48	57	63
	good	21	30	41	48
Pinyon-Juniper	poor	61	75	85	89
	fair	44	58	73	80
	good	29	41	61	71
Shrubs	poor	63	77	85	88
	fair	55	72	81	86
	good	49	68	79	84

[a] density < 30 is poor; 30≤density<70 is fair; density≤70% is good

development was assigned good, fair and poor conditions, respectively. Rock quarry development is assigned the same CN as grass given poor conditions. Figures 7, 8 and 9 show the resulting spatial distributions of cover type, condition and CN for the project area, respectively. In general, CN at higher elevations is less than 60 based on soils with larger infiltration capacities, abundance of tree cover with greater than 30% cover density. Lower portions of each watershed contain CNs mostly in excess of 80 as a result of poor soils and vegetation dominated by grasses and shrubs at densities less than 30%.

6.3 Direct Runoff

Time of concentration parameters required to estimate t_{lag} were computed with a 30m digital elevation model (DEM) and the USACE HEC-GeoHMS (2013) interface for geographic information systems analysis. Initial t_{lag} values computed with HEC-GeoHMS were then adjusted in the calibration process and discussed in more detail in section 7.1

6.1 Baseflow

Baseflow is an important flow component in Hunter Creek and it was necessary to include baseflow when when calibrating to observed flows. Initial baseflow (Q_o) and an exponential decay constant (k) equal to 5 cfs and 0.995, respectively, best match observed flow behavior in Hunter Creek. A decay rate nearly equal to 1 suggests most baseflow out of Hunter Creek is groundwater derived, not obtained from interflow or surface

Table 3: List of soils by HSG for the project area. ID numbers correspond to USDA classification.

HSG	ID	Soil Names
A	140; 460; 480; 482; 490; 513; 520; 530; 640; 756; 810; 812; 813; 1100; 1120; 1121; 1440	Haybourne loamy sand; Surprise loamy sand; Holbrook gravelly loamy sand; Holbrook cobbly loamy sand; Graufels bouldery sand; Settlemeyer-Notus complex ; Dressler loamy sand; Notus stony loamy fine sand; Toiyabe-Corbett-Haypress association; Rose Creek fine sandy loam; Rose Creek loamy fine sand; Rose Creek gravelly fine sandy loam; Graylock-Temo-Rock outcrop complex; Apmat very stony coarse sand
B	430; 660; 661; 662; 668; 669; 1470	Sagouspe variant loamy very fine sand; Oest very bouldery sandy loam; Oest bouldery sandy loam; Oest extremely stony sandy loam; Oest very bouldery sandy loam (steep); Oest gravelly sandy loam; Carioca-Sibelia variant-Fugawee association
C	203; 454; 470; 510; 550; 551; 553; 559; 590; 595; 600; 601; 602; 621; 623; 670; 800; 805; 890; 910; 911; 971; 974; 1080; 1130; 1194; 1430; 1432; 1460; 230; 450; 514; 740	Northmore sandy loam; Voltaire silty clay drained; Dalzell loamy fine sand; Settlemeyer fine sandy loam; Leviathan stony sandy loam; Leviathan very stony sandy loam; Leviathan extremely stony sandy loam; Springmeyer stony loam; Springmeyer sandy clay loam; Idlewild clay loam ; Idlewild sandy loam; Idlewild gravelly loam; Orr stony sandy loam; Orr sandy loam; Truckee silt loam; Truckee silt loam ; Truckee sandy loam gravelly substratum; Washoe gravelly sandy loam; Indiano gravelly loam warm; Vamp fine sandy loam; Vamp silt loam; Aladshi sandy loam; Aladshi gravelly sandy loam; Inville variant gravelly sandy loam; Dithod sandy loam; Spasprey stony sandy loam; Fraval-Booford-Jumbo association; Fraval-Hirschdale-Jumbo association; Jorge-Boomtown-Fugawee association; Cradlebaugh loam; Voltaire loam; Settlemeyer gravelly loam; Blackwell sandy loam
D	101; 171; 173; 174; 190; 210; 221; 222; 241; 250; 262; 280; 282; 310; 312; 313; 341; 350; 390; 391; 420; 585; 612; 613; 614; 615; 630; 632; 650; 652; 653; 681; 683; 730; 731; 753; 772; 775; 780; 861; 862; 863; 870; 871; 872; 880; 881; 882; 900; 903; 930; 931; 980; 982; 994; 1010; 1050; 1051; 1052; 1054; 1271; 1450; 1480	Aquinas sandy loam; Indian Creek gravelly sandy loam; Indian Creek sandy loam; Indian Creek extremely stony sandy loam; Manogue cobbly clay; Luppino gravelly sandy loam; Oppio cobbly sandy loam; Oppio cobbly sandy loam; Updike loam gravelly substratum; Cassiro gravelly sandy loam; Acrelane very stony sandy loam; Wedekind gravelly loam; Wedekind gravelly sandy loam; Risley-Rock outcrop complex; Risley cobbly loam; Risley cobbly clay loam; Yuko stony loam; Mizel very gravelly coarse sandy loam; Duckhill stony loam; Duckhill-Hirschdale-Fraval association; Godecke loamy sand; Barnard-Trosi association; Verdico very stony sandy loam; Verdico extremely stony sandy loam; Verdico extremely stony sandy loam; Verdico sandy loam; Fleischmann gravelly clay loam; Fleischmann loam; Chalco very stony clay loam; Chalco stony loam; Chalco cobbly sandy loam; Reno very stony fine sandy loam; Reno stony sandy loam; Stodick very stony loam; Stodick stony loam; Toiyabe-Corbett-Rock outcrop association; Booford very stony sandy loam; Booford very stony loam; Bieber stony sandy loam; Reywat extremely stony loam; Reywat very cobbly sandy loam; Reywat-Rock outcrop complex; Xman-Rock outcrop complex; Xman very stony loam; Xman very stony sandy loam; Zephan-Rock outcrop-Smallcone complex; Zephan very gravelly sandy loam; Zephan stony sandy loam; Flex very gravelly sandy loam; Flex stony sandy loam; Old Camp stony sandy loam; Old Camp-Rock outcrop complex; Koontz gravelly loam; Koontz stony loam; Badland-Chalco-Verdico complex; Gabica very gravelly sandy loam; Waspo clay; Waspo stony clay; Waspo-Rock outcrop complex; Waspo gravelly clay; Tristan-Barshaad-Arzo association; Meiss-Sibelia-Rock outcrop association; Macareeno-Blackwell-Carioca association

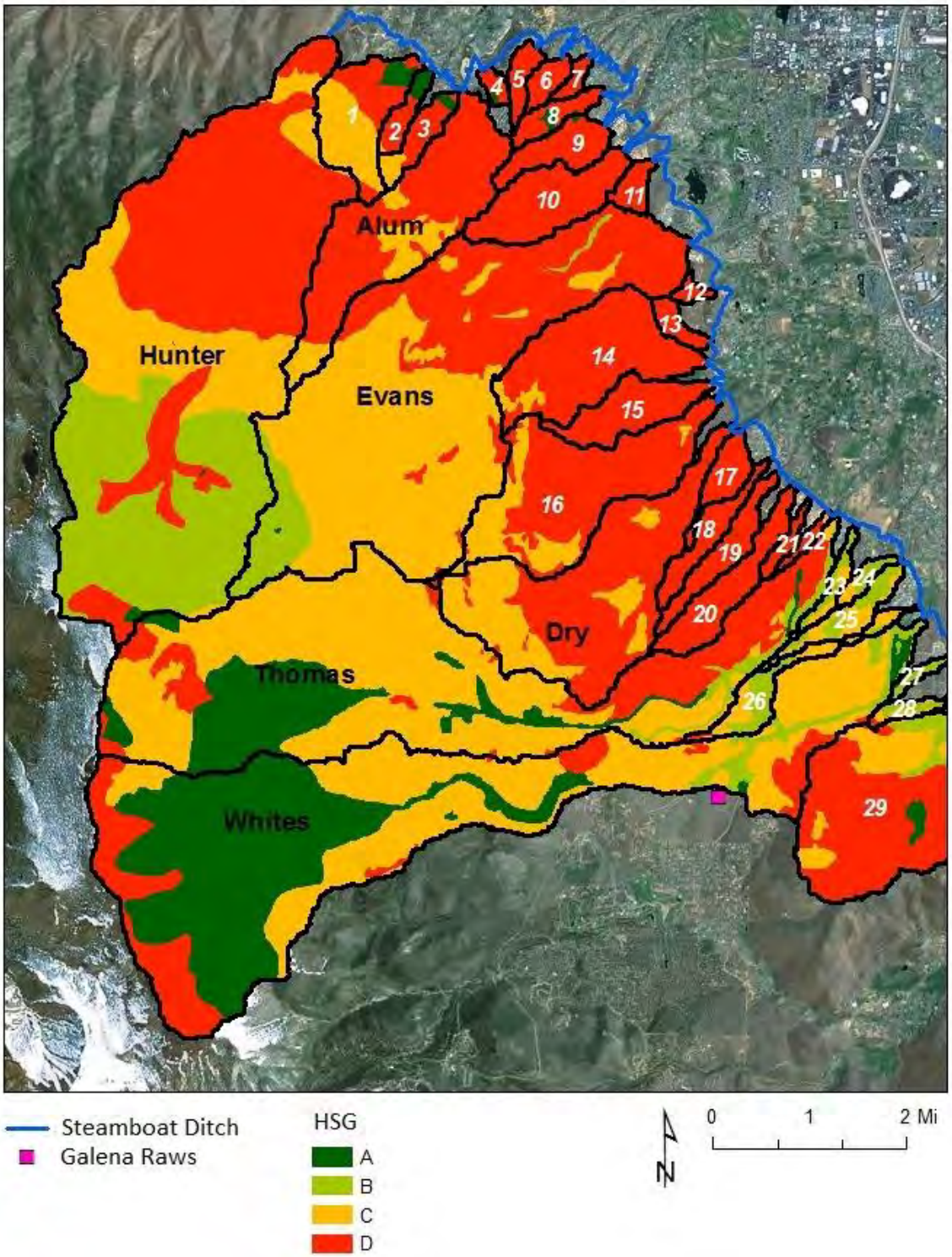

Figure 6: Spatial distribution of hydrologic soil groups (HSG).

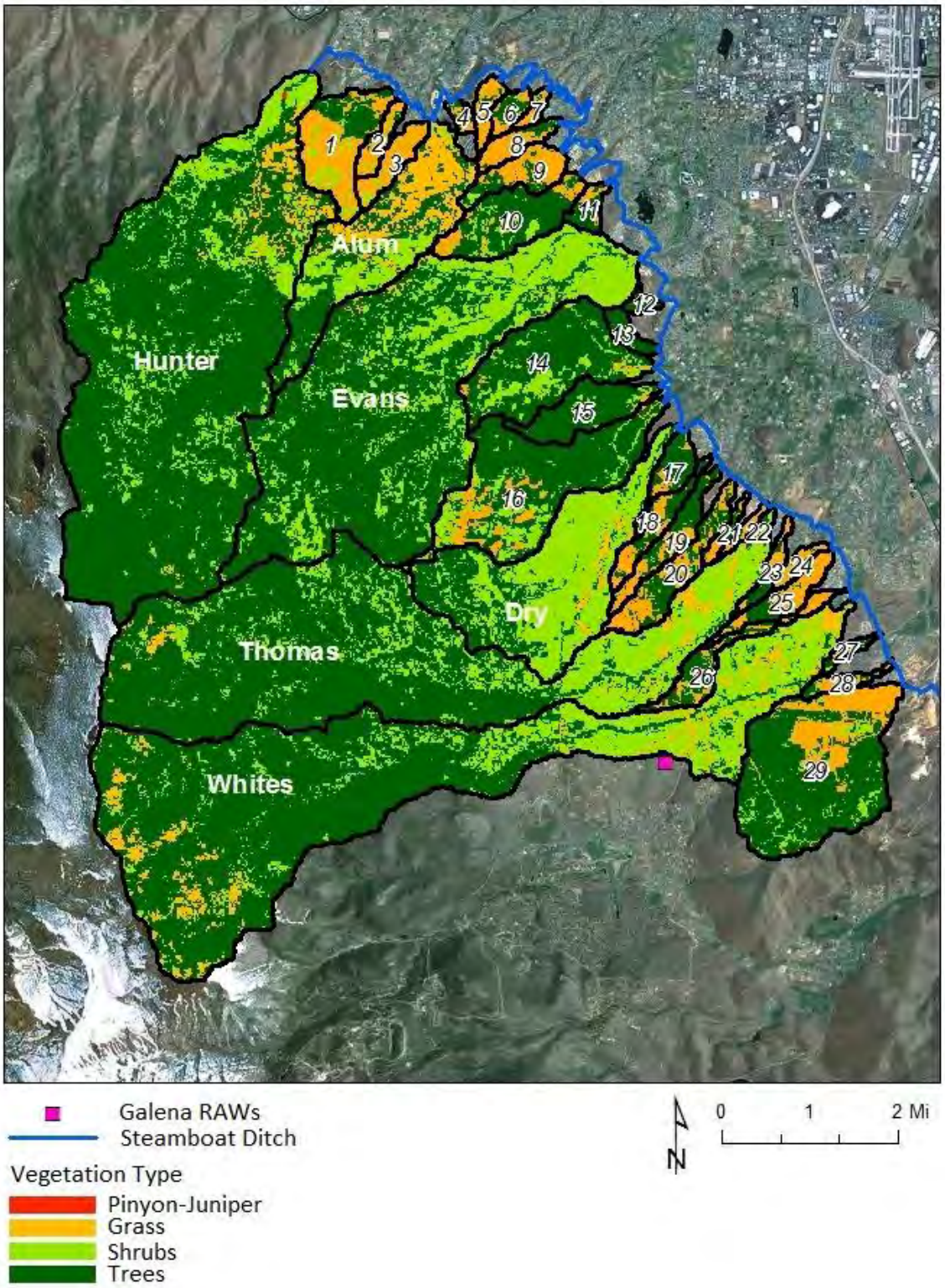

Figure 7: Spatial distribution of cover type.

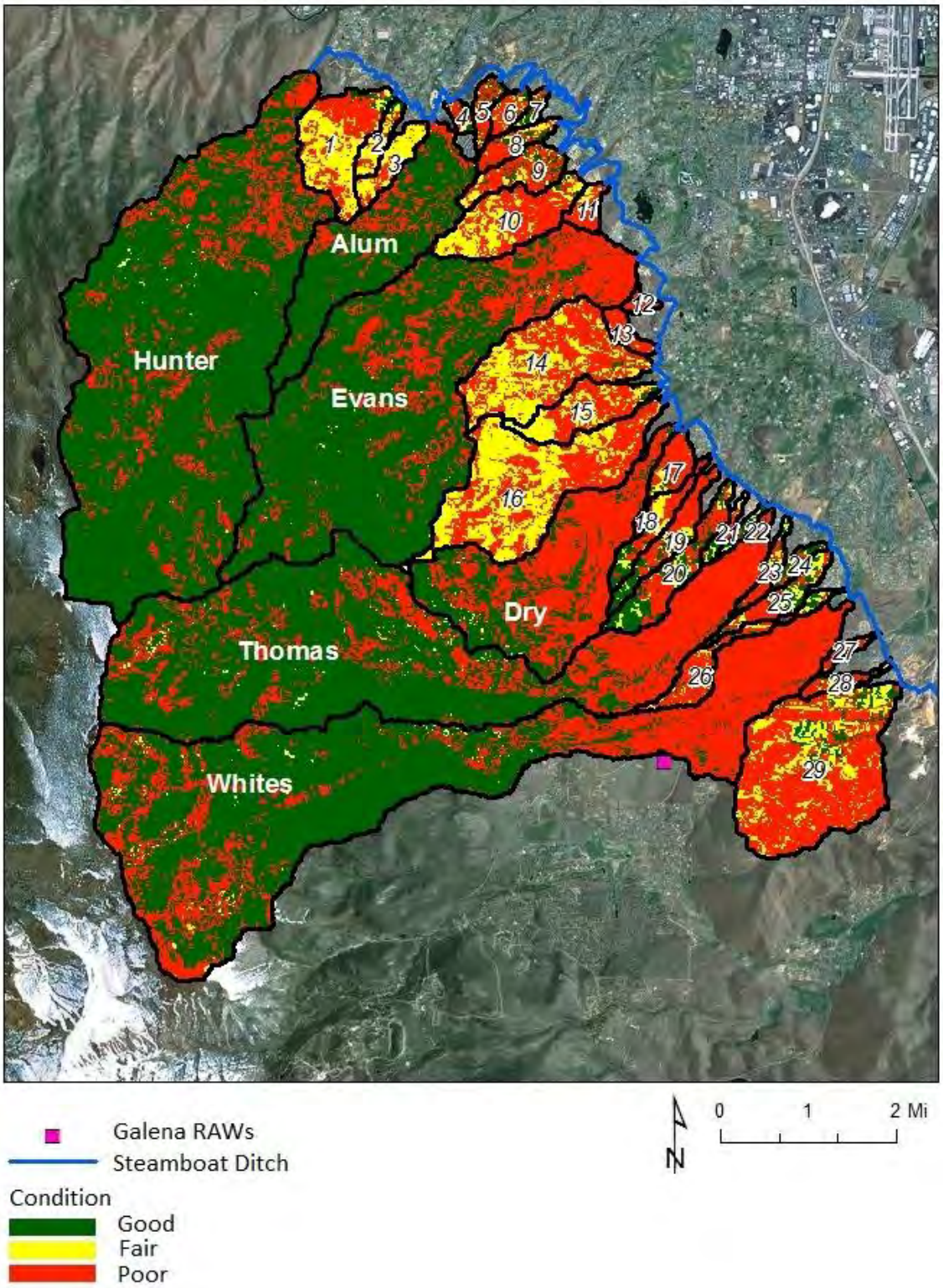

Figure 8: Spatial distribution of cover condition.

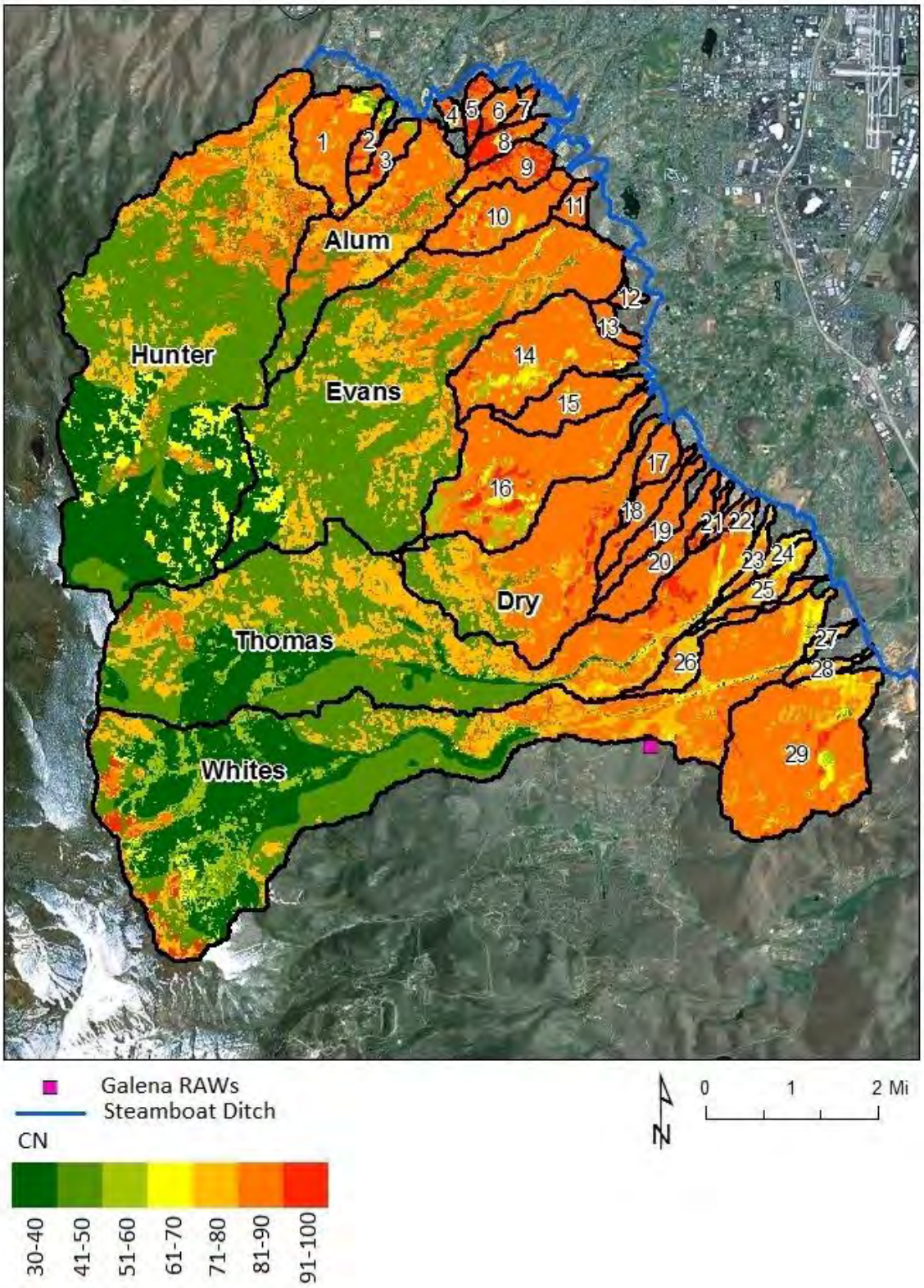

Figure 9: Spatial distribution of CN.

runoff. Baseflow is not included in the other watershed models despite casual observation that baseflow does exist in some of these creeks. Specifically, Alum Creek was flowing on the order of 1 cfs when visited by Pohll and Carroll during September 25, 2013. Other than Hunter Creek, all basins are only modeled with hypothetical design-storms and it is assumed that baseflow is not a significant contributor to peak discharge under these conditions.

6.2 Stream Flow Routing

Hunter Creek stream geometry is assumed representative of all stream reaches modeled. Figure 10 shows the creek as it is traverses under Steamboat Ditch on September 25, 2013 flowing at 3.5 cfs (daily average). The channel is assumed trapezoidal in shape with a bottom width of 8.8 ft and a side slope of 2:1. Dimensions were obtained by best matching observed rating curves describing flow-versus-area and flow-versus-width and appear reasonable upon casual observation of the creek. Channel bottom slopes were calculated using a 30 m DEM for each stream reach while the Manning roughness coefficient was assumed 0.03.

Figure 10: Hunter Creek exiting its watershed under Steamboat Ditch, September 25, 2013. Mean daily discharge this day was 3.4 cfs. Photo taken by R. Carroll (DRI).

6.3 Snowmelt

In addition to those parameters defined for rain-only simulations, rain-on-snow requires one to parameterize temperature and snowmelt.

Temperature lapse rates are assumed only a function of elevation and set equal to a loss of 2.39°F per 1,000 ft elevation gained. This rate was calculated using average daily temperatures between the Galena RAWs and Mt. Rose SNOTEL station located just south-west of Whites Creek watershed and situated at an elevation of 8,801 ft.

Initial conditions for SWE (ICSWE) were obtained from daily NOAA SNODAS data sets (http://nsidc.org/data/docs/noaa/g02158_snodas_ snow_cover_model/index.html) given 1-km resolution and set to those observed on December 30, 2010 (Figure 11). This date was chosen to ensure snow covered all portions of the study site, while maintaining a realistic distribution of SWE across the basin for the time of year that most rain-on-snow events occur in the region (i.e. December through January). It is noted that initial SWE depths are not infinite and therefore can become rate-limiting. In other words, all accumulated SWE has the potential to melt before the simulation is complete. However, it is felt that unlimited initial SWE is not a realistic assumption.

No calibration was done for snowmelt modeling. Instead, initial parameters describing the cold rate coefficient (INITCC), liquid content (ICLIQ) and cold content antecedent temperature index (ICATICC) were not known and set to HEC-HMS manual recommendations. Other snowmelt parameters were obtained from the USACE (2011) study for the Sacramento River Basin. For an initial simulation Galena RAWs air temperatures (T_{air}) was assumed 38°F to place a few degrees above BASE. T_{air} was held constant for the duration of the 24-hour precipitation event, after which it was lowered below BASE to ensure no further melt of the snowpack. Snowmelt input parameters are provided in Table 4.

6.1 Sub-Watershed Discretization

Using the USACE HEC-GeoHMS (2013) geographic information systems tool and a 30 m digital elevation model (DEM), sub-watershed delineation was accomplished by flow direction and accumulation thresholds. Sub-watersheds with tributaries that were either small in length or accessed terrain of similar hydrologic characteristics was, in some instances, combined. The resulting sub-watershed delineation and stream reaches modeled with HEC-HMS are given in Figure 12. Area-weighted average input parameters were then assigned to each sub-watershed. Peak discharge was modeled from each sub-watershed to determine which portions of each basin have the greatest potential to contribute flooding during major storm events.

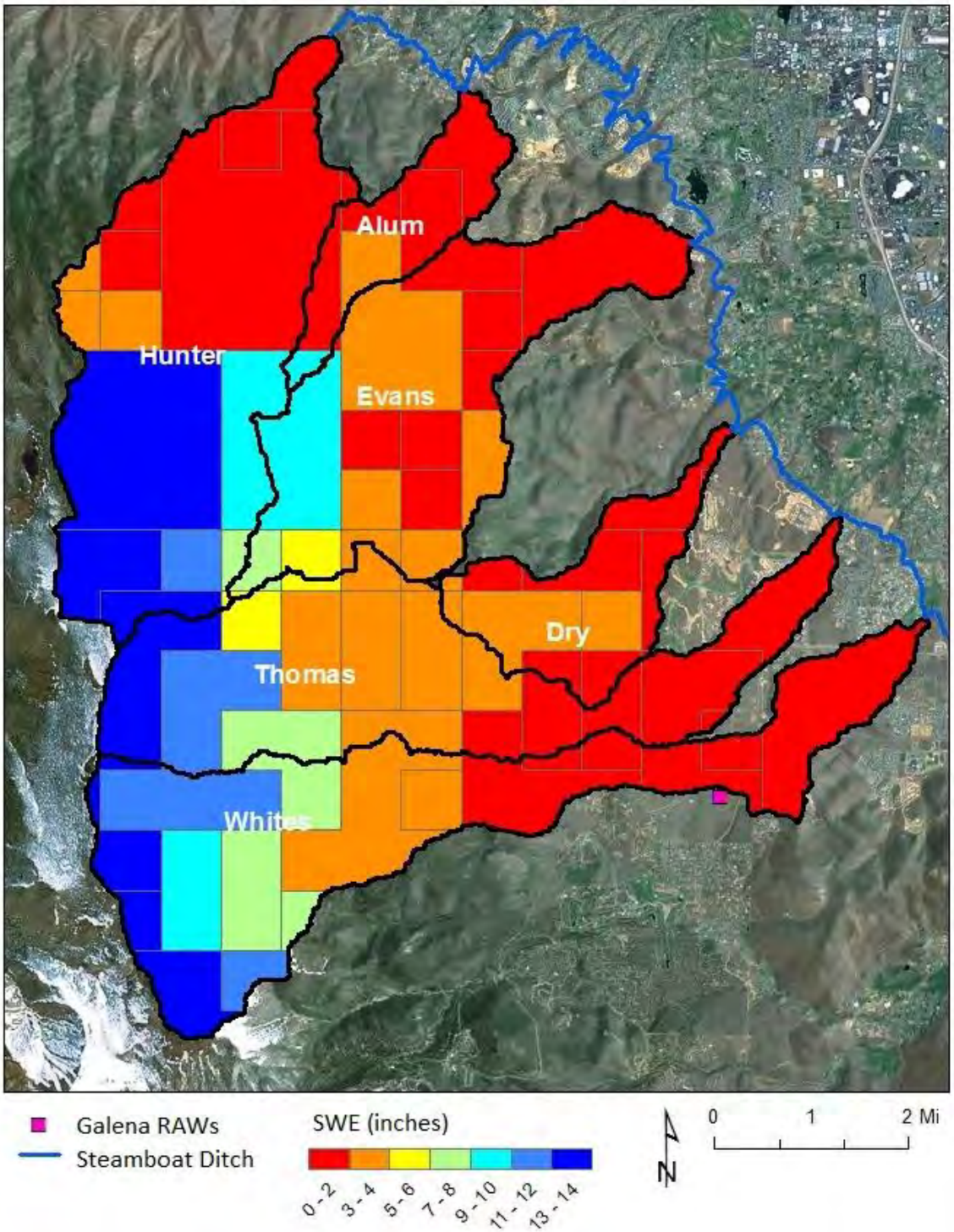

Figure 11: Initial SWE in inches for rain-on-snow modeling done with HEC-HMS.

Table 4: HEC-HMS snowmelt parameter values.

Parameter	Units	Value
ICSWE	in	22-Dec-10
INITCC	in	0
ICLIQ	in	0
ICATICC	°F	32
BASE	°F	32
PX	°F	35
RAIN-RATE-LIMIT	in/hr	0.02
WET-MELTRATE	in/°F	0.15
COLD-LIMIT	in/d	0.04
ATI-MELTRATE	-	0.98
ATI-COLDRATE	-	0.9
PLWHC	%	5
GMLT	-	0.02
ATIMR	in/°F-d	0.3
Galena T_{air}	°F	38

7.0 MODEL CALIBRATION

Hunter Creek is the only watershed in the study site with a USGS gaging station and available data for calibration. Observed discharge at the 15-minute timescale was used over a four-day period (Figure 13) beginning October 18, 2010 (midnight) through October 22, 2010 (midnight).

7.1 Rain-Only

Calibration was done in a two-step process. First it was necessary to adjust precipitation losses to match observed total discharge. Second, lag-time of direct flow was adjusted to match hydrograph shape.

Estimated precipitation volume onto Hunter Creek (described in section 6.1) equals nearly 3,500 acre-feet for the duration of the October rain event. Observed stream discharge during the same event, including baseflow, is 83 acre-feet, or about 2% estimated precipitation. This ratio of observed discharge to potential precipitation is low compared to other basins in the region which have discharge ranging from 21 to 27% of

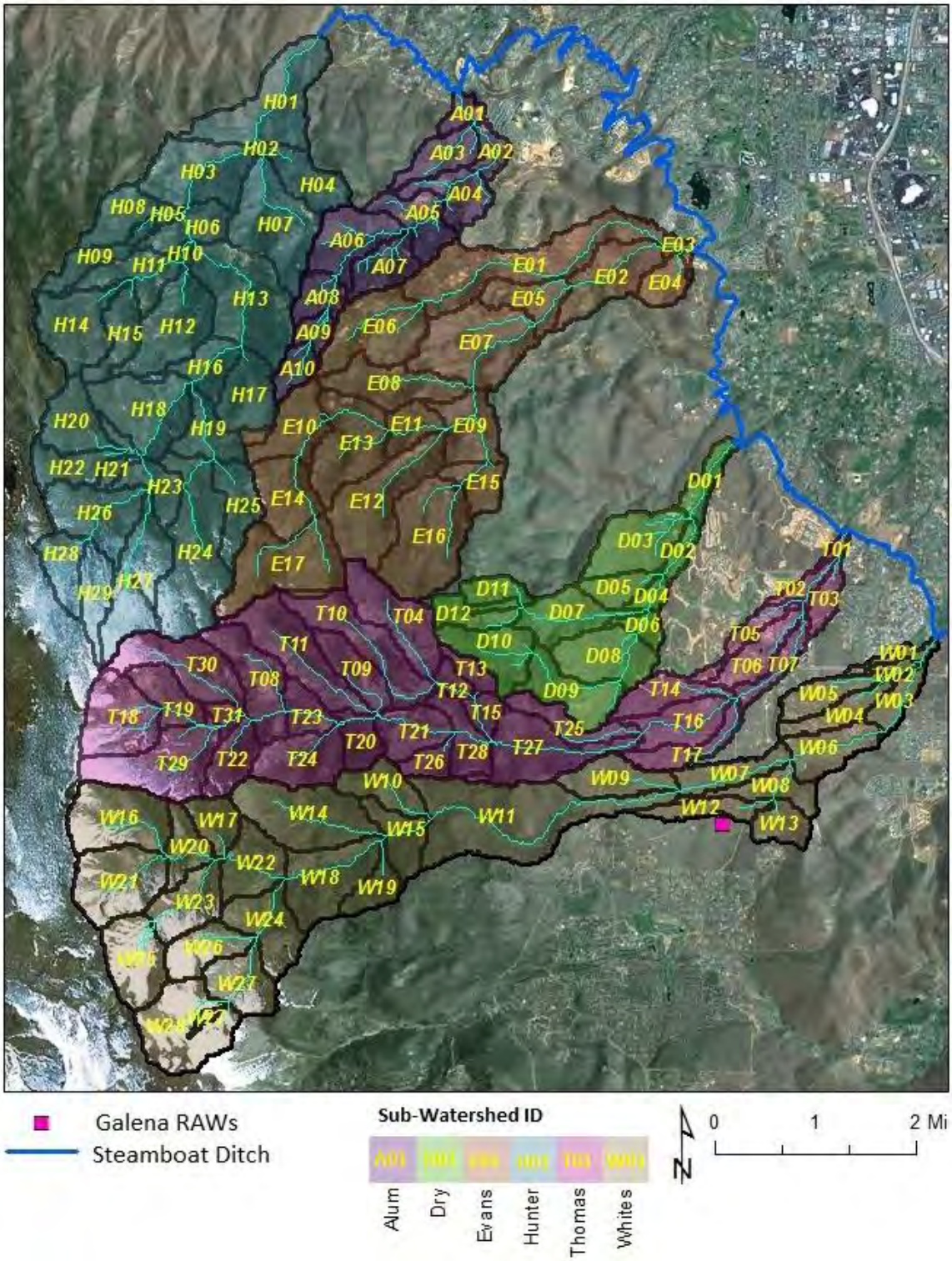

Figure 12: Sub-basin delineation with identification numbers and associated stream reaches modeled with HEC-HMS.

precipitation (Maurer and Berger, 1997). Using the area-weighted CN from Table 2 (and Figure 9) produced modeled stream flows 16 times larger than observed. To compensate, one must either assume that initial CNs are too high or estimated precipitation depths are too large. Observed total stream discharge can only be matched by reducing CNs by more than 50% forcing CN to range from 23 - 41. The calibrated range is extremely low compared to USACE recommendations in which CN is always greater than 30. Instead of adjusting CN, it is assumed that localized storm conditions centered more strongly at the Galena RAWs station. Under these circumstances, total observed discharge can be matched by reducing precipitation by 72% thereby forcing the percentage of discharge to precipitation from 2% to 8% to move more closely to match other studies in the region while simultaneously maintaining a more realistic spatial distribution of CNs.

Hunter Creek contains 29 sub-watersheds (refer to Figure 12). Sub-watersheds were aggregated into three regions; lower, middle and upper, based on their centroid's elevation.

- Lower basin: elevation < 6,500 ft
- Middle basin: 6,500 ft ≤ elevation < 8,000 ft
- Upper basin: 8,000 ft ≤ elevation

The simulated hydrograph shape in Figure 13 was calibrated to match the observed shape by adjusting $t_{lag.}$ Initial estimates of $t_{lag.}$ were calculated using equations (8). A multiplier for each elevation region then scaled these initial guesses. Multipliers for the lower, middle and upper basins equal to 6.0, 11.0 and 15.0, respectively resulted in a root-mean-squared error of 1.4 cfs, or a 4% error (refer to Figure 13)

Hunter Creek area-weighted input parameters for each of the 29 sub-watersheds are provided in Table 5 along with discharge estimates. Calibrated results suggest that the lower basin provides 32% of the volume of water discharged as a result of the rain event, while the middle basin provides 39% and the upper basin contributes 24%.

Peak discharge is provided in cfs and discharge volume is normalized by sub-watershed area and given in inches. While the upper basin receives more than two-times the precipitation compared to the lower basin, upper basin discharge volumes and peaks are generally lower due to much lower CN values and less simulated direct runoff.

8.0 DESIGN STORM RESULTS

Peak discharge rates and total discharge volumes were simulated for a suite of return intervals ranging from a 2-year, 24-hour rain event to the 1,000-year, 24-hour rain event Localized storm conditions were not considered for design storms, nor were area-reduction-factors applied to adjust storm depths as a function of watershed area. Instead, NOAA Atlas-14 total storm depths are assumed to occur across the entire watershed to provide a conservative estimate of discharge.

HEC-HMS could not be calibrated for Alum, Evans, Dry, Thomas or Whites Creek due to lack of observed data. Development of input parameters for these un-calibrated basins

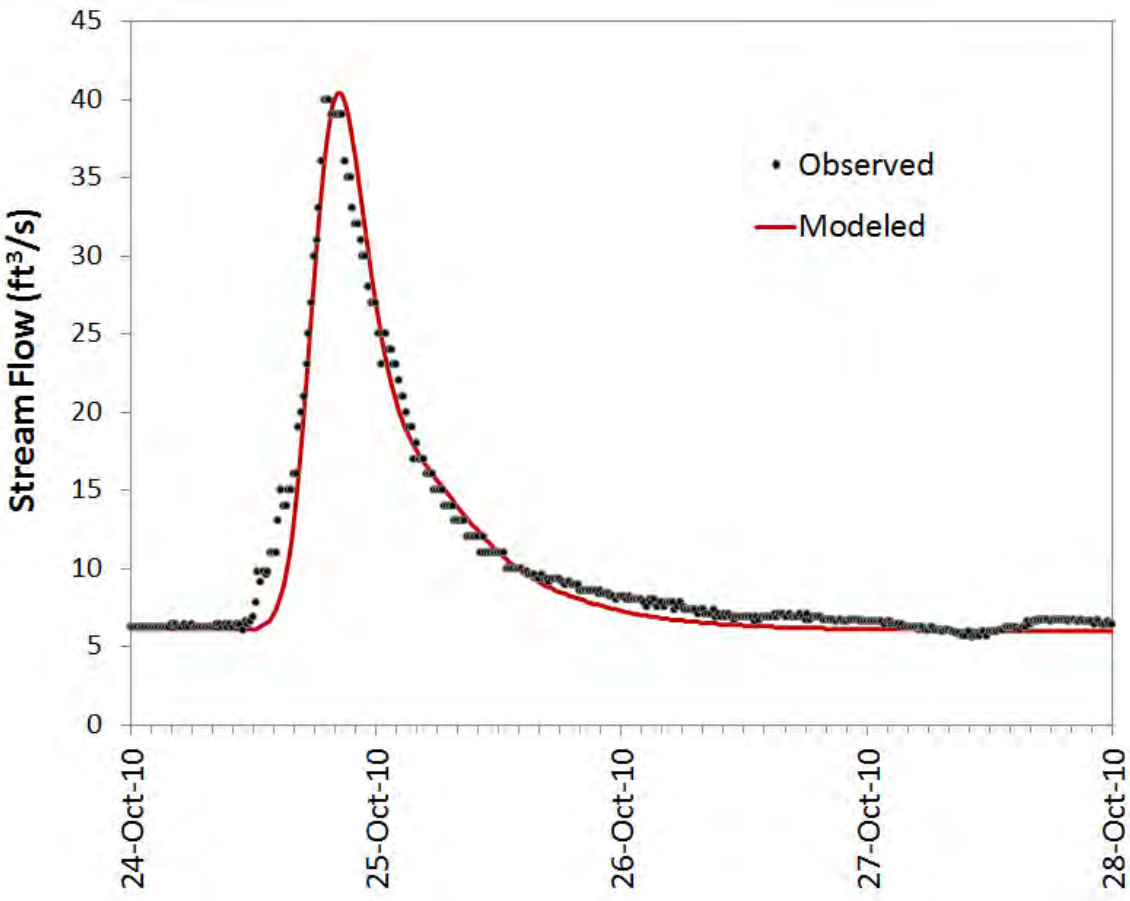

Figure 13: Calibrated discharge for Hunter Creek during a rain-only event.

Table 5: Hunter Creek sub-watershed input parameters for HEC-HMS with scaled precipitation depths and t_{lag} calibrated to historic, rain-only conditions.

ID	Elev.	Slope	Area	CN	t_{lag}	Imprev.	Precip.[a]	Discharge[b]	
	feet	%	mi²		hours	%	inches	inches	cfs
H01	5,512	21	0.76	83	2.9	0.0	0.86	1.3	6.5
H02	5,545	24	0.02	88	0.4	0.0	0.99	0.2	0.5
H03	5,718	26	0.61	78	2.6	0.0	1.17	0.1	7.7
H04	6,299	16	0.31	84	2.2	0.0	1.08	0.2	7.1
H05	6,194	27	0.04	75	1.1	0.0	1.28	0.0	0.7
H06	6,138	23	0.28	77	1.7	0.0	1.29	0.1	4.9
H07	6,758	21	0.71	83	5.5	0.0	1.21	0.1	11.0
H08	7,136	21	0.29	69	4.5	0.0	1.36	0.1	1.0
H09	7,684	15	0.24	61	9.2	0.0	1.47	0.0	0.0
H10	6,335	18	0.03	66	1.4	0.0	1.38	0.1	0.1
H11	6,965	24	0.34	67	6.4	0.0	1.46	0.1	0.9
H12	7,431	20	0.57	64	8.0	0.0	1.53	0.1	0.8

H13	6,821	22	0.82	68	8.1	0.0	1.40	0.1	1.7
H14	8,350	11	0.51	61	12.0	0.0	1.68	0.0	0.5
H15	8,268	17	0.27	64	9.7	0.0	1.63	0.1	0.5
H16	7,270	24	0.27	64	5.2	0.0	1.57	0.1	0.6
H17	8,054	15	0.37	58	10.0	0.0	1.58	0.0	0.1
H18	8,127	17	0.62	65	10.1	0.0	1.75	0.1	2.2
H19	8,087	12	0.29	58	9.4	0.0	1.69	0.0	0.1
H20	8,533	7	0.36	54	13.2	0.0	1.90	0.0	0.1
H21	8,156	8	0.12	57	7.8	0.0	1.89	0.0	0.1
H22	8,583	7	0.26	53	13.6	0.0	2.00	0.0	0.0
H23	8,202	12	0.24	62	8.4	0.0	1.89	0.1	0.9
H24	8,684	9	0.94	55	19.6	0.0	1.96	0.0	0.3
H25	8,346	8	0.32	54	12.9	0.0	1.82	0.0	0.0
H26	8,432	12	0.52	57	10.7	0.0	2.05	0.0	0.8
H27	9,206	11	0.61	53	16.8	0.0	2.13	0.0	0.2
H28	8,980	10	0.27	50	10.7	0.0	2.19	0.0	0.1
H29	9,275	11	0.27	52	13.1	0.0	2.22	0.0	0.1

[a] observed depth of precipitation at Galena RAWs = 2.66 inches

[b] baseflow removed

was done using the same methodology as Hunter Creek. CN distributions from the US ACE (Table 2) were maintained. Lag-time was adjusted based on sub-watershed centroid elevation. Hunter Creek discharge values include an assumed 5 cfs for baseflow. None of the other modeled basin peak discharge rates include a baseflow component.

8.1 100-Year, 24-Hour Design Storm

While a range of design storm return periods are modeled with HEC-HMS, only the 100-year event is evaluated in greater detail to assess where the majority of flow originates within each basin what HEC-HMS parameters are most important in predicting peak discharge and is used to development of a simple regression-model to estimate peak discharge for a 100-year event for un-modeled basins.

Spatial distribution of storm depths for the 100-year, 24-hour event are provided in Figure 14. The figure shows that the greatest rain depths (8-10 inches in 24 hours) occur primarily in the highest elevations found in the upper watershed of Whites Creek and, to a lesser degree, Thomas Creek. Lower elevations in all watersheds are dominated by rain depths of 2 inches to 4 inches, with the areal coverage of these lower depths increasing with movement toward the south. Consequently, the areal coverage of low precipitation in Hunter Creek is less than other basins modeled, while low precipitation in Whites Creek is extensive by comparison.

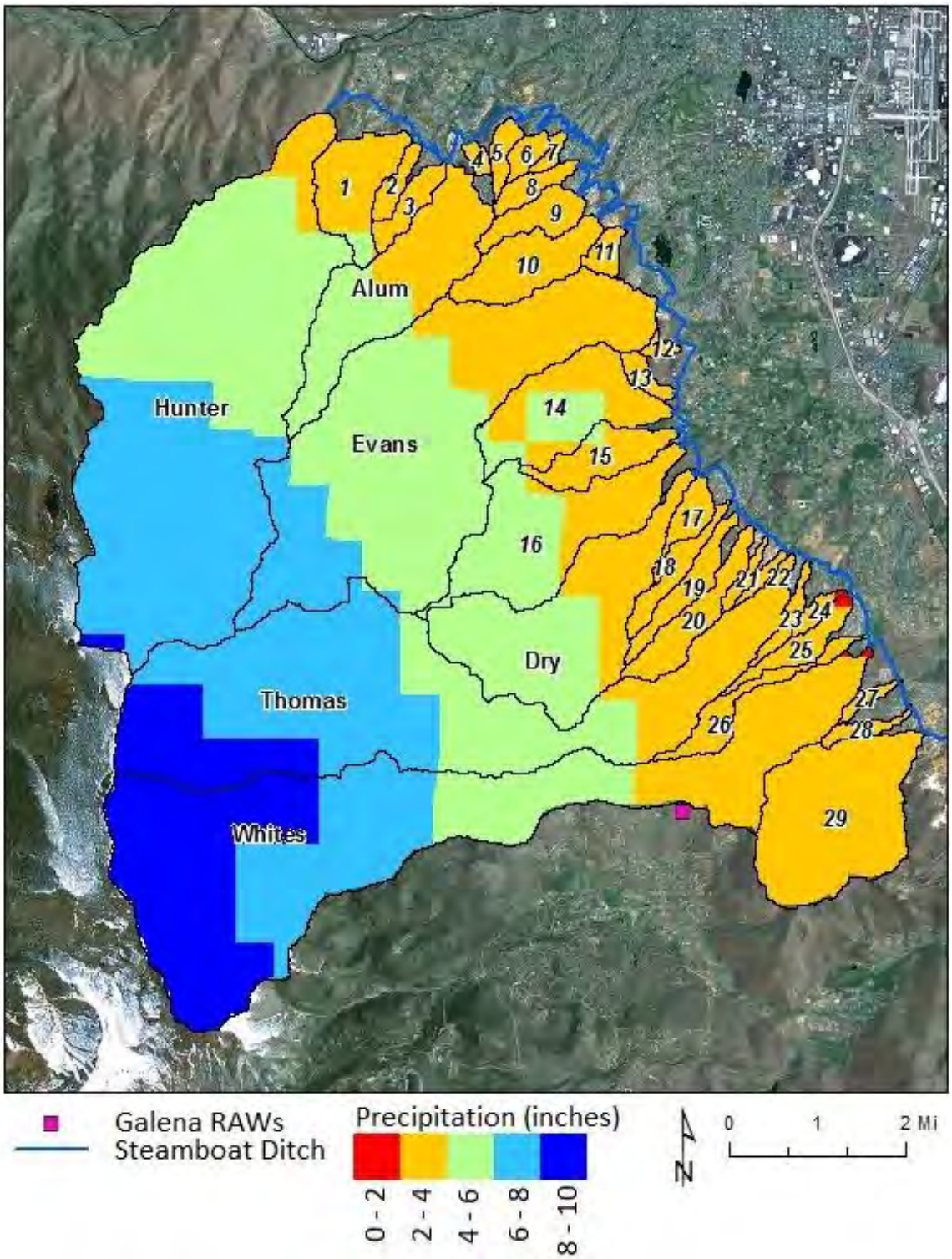

Figure 14: Design storm total precipitation depths for the 100-year, 24-hour event.

8.1.1 Hunter Creek

Hunter Creek CN and t_{lag} parameters provided in Table 5 are maintained for all design storm simulations. Table 6 gives 100-year discharge for each sub-watershed and this is displayed graphically in Figure 15. Discharge volumes are given in inches and represent total discharge volume (not peak discharge in cfs) divided by sub-watershed area. The 100-year event produces a peak discharge at the basin exit equal to approximately 1,160 cfs, or nearly 12-times Steamboat Ditch flow capacity. This peak discharge is only slightly larger than the largest recorded discharge from the basin, 986 cfs, occurring on January 31, 1963. The discharge hydrograph is compared with other modeled basins in Figure 16 with Hunter Creek time to peak discharge occurring at 20 hours from the initiation of the storm event. In general the majority of flow in Hunter Creek originates in the lower portions of the basin. Specifically, H01, H03, H07 and H13 produce large discharge volumes per unit area and peak rates. The lower basin is described mostly with large CNs, large contributing areas and relatively steep slopes. However, in contrast to the calibration event, the upper basin sub-watersheds also contribute significant flow. In particular, H18 and H24 have large discharge volumes and peak rates despite relatively low CN values. This is likely a threshold response to large precipitation depths combined with sub-watershed area forcing significant direct runoff to occur that is not experienced during the the calibration event. As a result of upper watershed contributions, the hydrograph shows fairly sustained flows with discharge greater than 140 cfs lasting until 40 hours after the storm began.

Table 6: Hunter Creek sub-watershed precipitation totals and discharge for a 100-year, 24-hour rain event.

ID	Precip.	Discharge	
	inches	inches	cfs
H01	3.94	5.36	141
H02	4.07	2.19	16
H03	4.49	1.78	92
H04	4.15	1.97	60
H05	4.79	1.82	9
H06	4.92	2.01	42
H07	4.59	2.21	271
H08	5.01	2.51	31
H09	5.45	2.10	37
H10	5.15	2.34	9
H11	5.46	2.68	44
H12	5.78	2.65	71
H13	5.29	2.64	124
H14	6.05	2.61	69
H15	5.97	2.86	34

ID	Precip.	Discharge	
	inches	inches	cfs
H16	6.01	2.83	51
H17	6.04	2.31	43
H18	6.48	3.71	109
H19	6.39	2.82	37
H20	6.70	2.58	34
H21	6.85	3.00	13
H22	6.88	2.58	18
H23	6.96	3.77	47
H24	7.19	3.07	155
H25	6.74	2.62	30
H26	7.22	3.36	74
H27	7.52	3.10	68
H28	7.53	2.78	22
H29	7.63	3.03	34

8.1.2 Alum Creek

Alum Creek sub-watershed area-weighted input parameters and resulting 100-year discharge values are given in Table 7 and displayed graphically in Figure 15. Total modeled discharge exiting Alum creek is approximately 400 cfs, or four times the estimated capacity in Steamboat Ditch. At the sub-watershed scale, A06 produces a peak discharge of 171 cfs, or approximately 40% the peak discharge from the basin. Large peak discharges from A06 are the combined result of poor soils (HSG = D) and a relatively large contributing area (0.44 mi^2). The sum of peak discharges from sub-watersheds A04 and A05 combine to equal more than a third of total peak discharge. Similar to A07, this is the result of larger CNs and sufficient area to generate flow. In contrast, sub-watershed A09 produces the most discharge per unit area (3.5 inches) as a result of a large CN, but its small contributing area keeps it from generating more than 29 cfs peak discharge. Given that a majority of flow originates in the lower basin and that contributions from the upper watershed are limited, the time to peak discharge (refer to Figure 16) occurs relatively quickly at 14 hours and the hydrograph is "flashy" compared to Hunter Creek.

Table 7: Alum Creek sub-watershed input parameters and 100-year, 24-hour rainfall depths and discharge.

ID	Elev.	Slope	Area	CN	t_{lag}	Imprev.	Precip.	Discharge	
	feet	%	mi^2		hours	%	inches	inches	cfs
A01	4,947	12	0.12	74	1.17	12	3.34	1.4	34
A02	5,246	18	0.14	75	1.46	1	3.30	1.2	27
A03	5,538	16	0.27	59	1.93	0	3.53	0.5	15
A04	5,650	18	0.31	77	1.82	0	3.63	1.5	74
A05	5,833	19	0.32	76	2.19	0	3.90	1.7	71
A06	6,414	18	0.44	83	1.98	0	4.41	2.6	171
A07	6,276	20	0.28	48	2.72	0	4.38	0.4	9
A08	7,024	17	0.20	79	4.28	0	5.04	2.8	45
A09	7,401	13	0.12	84	5.14	0	5.21	3.5	29
A10	7,743	8	0.16	53	9.30	0	5.66	1.2	8

8.1.3 Evans Creek

Evans Creek sub-watershed area-weighted input parameters and resulting 100-year discharge values are given in Table 8 and displayed graphically in Figure 15. Simulated results suggest Evens Creek generates approximately 390 cfs during the 100-year precipitation event. Figure 16 shows that time to peak discharge occurs around 16 hours. Peak discharge and time to peak are similar to Alum Creek due to that simulated r contributions of flow occur mostly from the lower portions of the basin. In particular, sub-watershed E07 is the largest contributor to peak discharge, at 128 cfs. Sub-watersheds, E03 and E04 have larger CNs than E07 and produce more volume per area.

This is compounded for E03 which also contains significant development, with impervious area equal to 19%. However, peak discharge from these sub-watersheds were limited to 20 - 51 cfs by their small areal extent. Unlike, Alum Creek, Evans Creek has a larger area extending to higher elevations where precipitation depths are larger. Low CNs, however, limit upper watershed contributions.

Table 8: Evans Creek sub-watershed input parameters and 100-year, 24-hour rainfall depths and discharge.

ID	Elev.	Slope	Area	CN	t_{lag}	Imprev.	Precip.	Discharge	
	feet	%	mi^2		hours	%	inches	inches	cfs
E01	5,495	16	1.00	74	5.7	0.2	3.67	1.4	84
E02	5,098	10	0.48	81	3.3	0.0	3.15	1.4	64
E03	4,888	7	0.07	84	1.4	19.0	2.87	1.7	20
E04	5,023	9	0.18	87	1.7	0.6	2.94	1.7	51
E05	5,525	16	0.29	76	2.3	0.0	3.63	1.5	53
E06	6,968	17	0.63	59	8.0	0.0	4.87	1.2	32
E07	5,902	19	0.76	73	3.0	0.0	4.16	1.6	128
E08	6,808	20	0.45	54	9.9	0.0	5.01	0.9	16
E09	6,345	20	0.37	62	2.8	0.0	4.56	1.2	42
E10	7,818	13	0.39	47	9.7	0.0	5.99	1.0	14
E11	6,719	22	0.21	56	5.0	0.0	5.13	1.1	15
E12	7,484	16	0.74	46	13.5	0.0	5.74	0.8	17
E13	7,421	19	0.60	47	8.7	0.0	5.72	0.8	19
E14	8,166	12	0.41	47	13.6	0.0	6.44	1.1	14
E15	6,758	18	0.24	58	5.0	0.0	4.79	1.1	16
E16	7,595	13	0.80	50	12.2	0.0	5.50	0.9	24
E17	8,514	9	0.71	48	21.4	0.0	6.79	1.4	21

8.1.4 Dry Creek

Dry Creek sub-watershed area-weighted input parameters and resulting 100-year discharge values are given in Table 9 and displayed graphically in Figure 15. Dry Creek has the potential to produce a peak discharge of 740 cfs, or seven times estimated canal capacity. Figure 16 shows that Dry Creek is the "flashest" system modeled, in that peak flow occur quickly (14 hours) and then dissipates quickly. Much of Dry Creek occurs at lower elevations with large CNs and some development (percent impervious reaches about 8%). Therefore, direct runoff is large. Several sub-watersheds contribute significantly to peak discharge, with D03, D07, D09 producing peak flows greater than 100 cfs and D08 potentially contributing more than 230 cfs. Those sub-watersheds in the lower basin with relatively low peak discharges generally contribute the same volume per area discharge, but small contributing areas keeps peak discharge low.

Table 9: Dry Creek sub-watershed input parameters and 100-year, 24-hour rainfall depths and discharge.

ID	Elev.	Slope	Area	CN	t_{lag}	Imprev.	Precip.	Discharge	
	feet	%	mi²		hours	%	inches	inches	cfs
D01	5,026	11	0.18	82	2.25	0.2	3.90	1.5	36
D02	5,325	8	0.10	86	1.83	5.6	4.35	2.2	35
D03	5,525	12	0.49	86	2.44	1.0	5.27	2.3	143
D04	5,499	6	0.09	86	1.62	7.9	3.14	2.5	38
D05	5,781	14	0.19	87	1.55	1.0	3.49	2.7	91
D06	5,584	6	0.11	85	1.88	6.4	3.83	2.5	42
D07	6,270	18	0.42	79	3.21	0.1	4.69	2.4	103
D08	5,830	14	0.47	86	1.63	1.5	4.08	2.9	231
D09	6,368	14	0.41	75	2.75	0.0	5.36	2.2	101
D10	7,005	11	0.39	61	7.55	0.0	5.14	1.5	28
D11	6,972	14	0.16	64	4.94	0.0	3.73	1.7	18
D12	7,411	13	0.19	63	6.91	0.0	4.56	1.7	17

8.1.5 Thomas Creek

Thomas Creek sub-watershed area-weighted input parameters and resulting 100-year discharge values are given in Table 10 and displayed graphically in Figure 15. Thomas Creek produces a relatively low peak discharge (540 cfs) for its contributing area. Figure 16 shows simulated peak discharge occurs at 18 hours and diminishes slowly as a result of upper watershed contributions. Upper watershed contributions come primarily from the northern edge where slopes are steep and soils have limited infiltration capabilities. The largest peak discharges (>110 cfs) occur from T18 and T19 where precipitation depths are about 8 inches in 24 hours and CNs are greater than 53. Lower in the basin, T06, T14 and T16 have CNs greater than 79, and contain developed land with their percent impervious area ranging from 3% to 16%. These sub-watersheds produce peak discharges in excess of 100 cfs.

Table 10: Thomas Creek sub-watershed input parameters and 100-year, 24-hour rainfall depths and discharge.

ID	Elev.	Slope	Area	CN	t_{lag}	Imprev.	Precip.	Discharge	
	feet	%	mi²		hours	%	inches	inches	cfs
T01	4,849	3	0.05	77	2.8	18.9	2.84	2.3	11
T02	5,039	6	0.11	84	2.5	14.2	3.02	2.9	41
T03	5,007	4	0.14	81	3.3	11.4	2.92	2.5	35
T04	7,657	14	0.51	62	8.7	0.0	6.12	4.1	93

T05	5,351	4	0.17	87	3.8	12.6	3.37	3.6	53
T06	5,302	5	0.28	88	3.3	16.4	3.32	3.6	102
T07	5,292	5	0.19	75	4.6	7.1	3.28	2.4	33
T08	8,218	21	0.48	47	12.0	0.0	7.41	3.5	59
T09	7,372	17	0.28	56	7.5	0.0	6.67	4.0	57
T10	7,677	12	0.57	56	14.3	0.0	6.49	3.8	64
T11	7,884	17	0.61	46	13.9	0.0	7.05	3.1	58
T12	7,083	19	0.10	57	4.3	0.0	5.75	3.1	23
T13	7,139	12	0.12	65	5.0	0.0	5.56	3.9	32
T14	5,738	5	0.28	86	3.9	3.0	3.99	4.1	103
T15	6,594	19	0.12	57	3.7	0.0	5.59	3.0	32
T16	5,709	6	0.36	79	3.9	6.5	3.95	3.5	109
T17	5,705	4	0.24	79	5.3	5.4	4.35	4.0	66
T18	9,203	20	0.52	53	8.5	0.0	8.16	5.2	124
T19	8,478	24	0.28	64	6.0	0.0	7.93	6.5	111
T20	7,339	15	0.21	39	9.2	0.0	6.70	1.9	16
T21	6,919	22	0.31	50	6.9	0.0	6.24	2.7	44
T22	8,172	22	0.28	32	13.8	0.0	8.14	1.9	16
T23	7,280	18	0.20	35	9.7	0.0	7.15	1.7	13
T24	7,477	20	0.42	41	9.2	0.0	7.74	3.0	52
T25	6,434	11	0.24	72	4.2	0.4	4.80	3.7	73
T26	6,972	18	0.14	40	7.6	0.0	6.15	1.6	11
T27	6,168	18	0.61	53	6.2	0.0	5.14	2.2	73
T28	6,493	17	0.28	46	4.3	0.0	5.73	1.9	37
T29	8,809	19	0.65	41	17.6	0.0	8.17	3.3	55
T30	8,881	18	0.54	49	15.5	0.0	7.74	4.2	65
T31	7,812	19	0.08	34	6.1	0.0	7.71	1.9	8

8.1.6 Whites Creek

Whites Creek sub-watershed area-weighted input parameters and resulting 100-year discharge values are given in Table 11 and displayed graphically in Figure 15. Peak discharge is 660 cfs with the longest modeled time to peak at 23 hours (Figure 16). In contrast to several basins modeled, much of the direct runoff originates in the upper basin. The lower basin has large CNs and percent impervious area and can produce moderate volumes per unit area (> 2 inches) but are limited in both discharge volume per area as well as peak discharge given lower precipitation depths, shallow slopes and smaller contributing areas. The largest discharge volumes per unit area and peak rates occur in sub-watersheds W16, W21, W25 and W29 where precipitation is in excess of 8.4 inches and CNs are greater than 53.

Table 11: Whites Creek sub-watershed input parameters and 100-year, 24-hour rainfall depths and discharge.

ID	Elev.	Slope	Area	CN	t_{lag}	Imprev.	Precip.	Discharge	
	feet	%	mi²		hours	%	inches	inches	cfs
W01	4,947	4	0.10	78	7.86	23.7	2.85	1.4	7
W02	4,970	5	0.06	78	4.05	20.7	2.86	1.4	7
W03	5,010	5	0.18	75	6.89	9.2	2.91	1.1	10
W04	5,190	4	0.23	84	7.76	29.4	3.12	2.1	23
W05	5,200	4	0.26	83	8.03	16.3	3.13	1.8	22
W06	5,233	6	0.34	81	5.74	12.7	3.20	1.7	35
W07	5,551	4	0.28	78	8.96	12.4	3.77	2.0	24
W08	5,456	6	0.27	83	6.65	20.3	3.61	2.3	34
W09	5,876	9	0.34	80	5.98	2.2	5.30	3.2	67
W10	7,490	16	0.29	48	9.37	0.0	6.55	1.3	15
W11	6,811	16	1.28	54	19.02	0.6	5.56	1.2	37
W12	5,620	3	0.21	79	10.51	15.2	4.14	2.4	19
W13	5,476	5	0.20	82	3.73	12.0	3.66	2.2	38
W14	7,510	23	0.73	38	13.63	0.0	7.74	1.0	22
W15	6,818	19	0.29	45	8.35	0.0	6.39	0.9	11
W16	8,976	24	0.60	52	9.92	0.0	8.39	2.8	66
W17	8,300	27	0.22	34	10.04	0.0	8.26	0.9	7
W18	7,349	24	0.57	38	12.42	0.0	7.14	0.7	14
W19	7,595	20	0.19	42	7.95	0.0	6.68	0.9	7
W20	8,130	27	0.24	39	9.31	0.0	8.25	1.3	12
W21	8,904	27	0.58	53	8.11	0.0	8.57	3.0	79
W22	7,592	24	0.37	34	8.81	0.0	7.90	0.7	10
W23	8,599	27	0.39	41	12.69	0.0	8.28	1.5	18
W24	8,041	23	0.42	41	13.39	0.1	7.65	1.2	16
W25	9,488	21	0.46	53	8.22	0.0	8.76	3.1	66
W26	9,216	23	0.32	45	11.25	0.0	8.24	1.9	21
W27	8,714	24	0.38	44	11.82	0.0	8.06	1.7	22
W28	9,829	18	0.65	56	9.05	0.0	8.69	3.4	94

8.1.7 Basins Modeled with a Regression

HEC-HMS models were not developed for 29 smaller basins residing along Steamboat Ditch (refer to Figure 1) but 100-year, 24-hour, peak discharge estimates are obtained using regression analysis. HEC-HMS input parameters and simulated peak discharge rates were used to generate the regression. A correlation analysis found, unsurprisingly, CN and sub-watershed area the most important factors describing simulated peak discharge. Larger areas and larger CNs produce larger peak discharge rates. The influence of slope, while directly related to peak discharge, was only nominally so. Precipitation, in many cases,

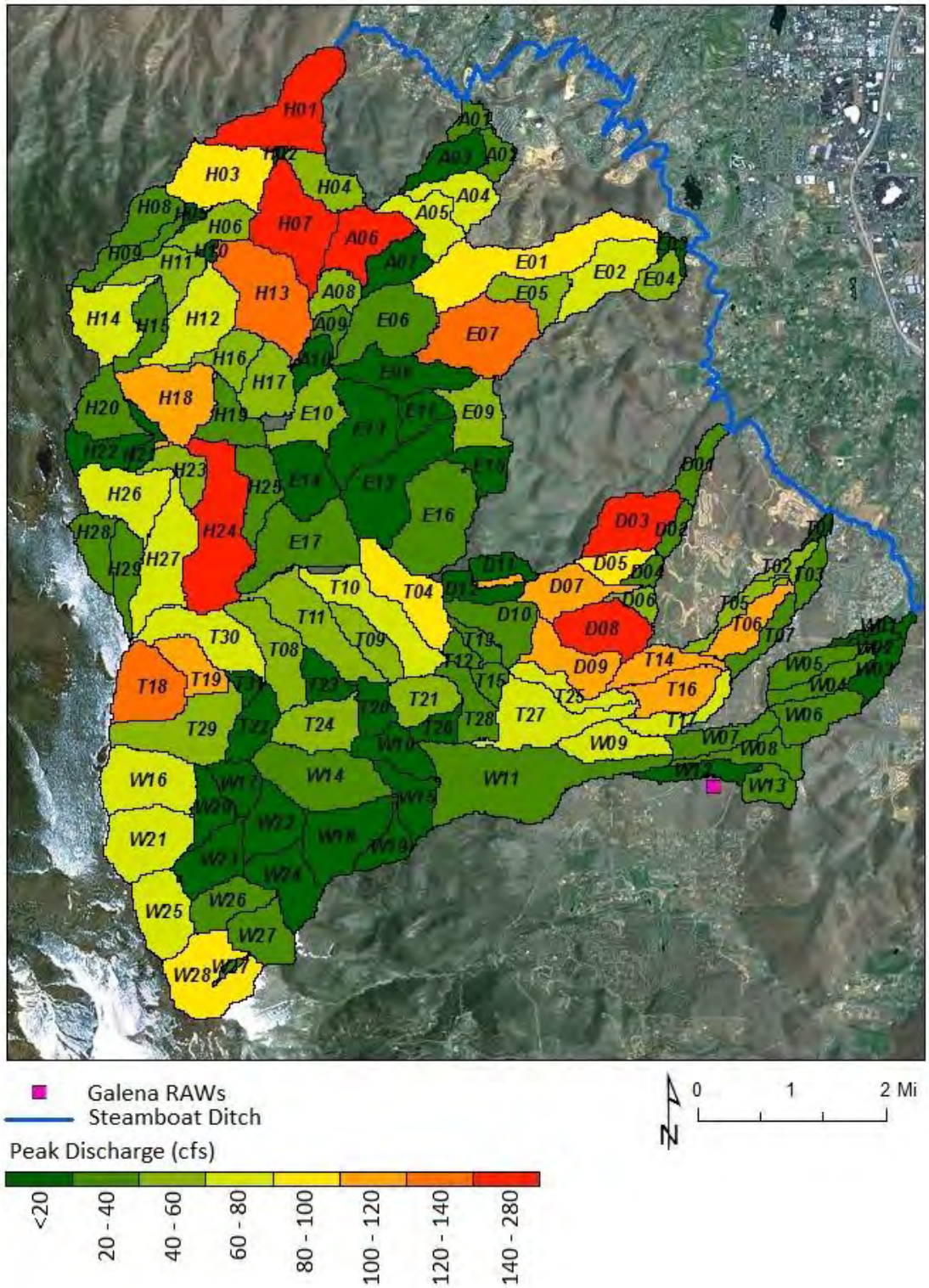

Figure 15: Peak discharge for the 100-year, 24-hour precipitation event for all sub-watersheds modeled with HEC-HMS. Sub-watershed identification numbers superimposed.

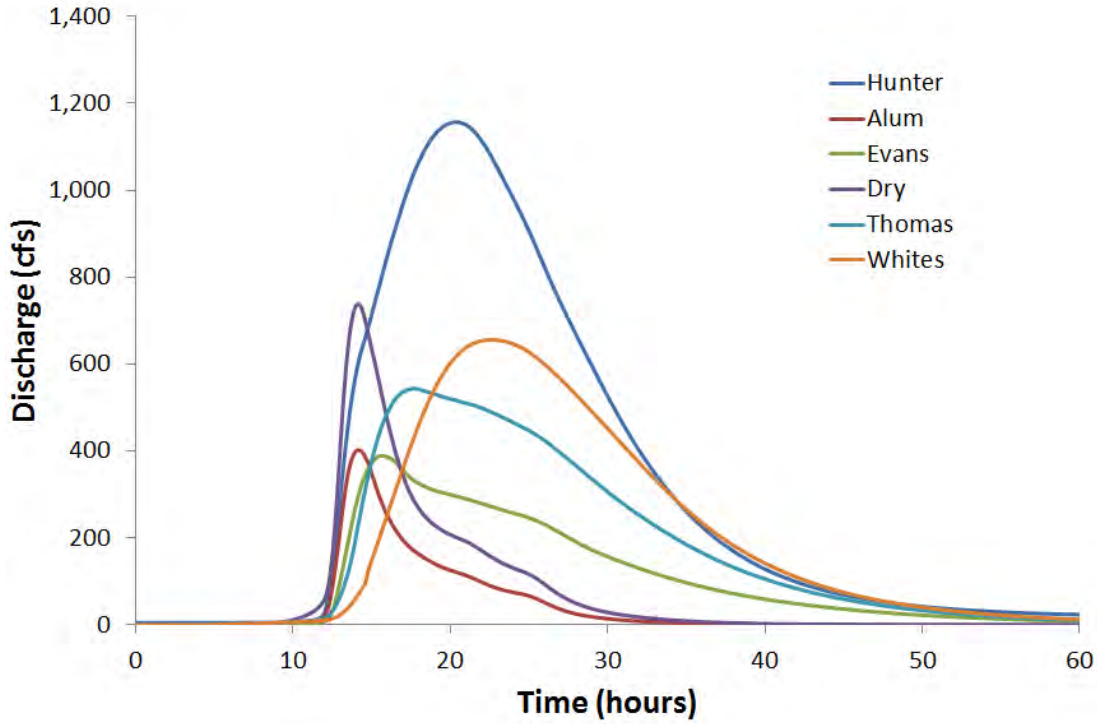

Figure 16: Modeled hydrographs for the 100-year, 24-hour design storm. Time represents hours since the start of the precipitation event.

was indirectly related to peak discharge due to the influence of CN, which tends to be lower at higher elevations to reduce the correlation between precipitation and discharge rates. HEC-HMS modeled sub-watersheds are biased toward non development and including the percent impervious area in the regression analysis provided no predictive capabilities. However, the basins located in close proximity to Steamboat Ditch not modeled with HEC-HMS contain more impervious area than those modeled with HEC-HMS (refer to Figure 5) and it is likely that the regression built to predict mostly undeveloped watersheds under predicts those that are.

The final regression model was done using only data pertaining to Alum, Evans and Dry Creek (n = 39) based on the notion that these three basins are more representative of basins quantified with the regression approach (i.e. similar elevations, soils, vegetation, precipitation). Using the limited data set resulted in an exponential regression

$$Y = 0.1749e^{0.0736CN}, \qquad (11)$$

with an $r^2 = 0.92$. Y is the normalized metric of flow (cfs) per unit area (mi^2) per unit precipitation depth (inches). Input parameters and estimated peak discharges for those basins modeled with equation (11) are shown in Table 12 and are compared to HEC-HMS modeled basin discharges in Figure 17. Basins expected to flood Steamboat Ditch

Table 12: Parameter input information and resulting peak discharge estimates for basins modeled with a regression. ID numbers are provided in Figure 1.

ID	Approx. Location (closest cross streets)	Area mi^2	CN	Precip. inches	Discharge cfs	Exceed Capacity[a]
1	Caughlin Ranch Footbridge	0.95	88	3.47	361	Y
2	Caughlin Parkway & Promontory	0.2	88	3.43	80	N
3	Sawbuck Rd. & Bitteroot Rd.	0.3	88	3.38	113	L
4	S. McCarren Blvd.	0.09	78	3.55	17	N
5	Bridgewater Dr. & Ashfork Dr.	0.18	85	3.18	51	N
6	Southampton Dr. & Prescott Way	0.17	83	3.57	47	N
7	Susileen Dr. & Markridge Dr.	0.09	80	3.73	21	N
8	Gully between Skyline View Dr. & San Mateo Ave.	0.24	89	3.32	99	N
9	Gibraltar Dr. & Heavenly Valley	0.43	86	3.48	152	Y
10	Gully between Sage Ridge Dr. & Skyline Blvd.	0.9	88	3.24	338	Y
11	S. McCarren Blvd. & Beksera Ct.	0.17	86	3.50	57	N
12	Lurie Ln. & Faretto Ln.	0.07	88	5.89	47	N
13	Cassas St. & MacFarlane Dr.	0.15	88	3.07	51	N
14	Gaspari Rd & Bellhaven Rd.	1.68	84	3.70	518	Y
15	Bellhaven R.	0.64	86	3.43	222	Y
16	Brunswick Mill Rd.	2.34	88	3.28	850	Y
17	Lamary Ln. & Lamary Cir.	0.29	82	4.07	86	N
18	Slide Mtn. Dr. & Ventana Pkwy.	0.36	89	2.96	128	L
19	Ventana Pkwy. & Marble Canyon Rd.	0.43	86	2.89	122	L
20	Ventana Pkwy. & north of Marble Canyon Rd.	0.64	72	2.78	63	N
21	Ventana Pkwy. & Mesa Rd.	0.12	88	3.32	44	N
22	Ventana Pkwy.	0.08	88	3.06	27	N
23	Ventana Pkwy. & Gila Bend Rd.	0.18	76	2.89	25	N
24	Fieldcreek Ln. & W. Zolezzi Ln.	0.12	87	3.36	44	N
25	W. Zolezzi Ln. & Edmands Ct.	0.37	76	2.91	49	N
26	Fieldcreek Ln. & Wolf Run Golf	0.48	76	2.87	67	N
27	Arrow Creek Pkwy. & Demonte View Ln.	0.09	79	3.30	17	N
28	Quicksilver Dr.	0.15	75	2.80	18	N
29	Steamboat Hills	2.74	84	3.14	724	Y

[a] N = not expected to cause flooding, Y = expected to cause flooding, L = likely to cause flooding

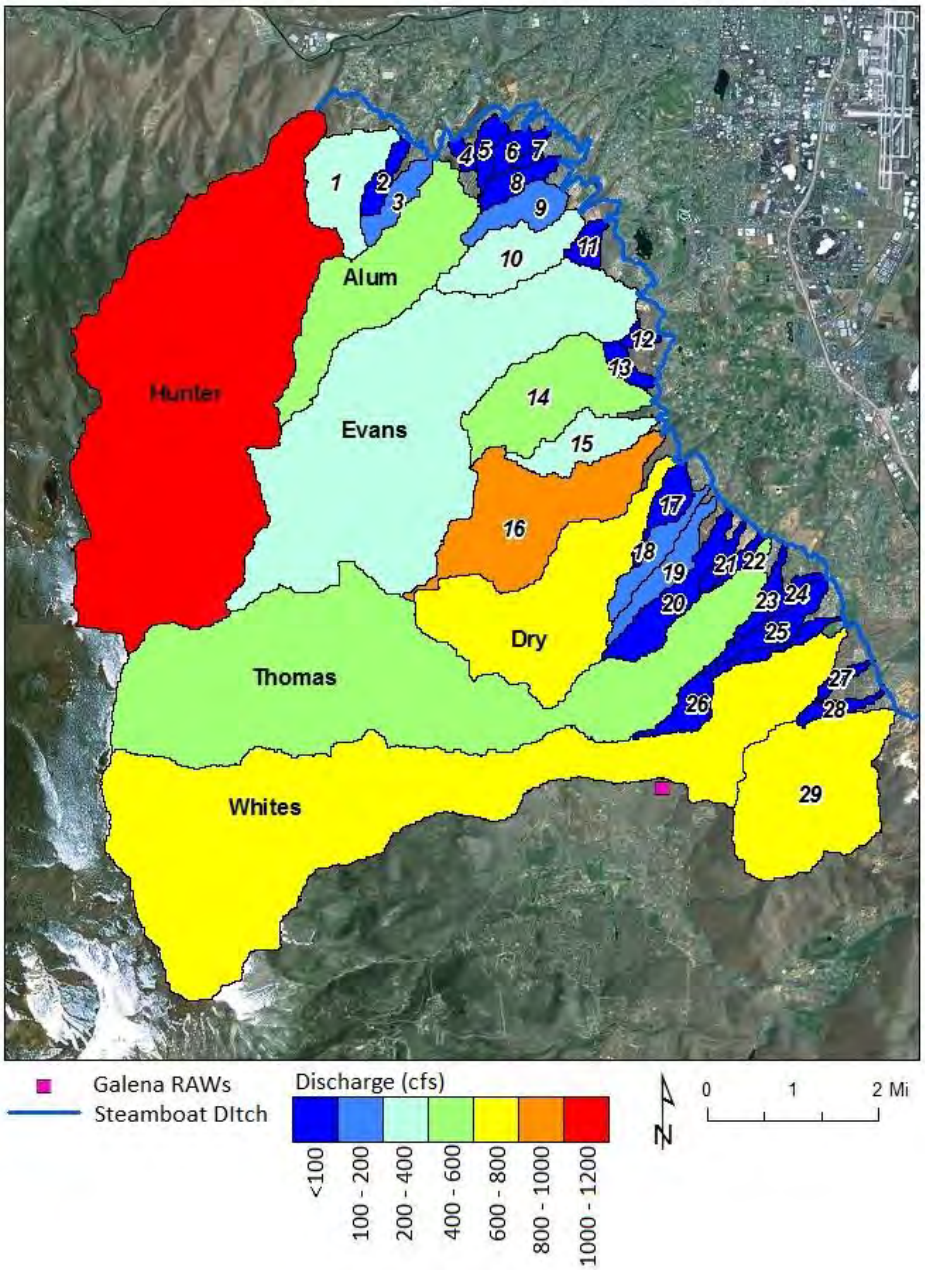

Figure 17: HEC-HMS and regression derived peak discharge a rain-only 100-year, 24-hour design storm.

have simulated discharges larger than 131 cfs, while those likely to produce flooding have simulated discharges larger than 100 cfs, but less than 131 cfs. Flooding of Steamboat Ditch is not expected for flows less than 100 cfs. To further illustrate, Figure 18 shows the potential for these basins to exceed ditch capacity.

8.1.1 Rain-on-Snow

Historically, rain-on-snow events create the largest floods along the Sierra Nevada Mountains (e.g. 1963, 1997, 2005). The purpose of including HEC-HMS rain-on-snow simulations in the analysis is to provide a relative magnitude shift in potential peak discharge toward Steamboat Ditch, as well as highlight those conditions most susceptible to flooding during the winter months.

A simple sensitivity analysis was done for each of the snowmelt parameters provided in Table 4. The metric for evaluation was fractional change in predicted peak discharge to fractional change of input parameter. BASE, PX and T_{air} proved the most sensitive parameters in runoff prediction. Given BASE and PX use regionally calibrated values, these were held constant during comparative modeling. The Galena air temperature (T_{air}) however, is completely subjective, and allowed to vary to test model response.

HEC-HMS simulations were run over a five-day period, with the 100-year, 24-hour storm occurring on the first day. Separate simulations were conducted for Galena air temperature ranging between 34°F and 56°F at 3°F increments for the 24-hour period of the storm. Plots of peak discharge versus T_{avg} are plotted in Figure 18. While T_{avg} may be larger than BASE, sub-watersheds may have temperatures lower than BASE, based on temperature lapse rate and sub-watershed centroid elevation, and no melt will occur.

The shape of the temperature-runoff curve provides insight into which basins extend to higher elevations and which do not (Figure 19). Low elevation watersheds (e.g. Alum, Evans and Dry) have the potential to generate discharge events larger than rain-only events at relatively low valley temperatures (T_{avg} ~ 35°F - 38°F) but tend to plateau in discharge contribution with increasing temperature based on a diminishing snowpack. Rain-on-snow events that occur on watersheds that extend to higher elevations (e.g. Hunter and Whites Creek) can generate larger discharge peaks compared to rain-only events, but only when valley temperatures are relatively high (T_{avg}~ 44°F). Otherwise, temperatures in the upper elevations are not sufficient to promote rain and snowmelt. In contrast to lower-elevation basins such as Alum, Evans and Dry Creek, Hunter, Thomas and Whites Creek have large depths of initial SWE in their upper reaches and discharge rates are not limited with rising temperature. Thomas Creek combines both upper- and lower-elevation systems by allowing large discharge from the lower and middle portions of its watershed at fairly low valley temperatures, but also generating large flows at higher temperatures based on ample snowpack in its upper reaches. Whites Creek shows an intermediate plateau. This plateau occurs when all snowpack is melted in the lower portions of the watershed and is only contributing rain, but temperatures are not yet high enough for the upper watershed to generate rain and snowmelt.

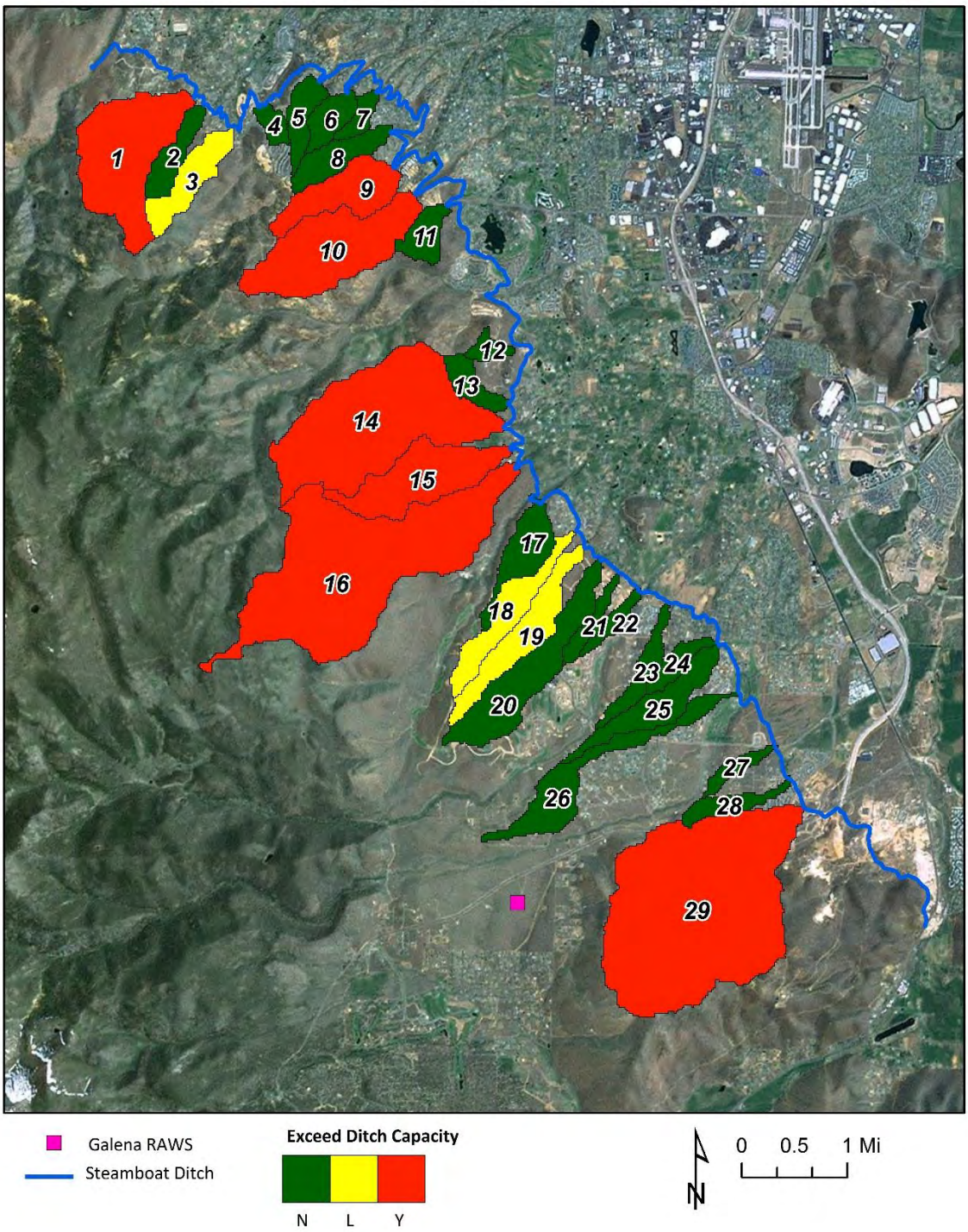

Figure 18: Potential for basins modeled with equation (11) to generate discharge that exceeds Steamboat Ditch flow capacity. N = no, L = likely and Y = yes.

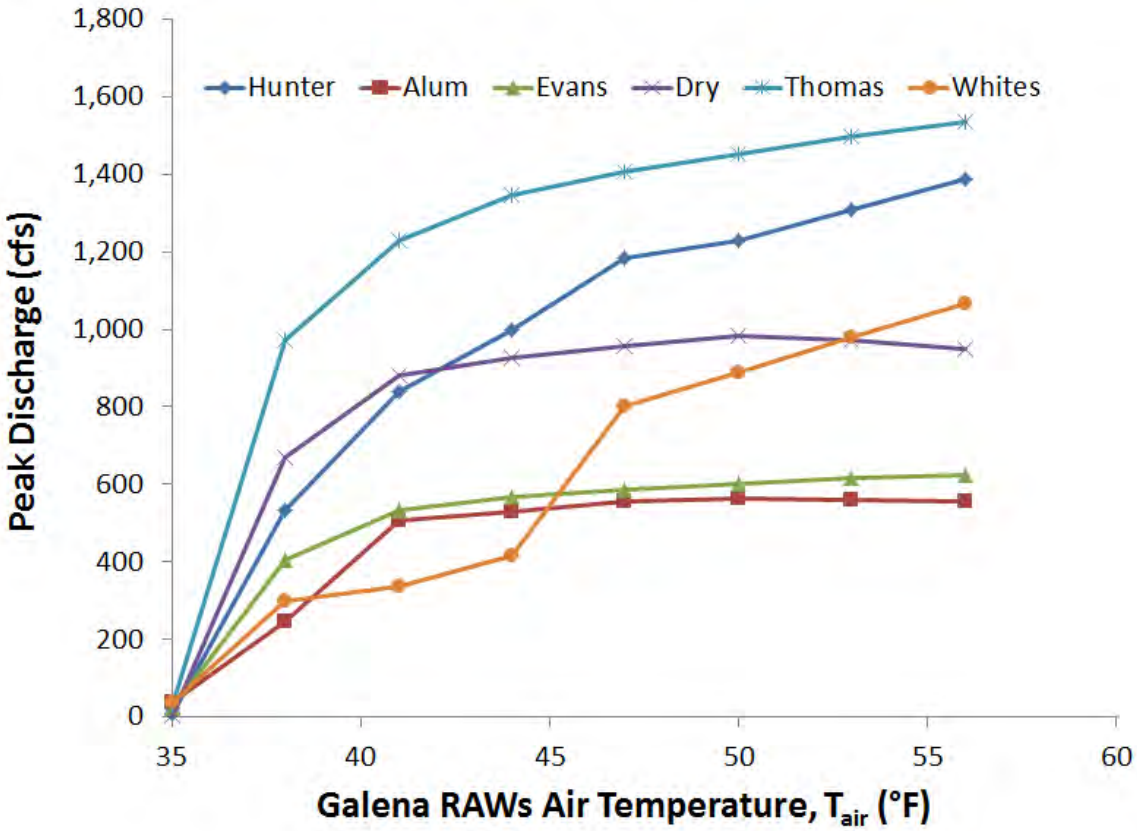

Figure 19: Rain-on-snow 100-year, 24-hour event peak discharge as a function of area-weighted temperature.

8.2 Range of Return Periods

HEC-HMS modeling was also done for a range of return periods spanning 2-years to 1,000 years with peak discharges provided in Table 13. Also included are the 100-year peak discharges for rain-on-snow simulations assuming Galena air temperature equals 47°F. With respect to rain-only simulations, Hunter Creek produces significantly larger discharge values for all return periods. Peak discharge ranges from approximately 220 cfs at the 2-year return period up to nearly 2,200 cfs given a 1,000-year return period. Dry Creek produces the second most discharge for return intervals less than 500-years. Whites Creek surpasses Dry Creek discharge only for very large return intervals of 500-years or greater. Alum Creek and Evans creek produce similar peak discharges for all return intervals and fall at the lowest end of simulated results. Thomas Creek peak discharges are similar to Whites Creek but values diverge toward lower discharge values compared to Whites Creek with increased return period.

Hunter Creek is simulated as least susceptible to large changes in peak discharge as a result of snowmelt with stream flow increasing by only 2%. Thomas Creek appears to be

most susceptible to snowmelt with modeled values increasing 160% over rain-only projections.

HEC-HMS, rain-only results presented in this study are generally less conservative than previous model estimates (Nimbus, 1990; Manhard, 2010) in that predicted discharge from major contributing watersheds are 8% to 4.5-times lower. However, HEC-HMS is calibrated to Hunter Creek at the spatial and temporal scale believed representative of the system and results line up well with the historical record. Nonetheless, even with lower predictions in discharge, all major drainages are simulated to produced enough discharge to surpass estimated ditch capacity even at fairly modest return intervals (i.e. 5-years or greater).

Table 13: Peak discharge (cubic feet per second, cfs) for each modeled watershed given a range of return periods.

Return Period Years	Hunter	Alum	Evans	Dry	Thomas	Whites
2	220	60	40	160	80	80
5	340	110	90	260	140	150
10	560	170	140	360	210	250
25	840	250	220	490	320	370
50	1,060	320	300	610	430	500
100	1,160	400	390	740	540	660
200	1,480	490	490	870	680	830
500	1,840	630	650	1,070	920	1,080
1,000	2,190	740	780	1,230	1,080	1,420
100 (rain-on-snow)	1,180	560	590	960	1,400	800
% increase with snowmelt	2	40	51	30	159	21

9.0 MANAGEMENT RECOMENDATIONS

Model results suggest that all major drainages traversing Steamboat Ditch have the potential to contribute significant discharge during even modest storms to cause flooding along Steamboat Ditch. During the 100-year event, estimated peak discharge from major drainages exceeds ditch capacity by 3.9 to 11.6 times. Simulated discharges are largest for Hunter Creek (1,160 cfs) and lowest for Evans Creek (390 cfs). Dry Creek is modeled as most responsive, or flashy, while Whites Creek has the largest lag-time to peak discharge. Flooding is expected to be exacerbated during rain-on-snow events but is extremely sensitive to air temperature distribution across the study site. Of the six basins

modeled, Thomas Creek has the greatest potential to see a significant rise in peak discharge as a result of snowmelt. Smaller drainages also have the potential to flood Steamboat Ditch with 10-unnamed basins identified out of 29 investigated. Locations of potential flooding from these smaller drainages occur (1) within the Caughlin Ranch sub-division, (2) the major drainages in the vicinity of Skyline Blvd., (3) rural regions west of Lakeside Drive and north of Dry Creek, (4) the Arrow Creek sub-division and (5) Streamboat Hills south of the Mt. Rose Hwy (Hwy. 431).

Several major drainages (e.g. Hunter, Alum, Whites) are conveyed beneath Steamboat ditch. The ability to convey the 100-year discharge through these stream crossings should be assessed to ensure adequacy of structure. For conveyance structures that are too small, likelihood of water backing up and overtopping Steamboat Ditch will likely occur.

Using Steamboat Ditch as a mechanism to convey stream discharged directly into the ditch, or discharge that has overtopped the crossing structure, may quickly overwhelm the ditch and force a breach. Therefore, preliminary recommendations are to:

- Prioritize those areas where drainage needs to be diverted away from the ditch based on localized ditch condition, number and location of outflow drains along the ditch to diffuse peak runoff across areas less prone to damage and potential risk of damage below the ditch if flooded.
- It is not recommended to use Steamboat Ditch to divert elevated winter flows from the Truckee River as a storm water management tool.
- Engineering solutions may be needed to improve ditch conditions and/or improve stream crossings from major drainage basins.

10.0 REFERENCES CITED

Bonnin, G.M., Martin, D., Lin, B., Parzybok, T., Yekta, M., and Riley, D. 2011. Precipitation-frequency atlas map of the United States. Vol 1 version 5.0: semiarid southwest (Arizona, Southeast California, Nevada, New Mexico, Utah. U.S. Department of Commerce. National Oceanic and Atmospheric Admininstration, nationa Weather Service. 271p.

FEMA. 2013. Flood insurance study, Washoe County, Nevada and incorporated areas. 222 p.

Gray and Associates, 2003. Ditch Survey for Steamboat Ditch, Prepared for City of Reno, Public Works Department, 103p.

Kennedy/Jenks Consultants, 2004. Survey of the Steamboat Ditch through Washoe County, Prepared for Steamboat Canal and Irrigation Company, 63 p.

Manhard Consulting, 2010. Planning-Level Regional Hydrologic Analysis – Final Report, Prepared for the City of Reno, 341p.

Maurer, D.K., and D. L. Berger (1997), Subsurface flow and water yield from watersheds tributary to Eagle Valley Hydrographic area, west-central Nevada, U.S. Geological Survey Water-Resources Investigations Report 97-4191, 56 p.

Nimbus Engineering. 1990. Hydrologic analysis for Thomas Creek, Dry Creek and the Boyton Slough, Washoe County, Nevada. Prepared for the Federal Emergency Management Agency contract #EMW-89-C-2841. 137p.

NRCS. 1986. Urban Hydrology for Small Watersheds, TR-55. United Stated Department of Agriculture Conservation Engineering Division. 164 pp.

USACE. 1998. HEC-1 flood hydrograph package user's manual. Hydrologic Engineering Center, Davis, CA.

USACE. 2000a. HEC-HMS hydrologic modeling system user's manual. Hydrologic Engineering Center, Davis, CA.

USACE. 200b. Hydrologic modeling system HEC-HMS Technical Reference Manual. Hydrologic Engineering Center, Davis CA. 158p.

USACE. 2013. HEC-GeoHMS: Geospatial hydrologic modeling extension, User's Manual. Version 10.1. Hydrologic Engineering Center, Davis CA. 193p.

USDA. 2009. Chapter 7: Hydrologic Soil Groups. Part 630 Hydrology National Engineering Handbook. Natural Resources Conservation Service. 210-VI-NEH.

3.3.9 Flooding Along Irrigation Ditches and Canals

3.3.9.1 Nature

Figure 3-26. Flooded Intersection in a Residential Area of SW Reno.

Flooding was caused by overflow of Last Chance Ditch in 2006.

Most flooding occurs along canals and ditches in areas not mapped or designated by FEMA as floodplain because the canals and ditches are not natural waterways, but manmade waterways. Most canals and ditches in Nevada were constructed in the late 1800s and early 1900s to deliver agricultural water from Nevada's few rivers to otherwise dry farms and ranches located at some distance from those rivers. Since that time, urban expansion has extended to areas adjacent to these irrigation ditches such that in some places they are now surrounded by residential, commercial, industrial, and other development. In some cases, buildings have been built immediately adjacent to ditches and are often at elevations lower than the elevations of the ditches. The ditches are generally operated for several months of the year from spring to early fall, often coinciding with times of heavy rainfall and runoff that exacerbate flooding.

A listing of most ditches and canals in Nevada and their county and topographic quadrangle is shown in Appendix Q and available online at this link:

http://nevada.hometownlocator.com/features/cultural,class,canal,startrow,1.cfm

Failure of canals and ditches can be aggravated by any of the following factors acting individually or in combination:

- Structural weakness of levee walls
- Overwhelming by excess storm water runoff
- Strong earthquake shaking
- Rodent burrowing
- Diversion of river water into ditches during storms

- Clogging with debris

Canal and ditch failures can lead to inundation of homes and businesses as well as damage to personal property, such as parked cars, stored motor homes and warehoused materials. Infrastructure damage can include roads and utilities, as well as the structural damage to the canal or ditch itself. In some cases, the flooding can be deep and fast enough to endanger the lives of people who are caught in it or who might attempt to cross flooded areas.

3.3.9.2 History

There have likely been many breaks in ditches and canals in Nevada that were unrecorded because there were few people to witness or little property that was damaged. Most damage has been caused by severe storms, but some canal failure has been documented by strong shaking from earthquakes.

Storm runoff into a canal can quickly increase hydraulic loads and potentially overwhelm its water-carrying capacity. The rapid increase in storm water can find the weaker parts of a canal embankment, potentially causing them to fail. Rodent burrowing was a contributing factor in weakening the canal embankment that failed after heavy rains on January 5, 2008 in Fernley and flooded 800 homes. This canal failure resulted in a presidential disaster declaration.

Strong ground motion or surface rupture from earthquakes can also damage and fail canal embankments. In the July 6, 1954 Rainbow Mountain earthquake near Fallon, canal embankments liquefied and flowed into the canal and several breaches also occurred to embankments in the canal system (Steinbrugge and Moran, 1956). During the April 25, 2008 Mogul earthquake, an elevated section of the Chalk Bluffs Ditch was collapsed by a rockslide. Fortunately, the water flowed into a small runoff channel and away from homes in the immediate area. There are sparse records of flooding associated with canal and ditch failure; some instances are listed in Table 3-19.

Table 3-19. Nevada Canal and Ditch Flood History

Date	Location	Description; injuries; damage cost
6-Jul-1954	Fallon, NV	Canal damage caused by earthquake; embankment failures; 0 injuries; ~$1M estimated cost
1-Jan-1997	Reno, NV	At least 5 washouts of ditch; 0 injuries; no estimates of cost
31-Dec-2005	Reno, NV	Emergency declaration; Multiple canal breaks and houses flooded;0 injuries; no estimates of cost
5-Jan-2008	Fernley, NV	800 homes flooded; >1500 people evacuated;0 injuries;
25-Apr-2008	Mogul, NV	Canal damage caused by earthquake; 0 injuries; $1.8M cost
11-Jun-2008	Fernley, NV	Disaster declaration 1738; Substantial damages; 0 injuries; Approximate cost $3.5M

3.3.9.3 Location, Severity, and Probability of Future Events

Generally, the greatest hazard from canal and ditch flooding is in developed areas, although damage to an important roadway in a rural environment can have serious consequences as well. Flooding in open fields or on agricultural lands is usually limited to economic loss of crops or livestock. Within developed areas, the most vulnerable structures are those closest to the canals or ditches, in potential flood paths, or where flooding may be concentrated. Within areas of dense building development, flooding will commonly follow the relatively unrestricted roadways. Where the flood can spread out, become shallower in water depth, and have a lower flow velocity, the hazard and severity decreases. Areas that can be the most severely affected occur where canal or ditch flow can join other water flow, such as at stream channel intersections.

The probability of flooding from canals and ditches is largely tied to large storm events, occurrence of local earthquakes, and to development adjacent to ditches and canals. The probability of recurrence is likely based on historical events, unless mitigation activities such as new codes and regulations for land use planning are successful in restricting new development in flood-prone areas adjacent to ditches and canals.

In the western United States, climate change has led to warmer overall climate conditions compared to what has been observed in the past, with the trend is expected to continue. Nevada will likely see more frequent flooding events under a warmer climate, as snow levels on average, will be higher during winter storms, resulting in more precipitation falling as rain over river basins. This will allow much larger portions of river basins to contribute to runoff, leading to higher flows resulting in more frequent flooding events. In addition, warmer air can hold more moisture (water vapor) which can potentially be converted into heavy precipitation, making flood events more extreme in the future.

Table 3-20. Flood Mitigation Goals and Strategic Actions

Goal/Lead Agency	#	Strategic Action	Changes and reason for (a) modification, or (b) deletion
Goal 1: ***Reduce the loss of life and injuries*** ***Nevada Division of Emergency Management and Nevada Hazard Mitigation Planning Committee***	1.A	Improve awareness of the locations, potential impacts and links among hazards, vulnerability and measures to protect life safety and health.	
	1.B	Provide current information and workshops about hazards, vulnerabilities, mitigation processes and technical assistance for planning and grant availability and application procedures to State and local agencies.	
	1.C	Encourage the incorporation of mitigation measures into repairs, major alterations, new development and redevelopment practices.	
	1.D	Promote the modification of structures to meet life safety standards.	
	1.E	Improve communication, collaboration and integration among stakeholders and promote hazard mitigation as an integrated public policy.	
	1.F	Encourage local governments, special districts and tribal organizations to develop, adopt, implement maintain and update hazard mitigation plans.	The words “maintain and update” were added because a majority of local plans are developed or in progress and will require only maintenance

			and updating from now on
	1.G	Develop a hazard communication system that can be used to rapidly detect and provide early warning for multiple hazards, including earthquakes and wildfires.	
Goal 2: ***Improve Local Hazard Mitigation Plans Technical Assistance***	2.A	~~Promote local hazard evaluation and mitigation planning and assist in developing local hazard mitigation plans~~ Provide technical assistance, guidance, resources and tools to local governments and tribal entities to promote hazard evaluation and to develop and update hazard mitigation plans.	Combined 2a and 2b The majority of local plans are developed or area in progress and will require only updating from now on; while most tribes still require plan development.
Nevada Division of Emergency Management and Nevada Hazard Mitigation Planning Committee	~~2.B~~	~~Provide technical assistance, guidance, resources and tools to local governments for all aspects of local hazard mitigation planning~~	Combined 2a and 2b The majority of local plans are developed or in progress and will require only updating from now on; while most tribes still require plan development.
	2.B	Provide specialized training and exercises to state agency staff and local governments concerning local hazard mitigation planning and the local hazard mitigation plan program.	
	2.C	~~Develop~~ Maintain a tracking system for local and state government mitigation plans and projects.	Deleted word ”Develop” and added “Maintain” because plan is already developed; and requires only maintenance
	2.D	Provide training to local governments and state agency staff to clarify mitigation measures from response and recovery and preparedness measures.	
	2.E	~~Develop~~ Maintain a system to allow state agencies with hazard mitigation programs and plans to make recommendations about how local governments can incorporate these in support of the state’s mitigation program efforts.	Deleted word ”Develop” and added “Maintain” because NHMPC is this system that has been implemented and requires only maintenance
	2.F	Continue to build operational links between hazard mitigation, disaster preparedness and recovery programs with public and private sectors	
	2.G	Promote understanding by the general public of the benefits of hazard mitigation in reducing casualty and property losses and ensuring continuity of businesses, institutional and government functions	
	2.H	Promote coordination among state agencies, local governments and tribal organizations of regional hazard mitigation activities	
	2.I	Identify, enhance and integrate public education efforts by state and local agencies that have programs directed to hazard	

		mitigation	
Goal 4: ***Reduce the possibility of damage and losses due to flooding*** ***Div. of Water Resources, NHMPC***	4.A	Protect existing assets, as well as future development, from the effects of flooding.	
	4.B	Identify and prioritize areas in the State where existing flood hazard mapping is inadequate due to planned and existing significant development and conduct flood hazard mapping in those areas.	
	4.C	Conduct flood hazard mapping in piedmont and alluvial fan environments.	
	4.D	Retrofit State buildings to meet NFIP standards.	
	4.E	Assist communities and State with programs to elevate, dry-flood proof or wet-flood proof identified structures to obtain NFIP compliance and/or mitigate repetitive loss structures and severe repetitive loss structures.	Added SRL structures
	4.F	Assist communities and State with programs dealing with repetitive loss structures and severe repetitive loss structures; these programs may involve acquisition and demolition; relocation; elevation or other mitigation strategies.	Added SRL structures and broadened possible mitigation strategies to deal with them.
	4.G	Upgrade State-owned or operated infrastructure (e.g. servicing roads, culverts, bridges, channels, and structures) related to State-owned or operated critical facilities to protect critical facilities from flood damages or disruption of essential services.	
	4.H	Protect existing assets as well as future development from the effects of dam failure.	
	4.I	Maintain inventory of existing dams and add to the inventory as dams are discovered or constructed.	Added "Maintain" because inventory is already developed; and requires only maintenance
	4.J	Inventory and inspect existing dams for structural and hydraulic adequacy and implement operational constraints, if warranted.	
	4.K	Install early warning weather stations in watersheds with dams above populated areas.	
	4.L	Assist communities and State in structural mitigation measures, updates, repairs and maintenance to dams, ditches, and canals.	Added the words "maintenance, ditches, and canals" to incorporate mitigation activities for canals and ditches.
	4. M	Encourage local ordinances and regulations to reduce encroachment into flood-prone zones resulting from dam impoundment or high (non-failure) releases.	
	4.N	Identify hazards of flooding from man-made structures, such as irrigation ditches and canals, and integrate these into local zoning ordinances.	
	4.O	Develop laws and regulations that ensure reasonable standards of design and construction to reduce flood hazards.	
	4.P	Develop Emergency Action Plans to ensure swift coordinated response in the event of an emergency.	